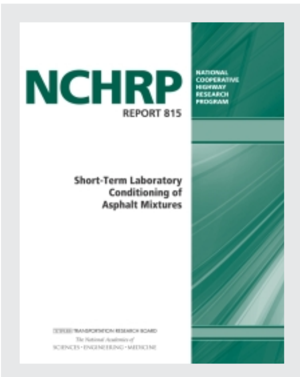

Short-Term Laboratory Conditioning of Asphalt Mixtures

DETAILS

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

NCHRP REPORT 815

Short-Term Laboratory Conditioning of Asphalt Mixtures

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TRANSPORTATION RESEARCH BOARD

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2015
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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

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FOREWORD

By Edward T. Harrigan
Staff Officer
Transportation Research Board

This report presents proposed changes to AASHTO R 30, *Mixture Conditioning of Hot Mix Asphalt (HMA),* and a proposed AASHTO practice for conducting plant aging studies. Thus, the report will be of immediate interest to materials engineers in state highway agencies and the construction industry with responsibility for design and production of hot and warm mix asphalt.

Laboratory conditioning of asphalt mixtures during the mix design process to simulate their short-term aging influences the selection of the optimum asphalt content. In addition, conditioning affects the mixture and binder stiffness, deformation, and strength evaluated with fundamental characterization tests to assess mixture performance. The conditioning process must also address the rapidly increasing use of warm mix asphalt (WMA), for which production temperatures are substantially lowered from those required for hot mix asphalt (HMA). Past research using limited field data found that 2 hours of oven conditioning at the anticipated compaction temperature reasonably reproduced the binder absorption and stiffening that occurs during construction for both WMA and HMA. As would be expected, the binder in a WMA mixture conditioned 2 hours at its lower compaction temperature is less stiff than that in a similarly conditioned HMA mixture. Such reduced aging is significant in interpreting WMA performance test results compared to those of HMA for both laboratory- and field-produced mixtures. The current standard conditioning procedure, AASHTO R 30, *Mixture Conditioning of Hot Mix Asphalt (HMA),* was developed over two decades ago and was reviewed to determine whether it required updating to address today's asphalt materials and mix production processes.

The objective of this research was to develop procedures and associated criteria for short-term laboratory conditioning of asphalt mixtures—both HMA and WMA—for mix design and performance testing to simulate the effects of (1) plant mixing and processing to the point of loading in the transport truck and (2) an initial period of field performance. The research was performed by the Texas A&M Transportation Institute, Texas A&M University, College Station, Texas, in conjunction with the National Center for Asphalt Technology, Auburn, Alabama, and the Pavement Research Center, University of California, Berkeley, California.

The short-term aging of asphalt mixtures was investigated through a series of statistically designed laboratory and field experiments. The research confirmed a key finding of NCHRP Project 9-49, "Performance of WMA Technologies: Stage I—Moisture Susceptibility" (see *NCHRP Report 763*), that the effects of plant mixing and processing to the point of loading in the transport truck are well simulated by 2 hours of oven aging at either 275°F (135°C) for HMA or 240°F (116°C) for WMA. Further, the research found that an initial field performance period of 1 to 2 years (depending on the specific project climate) is well simulated by 5 days of oven aging at 85°C (185°F). This latter finding calls into question the original recommendation from research conducted by the Strategic Highway Research Program, i.e., that these conditions were equivalent to 5 to 10 years of field aging. Using resilient modulus as a metric, the field experiment found that the stiffness of WMA was initially lower than that of HMA for the majority of the field projects, but it caught up to HMA stiffness in about 17 months in warmer climates and 30 months in colder climates.

The report recommends using cumulative degree-days (heating, base 32°F[0°C]) rather than time in service to accurately quantify field aging and account for differences in construction dates and climates for various field sites. Practical outcomes of the project are presented as (1) proposed changes to AASHTO R 30 and (2) a proposed AASHTO practice to measure the effects of asphalt plant mixing and processing on binder absorption by aggregate and asphalt mixture characteristics in the field.

This report fully documents the research and includes seven appendices.

CONTENTS

SUMMARY

Short-Term Laboratory Conditioning of Asphalt Mixtures

The stiffening of asphalt mixtures over time due to oxidation, molecular agglomeration, and other chemical processes is referred to as aging. Aging occurs due to the heating of the binder during production and construction in the short term and to continued oxidation during its service life. The ability to simulate aging in asphalt binders and mixtures produced in the laboratory has been studied extensively, and procedures have been adopted for use in binder specifications and mixture design. In the past, the aging of asphalt mixtures has been assumed to be relatively consistent, with laboratory procedures resulting in the same degree of aging as would take place during production or during long-term aging in the field, depending upon the particular protocol. While the comparison of stiffening in laboratory-produced and plant-produced mixtures was never an exact match, there was an acceptance that the laboratory aging was representative of field aging. However, this occurred in an environment in which the amount of recycled materials was relatively low, polymer-modified asphalts were not common, and mixing temperatures were mostly in a consistent range.

Recent changes in asphalt mixture components, mixture processing, and plant design have left many paving technologists questioning the validity of current mix design methods in adequately assessing the volumetric needs of asphalt mixtures and the physical characteristics required to meet performance expectations. While a variety of other research studies either attempted or are attempting to address many of these issues, this project considered the impact of climate, aggregate type (to reflect differences in asphalt absorption), asphalt type and source, recycled material inclusion (reclaimed asphalt pavement [RAP] and recycled asphalt shingle [RAS]), warm mix asphalt (WMA) technology, plant type, and production temperature on the volumetric and performance characteristics of asphalt mixtures during construction and over an initial period of performance.

The objectives of NCHRP Project 9-52 were to (1) develop a laboratory short-term aging protocol to simulate the aging and asphalt absorption of an asphalt mixture as it is produced in a plant and then loaded into a truck for transport and (2) develop a laboratory long-term aging protocol to simulate the aging of the asphalt mixture through its initial period of performance. Accordingly, the project was divided into two phases: Phase I evaluated short-term aging protocols for hot mix asphalt (HMA) and WMA to simulate plant aging, and Phase II evaluated long-term aging protocols to simulate the aging of the asphalt mixture 1 to 2 years after construction. In addition to developing these mixture protocols to simulate the short-term and long-term aging occurring in the field, the research study sought to discern the effects of the following variables on aging: (1) WMA technology, (2) aggregate asphalt absorption, (3) plant temperature, (4) plant type, (5) presence of recycled materials, and (6) asphalt source.

Nine field sites in eight states (Figure S-1) were constructed to provide the materials necessary to complete this research. Each field site had one or more of the previously listed

Figure S-1. Locations of field sites used in NCHRP Project 9-52.

variables as its primary study component. Mineral aggregates and binders were sampled ahead of production and construction in order to fabricate laboratory-mixed, laboratory-compacted (LMLC) specimens to replicate conditions at mixture design; mixtures produced by the asphalt plant were sampled and compacted on or near the job site to provide plant-mixed, plant-compacted (PMPC) specimens; and roadway cores were obtained immediately after construction and, if possible, at intervals up to 2 years after construction.

Guidance on laboratory short-term oven aging (STOA) protocols for preparing LMLC specimens was obtained from the results of NCHRP Project 9-49, "Moisture Susceptibility of Warm Mix Asphalt" (Epps Martin et al. 2014). From the various STOA protocols investigated, the best match in mixture stiffness between LMLC specimens and PMPC specimens or cores at construction was obtained by conditioning loose mixtures for 2 hours at 240°F (116°C) and for 2 hours at 275°F (135°C) prior to compaction for WMA and HMA, respectively. These short-term aging protocols were adopted for this research effort. Two common long-term oven aging (LTOA) protocols on compacted LMLC specimens were evaluated: (1) 5 days at 185°F (85°C) per AASHTO R 35 and (2) 2 weeks at 140°F (60°C), which was included in NCHRP Project 9-49 (Epps Martin et al. 2014). On a limited basis, an additional LTOA protocol of 3 days at 185°F (85°C) was also evaluated.

The effects of both short-term and long-term aging on mixtures were primarily evaluated using the resilient modulus (M_R) test per ASTM D7369 at 77°F (25°C) and the Hamburg wheel-tracking test per AASHTO T 324 at 122°F (50°C). In addition, a limited amount of dynamic modulus (E*) testing was conducted in accordance with AASHTO TP 79-13. The M_R test was effective for evaluating the effects of aging because (1) the binder properties govern the M_R stiffness and (2) it can be used directly to compare the stiffness of cores and LMLC and PMPC specimens.

The STOA protocols developed under NCHRP Project 9-49 were validated across the wide range of mixture components and production parameters represented in this study. As seen in Figure S-2, there is an agreement in the M_R stiffness values at 77°F (25°C) between LMLC samples and PMPC specimens, which indicates the laboratory samples achieved a degree of aging similar to that caused by heating and mixing in the asphalt plants.

Factors affecting the short-term aging of asphalt mixtures included asphalt source, recycled material inclusion, aggregate absorption, and WMA technology. The effect of asphalt source on the magnitude of aging was demonstrated with the materials from the field site in West Texas. The presence of RAP and RAS in mixtures was found to significantly increase

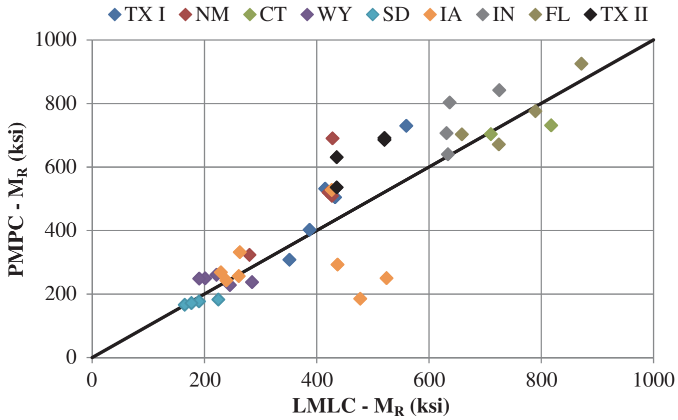


Figure S-2. Comparison of resilient modulus of LMLC and PMPC samples.

the stiffness of the material through the plant. Increased aggregate absorption was found to actually lower the initial stiffness of mixtures due to increased effective asphalt content dictated by volumetric mixture design requirements. WMA technologies produced lower stiffnesses in plant mixtures, primarily because of the presence of the technologies combined with lower production temperatures. Factors having no significant effects on the short-term aging of asphalt mixtures were plant type and plant production temperature, which was varied by as much as 30°F (17°C) within HMA and WMA.

The modeling of LTOA of asphalt mixtures is more challenging as the number of variables affecting the degree of aging increases. For instance, mixture parameters such as total air voids, the interconnectivity of air voids, asphalt binder film thickness, and asphalt source interact in complex ways with the field in-service temperature and time. To capture aging in the field, the use of cumulative degree-days (CDD) was proposed as a field metric that allowed the analysis to account for both in-service temperature (i.e., climate) and time. As defined herein, the CDD is the sum of the daily high temperature above freezing for all the days being considered from the time of construction to the time of core sampling. Thus, mixtures placed in two different climates, say New Mexico and Iowa, could be considered to age differently over the same period.

Aging taking place in the field can be quantified by a property ratio that is simply a certain property of the mixture at the time of sampling in the field relative to that same property at the initial stage (i.e., at construction). This allows consideration of the rate of aging by tracking the property ratio with the CDD. Asphalt mixtures with a higher property ratio demonstrate a higher rate of aging over a given period. As with STOA, M_R proved to be an adequate measure for evaluating the effect of field aging.

Seven of the sites from Phase I were included in the Phase II experiment. The M_R ratio for all field sites included in Phase II of the work is plotted against CDD in Figure S-3. As might be expected, a great deal of aging occurs early in the pavement's life, and then the rate of aging decreases as time passes.

As previously discussed, two LTOA protocols were selected for extensive evaluation in this project, and an additional protocol was tried on a limited basis. As shown in Figure S-4, for the Florida and Indiana field sites, the equivalent CDD values for various LTOA protocols determined based on M_R ratio results showed that the 5 days at 185°F (85°C) LTOA protocol stiffened mixtures more than the 2 weeks at 140°F (60°C) LTOA protocol, and the 3 days at 185°F (85°C) LTOA protocol was very similar to the longer 2 weeks protocol. Thus, the 5 days at 185°F (85°C) protocol represented a higher number of CDD at a given site compared to the other two protocols.

One of the questions this study attempted to answer was, "When does the stiffness of WMA equal that of HMA in the field?" Of the field sites included in Phase II of this project,

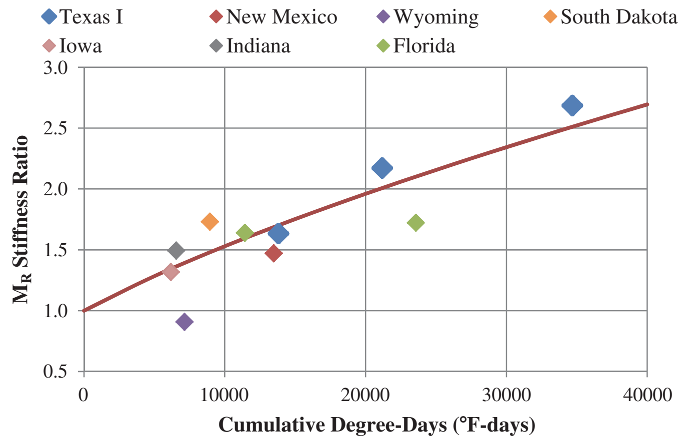


Figure S-3. Performance ratio versus cumulative degree-days for Phase II field sites.

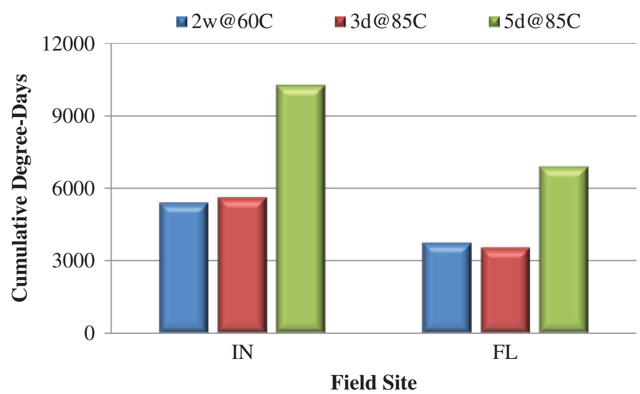


Figure S-4. Cumulative degree-days for Phase II field sites.

three different behaviors were exhibited for the aging of WMA compared to HMA. In one instance, the aging of WMA proceeded along a stiffening path parallel to that of HMA. For four of the field sites, WMA started at a lower stiffness but then intersected the HMA stiffness at CDD values between 16,000 and 28,000. Finally, in two field sites, the stiffness of the WMA mixtures was equivalent to the stiffness of the HMA mixtures over time.

In terms of factors affecting the long-term aging of the asphalt mixtures, it was found that the inclusion of recycled materials, aggregate absorption, and WMA technology were all significant factors, while plant type and production temperature were not as significant, which was consistent with the short-term aging study. The presence of recycled mixtures produced a lower rate of aging, as might be expected since aging has already occurred in the recycled materials. Mixtures with aggregates having higher asphalt absorption showed a higher rate of aging than those with lower asphalt absorption. WMA mixtures demonstrated a high rate of aging over the first 1 or 2 years of field service.

In summary, this project was successful in validating the proposed STOA protocol suggested for preparing WMA and HMA mixtures in the laboratory compared to plant-mixed, laboratory-compacted mixtures over a wide range of mixture components and production parameters. Certain LTOA protocols were developed to simulate aging in the field for a period of 1 to 2 years, depending upon the in-service climate. It is recommended that these field sites be further monitored to understand the long-term behavior of WMA and HMA mixtures.

CHAPTER 1

Background

Asphalt mixtures may be produced in either batch or drum mix plants and then compacted at temperatures ranging from 220°F (104°C) to 325°F (163°C) (Kuennen 2004; Newcomb 2005a). The goal of asphalt mixture production is to ensure complete drying of the aggregate, proper coating and bonding of the aggregate with the binder, and adequate workability for handling and compaction. These processes are important to the mixture's durability, resistance to permanent deformation, and cracking. Recent advances in asphalt technology—including the use of polymer-modified binders, use of more angular aggregates, and increased compaction requirements—have resulted in increased mixing and compaction temperatures up to a limit of approximately 350°F (177°C), where polymer breakdown in the binder can occur. The use of warm mix asphalt (WMA) technology can lead to reduced production and paving temperatures without sacrificing the quality of the final product. This has led to a wider range of available production temperatures that may be employed by the contractor. The conditions under which hot mix asphalt (HMA) or WMA is produced and placed may have a profound impact on its performance properties.

Traditionally, asphalt mixtures have been designed on the basis of volumetric parameters (Asphalt Institute 1984, 1995) in which the optimum asphalt content for a given aggregate gradation was dependent upon the compaction effort, air void (AV) content, voids in mineral aggregate, and absorption of the asphalt into aggregate voids. Laboratory mixing and compacting temperatures were dependent upon the stiffness or viscosity of the asphalt binder. This system was refined over decades of experience, first for the Marshall and Hveem procedures, and then for Superpave. The volumetric procedures worked well for agencies and contractors in an era when the components for asphalt mixtures were relatively constant. In the last two decades, changes have occurred in these components, and these changes are beneficial but leave the asphalt community in need of changing its practices on how mixtures are designed and evaluated. Increased use of polymer modifiers, increased use of reclaimed asphalt pavement (RAP) and recycled asphalt shingle (RAS), and the advent of WMA are departures from the norm under which volumetric mixture design was developed.

Compounding this complexity has been an evolution in asphalt plant design. The 1970s saw a rapid and persistent increase in the number of continuous plants (or drum mix plants [DMPs]) replacing batch mix plants (BMPs). In a BMP, the aggregates are dried prior to being loaded into hot bins in a batching tower. Gates on the bins are opened, allowing the proper proportion of aggregates to be weighed in a weigh bin prior to dropping into the pugmill. Asphalt binder is fed into the pugmill, usually through a weigh bucket. The aggregate and binder are then mixed and discharged either into a truck or onto a slat conveyor for loading into a silo. The material is then delivered to the job site via dump truck for deposition and paving. A variation in BMP design is for aggregates to be dried and then fed into a separate continuous pugmill, and then placed into a silo.

In general, a continuous plant differs from a batch plant in that cold aggregates are fed onto a weight belt in the proper proportions prior to entering the elevated end of the drum for drying. The aggregates are dried as they tumble through the drum toward the lower end, where they are mixed with binder before exiting to a slat conveyor for loading into a silo where the mixture is kept until it is deposited into a truck for transport to a paving site. The drum may either have the burner directed in parallel with the downward flow of the aggregate or directed upward against the flow of the aggregate. These designs are referred to as parallel-flow drum and counter-flow drum (CFD), respectively. There are two other variations for continuous plants. One is a dryer mixing drum, in which the aggregate is dried in a drum before it is loaded into a pugmill at the end of the drum for mixing with asphalt. The other variation is the unitized drum mixer design, which is a variation of the CFD in which the aggregate is dried as it goes through an inner drum. At the end of the inner drum, the material flow is reversed, feeding the aggregate into an outer drum where it is mixed with asphalt before being discharged.

As discussed, the changes in mixture components, mixture processing, and plant design have left many paving technologists questioning the validity of current mix design methods in adequately assessing the volumetric needs of asphalt mixtures and the physical characteristics required to meet performance expectations. While a variety of other NCHRP projects (9-43, 9-47A, 9-48, 9-49, and 9-49A) either attempted or are attempting to address many of these issues, this project considered the impact of climate, aggregate type (to reflect differences in asphalt absorption), asphalt type and source, recycled material (RAP and RAS) inclusion, WMA technology, plant type, and production temperature on the volumetric and performance characteristics of asphalt mixtures during construction and over an initial period of performance.

Project Objectives and Scope

The objectives of NCHRP Project 9-52 were to (1) develop a laboratory short-term aging protocol to simulate the aging and asphalt absorption of an asphalt mixture as it is produced in a plant and then loaded into a truck for transport and (2) develop a laboratory aging protocol to simulate the aging of the asphalt mixture through its initial period of performance. Accordingly, the project was divided in two phases: Phase I evaluated short-term aging protocols for HMA and WMA to simulate plant aging, and Phase II evaluated long-term aging protocols to simulate the aging of the asphalt mixture 1 to 2 years after construction.

This chapter presents the background and literature review summary on short- and long-term laboratory conditioning of asphalt mixtures, including related work on conditioning completed in NCHRP Project 9-49 (Epps Martin et al. 2014). Chapter 2 focuses on the research approach and details of the field sites used in this project. Findings for Phases I and II of the project are presented in Chapter 3, with detailed test results and analyses provided in the appendices. Conclusions and recommendations of this project are in Chapter 4.

Previous Research on Short-Term Conditioning

The aging of asphalt binders in mixtures has long been a concern to those in the pavement field. Several studies dating back to the 1960s and further evaluated the short-term aging of asphalt mixtures in the field, and a consistent conclusion was obtained that most short-term aging of asphalt mixtures occurred during production and construction due to the high temperatures involved (Heithaus and Johnson 1958; Traxler 1961; Chipperfield and Welch 1967). More recent studies have identified several factors of asphalt mixtures and production parameters that could significantly affect the short-term aging characteristics of asphalt mixtures; these factors include binder source, binder type, aggregate absorption, plant type, production temperature, and silo storage time (Traxler 1961; Chipperfield and Welch 1967; Lund and Wilson 1984, 1986; Topal and Sengoz 2008; Zhao et al. 2009; Morian et al. 2011; Rashwan and Williams 2011; Mogawer et al. 2012; Daniel et al. 2014). A brief summary of these studies on the short-term aging of asphalt mixtures is provided in Table 1-1.

The standard practice for laboratory asphalt mix design is to simulate the binder absorption and aging that occurs during production and construction by short-term oven aging (STOA) or conditioning the loose mix prior to compaction for a specified time and temperature. For HMA, the recommended procedure in AASHTO R 30 for preparing specimens for volumetric mixture design is 2 hours at the compaction temperature (T_c), while it is 4 hours at 275°F (135°C) for preparing specimens for performance testing. However, the implementation of WMA raised the question of the impact of lower plant temperatures on the aging characteristics and absorption of asphalt by aggregates in asphalt mixtures and how to adequately simulate any differences in the laboratory. Preparation of WMA specimens for quality assurance is also challenging since the question arises about how to account realistically for the compaction of WMA mixtures when reheating may be necessary to make them workable.

Table 1-1. Previous research on short-term aging.

Reference	Short-Term Aging	Major Finding
Heithaus and Johnson 1958	Field Aging	Most aging during production and construction through compaction
Traxler 1961		
Chipperfield and Welch 1967		
Aschenbrener and Far 1994		

Table 1-1. (Continued).

Reference	Short-Term Aging	Major Finding
Traxler 1961	Factor on Field Aging	Binder chemistry and aggregate absorption major effects
Chipperfield and Welch 1967		Aggregate gradation no effect
Terrel and Holen 1976		Plant type significant effect; DMP < BMP due to lower temperature and less moisture
Lund and Wilson 1984 & 1986		Binder type and binder source significant effects
Chollar et al.1989		Slightly more aging from DMP than BMP
Mogawer et al. 2012		• Production temperature, silo storage, inclusion of recycled materials, and reheating significant effects • Softer binder with RAP = harder binder without RAP
Daniel et al. 2014		• Production temperature and silo storage significant effects • Reduced difference in virgin vs. RAP mixtures after reheating
Aschenbrener and Far 1994	Factor on Lab Aging	Aggregate absorption important effect
Topal and Sengoz 2008		• Binder type and binder source significant effects • Reduced aging with polymers
Zhao et al. 2009		
Morian et al. 2011		• Binder type and binder source significant effects • Reduced aging with polymers • Aggregate absorption and gradation important effects
Aschenbrener and Far 1994	2 h at T_c	• Reheating significant effect on Hamburg wheel-tracking test (HWTT) results • Recommend 2 h @ T_c
Estakhri et al. 2010	4 h at 275°F (135°C)	• WMA 4 h @ 275°F (135°C) comparable to HMA 4 h @ 250°F (121°C) • Recommend 4 h @ 275°F (135°C) for WMA
Rashwan and Williams 2011	2 h at 302°F (150°C) (HMA) 2 and 4 h at 230°F (110°C) (WMA)	E* and flow number higher for HMA with different temperature and for mixtures with RAP
Jones et al. 2011	4 h at T_c	• Equivalent HWTT results and heavy-vehicle-simulator rutting for HMA and WMA • More HWTT rutting in WMA without short-term aging
Bonaquist 2011a	2 h at T_c 4 h at T_c	• G_{mm} (aggregate absorption) and indirect tensile strength comparable to cores at construction • Recommend 2 h @ T_c for WMA and suggested additional longer aging period for evaluating rutting and moisture susceptibility
Hajj et al. 2011	4–15 h at 250°F (121°C)	• Recommend compaction of WMA foaming within 4 h • Foaming effects lost @ 4–15 h @ 250°F (121°C)
Clements et al. 2012	0.5, 2, 4, and 8 h at 275°F (135°C) (HMA) 0.5, 2, 4, and 8 h at 237°F (114°C) (WMA)	• Equivalent disc-shaped compact tension results for WMA vs. HMA • Reduced E* and flow number and increased rutting for WMA vs. HMA
Estakhri 2012	2 h at 275°F (135°C) 4 h at 275°F (135°C)	• Equivalent HWTT for WMA vs. HMA • Aging time and temperature effect on HWTT and overlay tester results
Sharp and Malone 2013	1 h at 302°F (150°C)	Recommend 1 h @ 302°F (150°C) for WMA
Epps Martin et al. 2014	2 and 4 h at T_c 2 and 4 h at 275°F (135°C) 2 h at T_c + 16 h at 140°F (60°C) + 2 h at T_c	• Effect on aging: STOA temperature > STOA time • Recommend 2 h @ 275°F (135°C) for HMA and 2 h @ 240°F (116°C) for WMA

Over the last few decades, a number of studies were performed (1) to evaluate the effect of various laboratory STOA protocols for HMA and WMA in order to achieve equivalent binder aging and absorption that occur during production and construction in the field and (2) to evaluate the performance of short-term aged WMA compared to HMA. These studies are also summarized in Table 1-1.

A study by Aschenbrener and Far (1994) was conducted to evaluate the short-term aging of HMA. Nine projects were selected throughout Colorado and four different laboratory STOA protocols including 0, 2, 4, and 8 hours at field compaction temperatures were used to fabricate laboratory specimens. The specimens were tested to determine their theoretical maximum specific gravity (G_{mm}) values and Hamburg wheel-tracking test (HWTT) results in accordance with AASHTO T 209 and AASHTO T 324, and the results were compared against results obtained on counterpart mixtures produced in the field. During construction, the silo storage time and temperature were monitored in order to better quantify the short-term aging that occurred during plant production, and it was indicated that HMA stayed at elevated temperatures in the silo for approximately 1 to 2 hours at an average temperature of 263°F (128°C) prior to being loaded out and transported to the paving site. The comparison in G_{mm} values between laboratory mixtures with various STOA protocols versus field mixtures illustrated that for the majority of the mixtures, the laboratory STOA protocol of 2 to 4 hours at the field compaction temperatures produced asphalt mixtures that matched the asphalt absorption that occurred in the field specimens. The performance of the laboratory specimens in the HWTT was highly dependent upon the STOA time. In addition, laboratory specimens that were short-term aged for 1 to 3 hours at the field compaction temperature exhibited equivalent HWTT performances compared to field specimens. Thus, according to the G_{mm} and HWTT results, researchers recommended conditioning the laboratory-produced samples for 2 hours at the field compaction temperature in order to simulate asphalt aging and absorption during plant production.

A more recent study by Estakhri et al. (2010) evaluated the effect of three laboratory STOA protocols on a WMA mixture prepared with Evotherm DAT™: 2 hours at 220°F (104°C), 2 hours at 275°F (135°C), and 4 hours at 275°F (135°C). WMA performance was evaluated using the HWTT per AASHTO T 324 and compared against that of HMA conditioned for 2 and 4 hours at 250°F (121°C). The target AV content of the mixtures was 7 ± 0.5 percent. In addition, WMA mixtures prepared with Advera® and Sasobit® conditioned for 2 hours at 220°F (104°C) and 4 hours at 275°F (135°C) were also tested and compared against the results of HMA conditioned for 2 hours at 250°F (121°C). The results for WMA Evotherm DAT™ showed that the number of passes to generate a 0.5-in. (12.5-mm) rut depth increased with higher aging temperature and longer aging time. Improved rutting resistance due to increased laboratory STOA temperatures was also observed for the WMA mixtures prepared with the other two WMA technologies. In addition, the WMA Evotherm DAT™ mixture conditioned for 4 hours at 275°F (135°C) showed equivalent performance compared to the control HMA conditioned at 250°F (121°C). Based on these observations, a recommendation for laboratory STOA protocol of 4 hours at 275°F (135°C) for WMA was made and was incorporated in the mix design methods for the Texas Department of Transportation.

The recently completed NCHRP Project 9-43 on mix design practices for WMA (Bonaquist 2011a) recommended a conditioning protocol for WMA of 2 hours at T_c for both volumetric mixture design and performance testing, as stated in the draft appendix to AASHTO R 35. This conditioning protocol was selected based on comparisons of maximum specific gravity (AASHTO T 209) and indirect tensile (IDT) strength (AASHTO T 283) of laboratory-mixed, laboratory-compacted (LMLC) specimens subjected to the conditioning protocol versus the results obtained for plant-mixed, field-compacted (PMFC) cores. The specific gravity comparison showed equivalent maximum theoretical density for LMLC specimens and PMFC cores, indicating the same binder absorption level. The difference in IDT strength between LMLC specimens and PMFC cores was also insignificant based on a paired *t*-test comparison with a 95-percent confidence interval. In addition, further research was recommended to develop a two-step WMA conditioning procedure for the evaluation of moisture susceptibility and rutting resistance, similar to the conditioning protocol applied to HMA. The first step would be conditioning for 2 hours at T_c to simulate binder absorption and aging during construction, and the second step would consist of an extended conditioning time at a representative high in-service temperature but no longer than 16 hours.

NCHRP Project 9-49 (Epps Martin et al. 2014) performed a laboratory conditioning experiment focused on evaluation of the moisture susceptibility of WMA technologies and recommended a laboratory STOA protocol of 2 hours at 275°F (135°C) for HMA and 2 hours at 240°F (116°C) for WMA in order to simulate the short-term aging of the asphalt mixture occurring during production and construction. In the study, different laboratory STOA protocols were selected based on the available literature and used for fabricating HMA and WMA LMLC specimens. These specimens were tested to determine the effect of each laboratory STOA protocol on mixture resilient modulus (M_R) stiffness. Cores at construction were incorporated as part of the experimental design to represent short-term aged HMA and WMA specimens produced in the plant. Laboratory STOA protocols were able to produce asphalt mixtures with significantly increased stiffness. Additionally, the effect from the short-term aging temperature was more pronounced than the short-term aging time. Among the five selected laboratory STOA protocols, 2 hours at 275°F (135°C) and 2 hours at T_c yielded mixture-stiffness values similar to those of HMA and WMA cores at construction, respectively. Considering the difficulty in accurately defining T_c for WMA in the field and the common range of T_c for WMA, 2 hours at 240°F (116°C) instead of 2 hours at T_c was recommended as the standard laboratory conditioning protocol for WMA LMLC specimens. The final recommendation of NCHRP Project 9-49 for HMA LMLC specimens was 2 hours at 275°F (135°C).

In general, the main findings of the previous studies on short-term aging of asphalt mixtures can be summarized as follows:

- Substantial aging in the field occurs during production through placement and compaction.
- Binder type, binder source, aggregate absorption, WMA technology, recycled materials (i.e., RAP and/or RAS), production temperature, and silo storage are factors that have a significant effect on mixture short-term aging characteristics.
- An increase in laboratory STOA temperature, time, or both increases asphalt mixture stiffness. Short-term aging is more sensitive to STOA temperature than STOA time.
- Several laboratory STOA protocols have been proposed to simulate the short-term aging occurring in WMA during production and construction.

Despite the previous research efforts, there are still some aspects of the short-term aging of asphalt mixtures that have not been fully resolved, which include the following:

- A standard laboratory STOA protocol for WMA.
- A comprehensive study to establish short-term aging protocols that encompass the effects of aggregate absorption, asphalt type and source, recycled material inclusion, WMA technology, plant type, and production temperature.

Previous Research on Long-Term Aging

Aging of asphalt pavements continues throughout their in-service life, though at a lower rate compared to that during production and construction. Therefore, it is important to account for the changes in asphalt mixture properties due to field aging when preparing laboratory samples for longer-term performance testing. The standard practice for laboratory mix design of asphalt mixtures is to simulate the field aging by storing the compacted specimens for 5 days at 185°F (85°C) in accordance with AASHTO R 30. In the past few years, studies have evaluated the effect of field and laboratory long-term aging on asphalt mixture properties and identified reasonable correlations between field aging and laboratory long-term oven aging (LTOA) protocols. A brief summary of these studies is provided in Table 1-2.

A study by Rolt (2000) evaluated the effect of field aging on asphalt mixture properties. Thirty-two full-scale test sections were constructed with various factors including exposure temperature, pavement thickness, pavement density, and binder content. Cores were obtained from the pavements at various in-service times. Binders were extracted from these cores and then tested for binder viscosity. The viscosity results of the extracted binders were used to evaluate the effect of each variable on mixture aging characteristics. Based on the test results, it was concluded that the factors of exposure time and exposure temperature had a significant effect on mixture aging characteristics, while the effect from pavement thickness, pavement density, and binder contents was insignificant.

A more recent study by Rondon et al. (2012) was performed to evaluate the evolution of asphalt mixture properties with environmental exposure. A typical HMA was subjected to 42 months of environmental exposure in Bogota, Colombia, prior to being tested in the laboratory to determine the changes in its mechanical properties. Test results indicated that the increase in mixture stiffness, rutting resistance, and fatigue resistance was observed for the first 29 months of environmental exposure and could be attributed to the aging of the asphalt binder due to temperature and ultra-violet radiation. However, an opposite trend was shown between 30 and 42 months.

Another study on laboratory aging by Safaei et al. (2014) evaluated the effect of long-term aging on HMA and WMA stiffness and fatigue resistance. In the study, laboratory HMA and WMA specimens were fabricated following the STOA protocol of 4 h at 275°F (135°C) for HMA and 2 h at 243°F (117°C) for WMA plus LTOA protocol of 2 days or 8 days at 185°F (85°C). Then, at different aging stages, these specimens were tested with dynamic modulus (E*) and direct tension tests to quantify mixture performance–related property evolution with aging. It was indicated that laboratory LTOA was able to produce both HMA and WMA mixtures with a significant increase in mixture stiffness over the short-term aged mixtures. Additionally, a reduction of the difference in mixture stiffness between HMA and WMA was observed after laboratory LTOA.

A study performed at the University of New Hampshire (Tarbox and Daniel 2012) evaluated the effects of laboratory LTOA on RAP mixtures. Plant-produced mixtures with 0, 20, 30, and 40 percent RAP were used to fabricate specimens in the laboratory and were then long-term aged for 2, 4, and 8 days at 185°F (85°C) prior to being tested with the E* test. The Global Aging System model was also used in the study to predict the changes in E* values at various long-term aging levels. These predictions were then compared against the measured values. Test results indicated that the laboratory LTOA was able to produce mixtures with a significant increase in stiffness

Table 1-2. Previous research on long-term aging.

Reference	Long-Term Aging	Major Findings
Kemp and Predoehl 1981	Field Aging	Air temperature, voids, and aggregate porosity have significant effects
Kari 1982		Pavement permeability and asphalt content have significant effects
Rolt 2000		• Exposure time and ambient temperature have significant effects • Binder content, mixture AV, and filler content have no effect
Rondon et al. 2012		• Increased mixture stiffness, rutting resistance, and fatigue resistance for first 29 months of environmental exposure • Opposite trend observed between 30 and 42 months
Farrar et al. 2013		• Field aging not limited to the top 25 mm of the pavement • Field aging gradient observed
West et al. 2014		• WMA has less aging than HMA during production • Reduced difference between WMA vs. HMA with field aging • Equivalent binder true grade and binder absorption for WMA vs. HMA after 2 years of field aging
Morian et al. 2011	Lab Aging (3, 6, and 9 months at 60°C)	• Increased mixture E* and binder carbonyl area (CA) with LTOA • Binder source has significant effect while aggregate source has no effect
Azari and Mohseni 2013	Lab Aging (2 days at 85°C 5 days at 85°C)	• Increased mixture resistance to permanent deformation with LTOA • Interdependence observed between STOA and LTOA
Tarbox and Daniel 2012	Lab Aging (2 days at 85°C 4 days at 85°C 8 days at 85°C)	• Increased stiffness with LTOA • Stiffening effect from LTOA: virgin mixture > RAP mixture • Global Aging System model > LTOA
Safaei et al. 2014	Lab Aging (2 days at 85°C 8 days at 85°C)	• Increased stiffness with LTOA • Reduced difference in stiffness for HMA vs. WMA with LTOA
Bell et al. 1994	Field vs. Lab Aging (4 days at 100°C 8 days at 85°C)	• STOA of 4 h at 135°C = field aging during the construction process • Effect on mixture aging: LTOA temperature > LTOA time • STOA plus LTOA of 4 days at 100°C and 8 days at 85°C = 9 years of field aging in Washington State
Brown and Scholz 2000	Field vs. Lab Aging (4 days at 85°C)	• Stiffness: LTOA of 4 days at 85°C = 15 years of field aging in the United States
Harrigan 2007 Houston et al. 2005	Field vs. Lab Aging (5 days at 80°C 5 days at 85°C 5 days at 90°C)	• Significant field and laboratory aging • AV content effect on field aging • AASHTO R 35 LTOA (5 days @ 85°C) vs. 7–10 years of field aging: lab > field when AV < 8%; lab < field when AV > 8%
Epps Martin et al. 2014	Field vs. Lab Aging (1 to 16 weeks at 60°C)	• Increased stiffness with field aging and laboratory LTOA • Pavement in-service temperature effect on field aging • Stiffness: WMA = HMA, after 6–8 months of field aging • Stiffness: STOA of 2 h at 135°C for HMA and 2 h at 116°C for WMA plus LTOA of 4–8 weeks at 60°C = first summer of field aging

compared to the short-term aged mixtures, and that the stiffening effect was more pronounced for virgin mixtures (i.e., mixtures with 0 percent RAP) than the RAP mixtures. The reduced susceptibility of RAP mixtures to laboratory aging was due to the inclusion of already-aged binders in the RAP. The comparison in predicted versus measured E* values illustrated that the Global Aging System model over-predicted the mixture aging compared to the laboratory LTOA protocols.

Bell et al. (1994) evaluated the correlation between mixture aging in the field versus laboratory aging in terms of STOA and LTOA protocols. In the study, field cores with a wide range of in-service times were acquired and tested to determine their triaxial and diametral M_R stiffness. Laboratory specimens were fabricated using materials from the field sites and conditioned with the selected laboratory STOA and LTOA protocols prior to being tested. M_R stiffness results indicated that a laboratory STOA protocol of 4 h at 275°F (135°C) was representative of the short-term aging occurring during production and construction. In addition, a more significant hardening effect of asphalt mixtures was achieved by LTOA at 185°F (85°C) and LTOA at a higher temperature of 212°F (100°C) for a shorter time. According to the study, LTOA of 4 days at 212°F (100°C) or 8 days at 185°F (85°C) was representative of about 9 years of field aging for the climate conditions in Washington State.

A study by Arizona State University (Harrigan 2007; Houston et al. 2005) evaluated the effects of asphalt mixture aging for performance testing and pavement structural design and identified the potential correlation between field aging and laboratory aging. In the study, field cores were obtained from three test sites after 7 to 10 years in service in Arizona, Minnesota (MnRoad), and Nevada (WesTrack). Plant mix was also obtained from the three sites and was compacted in the laboratory followed by LTOA of 5 days at 176°F (80°C), 185°F (85°C), and 194°F (90°C). Asphalt binders were extracted and recovered from the field cores and laboratory-aged samples and then tested with the dynamic shear rheometer (DSR) for determining viscosity in order to evaluate mixture aging in the field and laboratory. Test results indicated that pavement in-service temperature and pavement AV contents had significant effects on mixture aging in the field; warmer in-service temperatures and higher AV contents in the mixture were generally associated with increased mixture aging. The comparison in viscosity of extracted binders from field cores versus laboratory-aged specimens indicated that the laboratory LTOA protocol outlined by AASHTO R 30 produced more mixture aging than 7 to 10 years of aging in the field when the AV content of the field cores were less than 8 percent and that the reverse was observed when AV content were greater than 8 percent. Therefore, it was recommended that the laboratory aging protocol account for the AV content of the asphalt pavement.

Performance evolution of HMA and WMA with field and laboratory aging was studied as part of NCHRP Project 9-49 (Epps Martin et al. 2014). This study provided preliminary results toward understanding the correlation between intermediate- or long-term field aging and laboratory LTOA protocols, in addition to their effects on mixture properties. In the study, the laboratory STOA protocol of 2 hours at 275°F (135°C) for HMA and 2 hours at 240°F (116°C) for WMA on loose mixtures followed by LTOA protocol of 1 to 16 weeks at 140°F (60°C) on compacted specimens was selected for fabricating laboratory long-term aged HMA and WMA specimens from two field sites in Iowa and Texas based on available literature. Cores at construction and one or two sets of post-construction cores with certain in-service times were included in the experimental design to represent mixtures experiencing intermediate to long-term aging in the field. These long-term aged field and laboratory specimens were tested for M_R stiffness to evaluate the evolution of mixture stiffness with field and laboratory aging, and to explore the potential correlation between field aging and laboratory LTOA protocols. A significant effect on increasing mixture stiffness was observed from intermediate- or long-term aging in the field and laboratory. Additionally, the effect of field aging on mixture stiffness was more pronounced for aging in the summer than aging in the winter, which was likely due to the higher in-service summer temperatures. The difference in mixture stiffness between HMA and WMA was reduced with intermediate- or long-term aging; equivalent mixture stiffness was even achieved after the first summer of field aging or laboratory LTOA of 2 weeks at 140°F (60°C). The correlation between field aging and laboratory LTOA protocols was also explored based on the M_R stiffness. The laboratory STOA protocol of 2 hours at 275°F (135°C) for HMA and 2 hours at 240°F (116°C) for WMA on loose mixtures plus LTOA protocols of 4 to 8 weeks at 140°F (60°C) on compacted specimens were found to be representative of the first summer of field aging.

In NCHRP Project 9-47A, West et al. (2014) established relationships between the engineering properties of WMA binders and mixtures and their field performance. Six existing field sites and eight new field sites were sampled. Each of the field sites included at least an HMA control section and one WMA section. Six of the field sites included multiple WMA technologies, such that a total of 26 WMA and 14 HMA mixtures were sampled. Loose mix was collected, and samples for volumetric analyses and performance tests were compacted on-site, without reheating. Mixture verifications were performed according to the NCHRP 9-43 protocols. When performing the mixture verifications, blends were produced to match the field-produced gradation determined from extraction tests rather than the reported job-mix formula. Samples were prepared for moisture susceptibility, flow number, E*, and in some cases fatigue testing. Results from the testing of recovered binders taken at the time of construction and the testing of laboratory-produced short-term aged specimens showed that WMA mixtures had slightly less aging than HMA, as indicated by lower binder true grade and lower mixture stiffness. After 2 years, recovered binder properties revealed that the true grades of the WMA and HMA binders were not substantially different. Furthermore, there had been little or no aging of binders tested at construction and up to 2 years later. Similarly, it was noted that although the initial absorption of the binder into the aggregate was slightly lower for WMA, after 2 years there was little, if any, difference in absorption.

Ongoing NCHRP Projects 9-49A and 9-54 are directed toward investigating the long-term field performance of WMA technologies and evaluating long-term aging of asphalt mixtures for performance testing and prediction. Although these two projects have already started, their conclusions were not available prior to the completion of NCHRP Project 9-52.

To summarize, the main findings of the previous studies on long-term aging of asphalt mixtures are:

- Field aging is primarily quantified based on pavement in-service time.
- Pavement in-service temperature and time, pavement depth, AV content, and effective binder content in the mixture have significant effects on field long-term aging characteristics of asphalt mixtures.
- Field aging gradient with depth in asphalt pavement has been observed.
- Laboratory LTOA protocols produce asphalt mixtures with a significant increase in mixture stiffness and rutting resistance over unaged mixtures.
- The laboratory long-term aging of asphalt mixtures is more sensitive to LTOA temperature than LTOA time.
- The difference in mixture properties, including stiffness, strength, and rutting resistance, between HMA and WMA is reduced with field aging and laboratory LTOA.
- Reasonable correlations with field aging and laboratory LTOA protocols have been proposed.

Despite the previous research efforts on long-term aging of asphalt mixtures, the following issues still need to be fully addressed:

- How to account for the differences in construction dates and climates of various field sites.
- How to determine when WMA and HMA reach an equivalent stage of aging.
- Which laboratory LTOA protocols are required to achieve equivalent mixture performance–related properties between WMA and HMA.
- The need for a comprehensive study to explore the correlation between field aging and laboratory LTOA protocols that encompass the effects of aggregate absorption, asphalt type and source, recycled material inclusion, WMA technology, plant type, and production temperature.
- The LTOA protocols need to be further calibrated against field aging using the existing NCHRP 9-52 test sites.

CHAPTER 2

Research Approach

This chapter provides an overview of the experimental design for Phases I and II, field site descriptions, specimen fabrication protocols, and laboratory tests.

Experimental Design

Phase I Experiment

The objectives of the Phase I experiment were to (1) develop a laboratory STOA protocol for asphalt loose mix prior to compaction to simulate short-term asphalt aging and absorption of asphalt mixtures during plant production and (2) identify mixture components and production parameters (henceforth named *factors*) with significant effects on the performance-related properties of short-term aged asphalt mixtures. Figure 2-1 presents a schematic of the short-term aging methodology used for this experiment.

Nine field sites in Texas, New Mexico, Connecticut, Wyoming, South Dakota, Iowa, Indiana, and Florida were included in the Phase I experiment, which will be detailed in the following sections. Cores at construction and plant-mixed, plant-compacted (PMPC) specimens were obtained for each mixture at each field site, in conjunction with raw materials including asphalt binder, aggregates, and RAP and RAS used for LMLC specimen fabrication. Based on previous research for NCHRP Project 9-49 (Epps Martin et al. 2014), the laboratory STOA protocols of 2 hours at 275°F (135°C) for HMA and 240°F (116°C) for WMA were used for conditioning loose mix prior to compaction. All these asphalt mixtures subjected to short-term aging during production and compaction in the field or laboratory were tested with the M_R, E*, and HWTT tests. Binders were extracted from short-term aged asphalt mixtures from three field sites (i.e., Indiana, Florida, and Texas II), recovered, and then tested with the DSR and Fourier transform infrared spectroscopy (FT-IR) to characterize their rheological and chemical properties, including continuous performance grade (PG) and complex shear modulus (G*) at 77°F (25°C), and chemical characteristics in terms of the FT-IR carbonyl area (CA). The determination of the continuous PG was helpful in characterizing the change in asphalt binder properties with short-term aging, and the parameter G* at 77°F (25°C) was included in order to provide additional information on binder stiffness measured at the same temperature as the mixture M_R stiffness.

The mixture and binder results for LMLC specimens fabricated using the selected STOA protocols were compared against those for cores at construction and PMPC specimens to validate the laboratory STOA protocol of 2 hours at 275°F (135°C) for HMA and 240°F (116°C) for WMA. Additionally, a second set of comparisons was performed to evaluate the effects of mixture and production factors on the short-term aging of asphalt mixtures for each type of sample, i.e., cores at construction, PMPC specimens, and LMLC specimens.

Phase II Experiment

The objectives of the Phase II experiment were to (1) evaluate the evolution of performance-related properties of asphalt mixtures with intermediate-term aging in the field and laboratory, (2) develop a correlation between intermediate-term field aging and laboratory LTOA protocols, and (3) identify mixture components and production parameters with significant effects on the intermediate- or long-term aging characteristics of the asphalt mixtures. Additionally, the aging of WMA relative to HMA was determined in terms of when the WMA stiffness was equal to HMA or when the stiffness of WMA was equivalent to the initial stiffness of HMA. Figure 2-2 presents the research methodology used for the Phase II experiment.

Seven of the sites from Phase I were included in the Phase II experiment; the Connecticut and Texas II field sites were not used in this experiment because no post-construction cores were obtained due to the traffic concerns of the agency or time constraints of the project. Cores after certain in-service

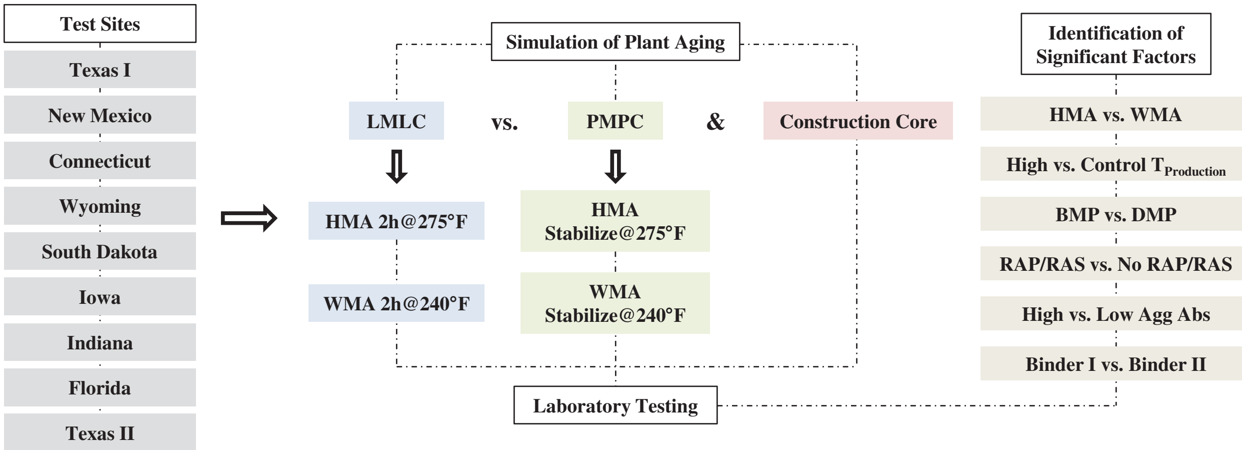


Figure 2-1. Research methodology for the Phase I experiment.

times (ranging from 8 to 22 months) were incorporated in Phase II to quantify aging in the field. To simulate long-term aging in the laboratory, LMLC specimens fabricated with the STOA protocol of 2 hours at 275°F (135°C) for HMA and WMA were further aged after compaction in the environmental room or oven (i.e., laboratory LTOA protocol) prior to testing. Two laboratory LTOA protocols were selected based on previous research: 5 days at 185°F (85°C) in accordance with AASHTO R 30 and 2 weeks at 140°F (60°C).

Consistent with the Phase I experiment, laboratory mixture tests included M_R, HWTT, and E*. Binders were extracted from Indiana and Florida long-term aged mixtures (i.e., cores and LMLC specimens with STOA plus LTOA protocols), recovered, and tested with DSR and FT-IR to characterize the change in the rheological and chemical properties.

Previous literature indicated that the field aging of asphalt mixtures has been commonly quantified by the in-service time of the pavement at the time of coring. However, a potential issue with using in-service time arose from the differences in construction dates and climates for the various field sites. To address that issue, the concept of cumulative degree-day (CDD; 32°F [0°C] base) was proposed in this project, as expressed in Equation (2-1).

$$\mathrm{CDD} = \sum (T_{\mathrm{dmax}} - 32) \qquad \text{Eq. (2-1)}$$

Where:

T_{dmax} = daily maximum temperature, °F.

The CDD values for two field sites that were part of NCHRP Project 9-49 (Epps Martin et al. 2014) indicated that an 8-month field aging period in Texas (including the summer) was equivalent to approximately a 12-month aging period in Iowa in terms of increasing mixture stiffness. Therefore, compared to the field in-service time, the CDD value provided a more discriminating measure of field aging when comparing

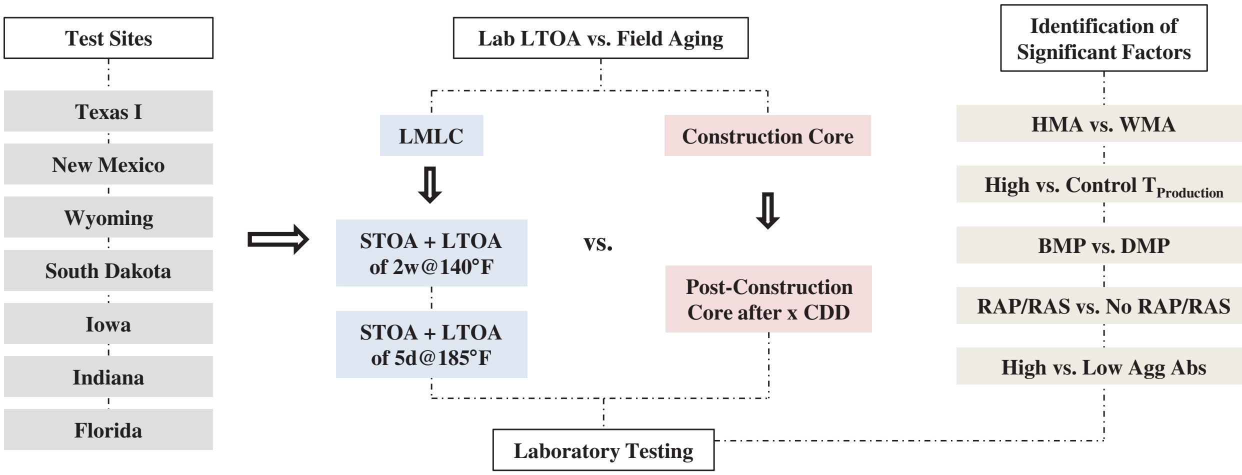


Figure 2-2. Research methodology for the Phase II experiment.

field sites built in different climates and at various times of the year. Details of the CDD values for field sites included in this project are presented and discussed in the next chapter.

To better quantify the evolution of mixture stiffness and rutting resistance, and binder stiffness and oxidation with field and laboratory aging, four binder or mixture property ratios—M_R ratio, HWTT rutting resistance parameter (RRP) ratio, DSR G* ratio, and FT-IR CA ratio—were proposed. As expressed in Equation (2-2), the M_R ratio is defined as the fraction of the M_R stiffness of either field or laboratory long-term aged (LTA) specimens over that of short-term aged (STA) specimens from either the laboratory or field.

$$M_R \text{ Ratio} = M_{R\text{-}LTA}/M_{R\text{-}STA} \qquad \text{Eq. (2-2)}$$

Where:

$M_{R\text{-}LTA}$ = M_R stiffness of long-term aged specimens including cores after certain in-service times or LMLC specimens with STOA plus LTOA protocols; and

$M_{R\text{-}STA}$ = M_R stiffness of short-term aged specimens including construction cores or LMLC specimens with STOA protocols.

Because field and laboratory aging produced asphalt mixtures with increased M_R stiffness, the M_R ratio was expected to be greater than 1.0. However, the HWTT RRP ratio exhibited the opposite trend with aging due to the mixture stiffening effect (i.e., less rutting with aging). Therefore, the HWTT RRP ratio was defined as the ratio of the RRP of STA specimens over the RRP of LTA specimens, as expressed in Equation (2-3), in order to also get a ratio greater than 1.0 with aging.

$$\text{HWTT RRP Ratio} = RRP_{STA}/RRP_{LTA} \qquad \text{Eq. (2-3)}$$

Where:

RRP_{STA} = HWTT RRP of short-term aged specimens including construction cores or LMLC specimens with STOA protocols; and

RRP_{LTA} = HWTT RRP of long-term aged specimens including cores after certain in-service times or LMLC specimens with STOA plus LTOA protocols.

Similar to the M_R ratio, the G* ratio and the FT-IR CA ratio are defined as the fraction of the DSR G* at 77°F (25°C) and FT-IR CA value of extracted and recovered binders from either laboratory or field LTA specimens over that of STA specimens, as expressed in Equations (2-4) and (2-5), respectively. In addition, both the G* ratio and the FT-IR CA ratio were expected to be greater than 1.0 since field and laboratory aging were likely to produce asphalt binders with higher stiffness and oxidation due to the formation of oxygen-containing C=O bonds.

$$\text{DSR } G^* \text{ Ratio} = G^*_{-LTA}/G^*_{-STA} \qquad \text{Eq. (2-4)}$$

Where:

G^*_{-LTA} = G* of extracted and recovered binders from long-term aged specimens including cores or LMLC specimens; and

G^*_{-STA} = G* of extracted and recovered binders from short-term aged specimens including construction cores or LMLC specimens.

$$\text{FT-IR CA Ratio} = CA_{-LTA}/CA_{-STA} \qquad \text{Eq. (2-5)}$$

Where:

CA_{-LTA} = CA of extracted and recovered binders from long-term aged specimens including cores after certain in-service times or LMLC specimens; and

CA_{-STA} = CA of extracted and recovered binders from short-term aged specimens including construction cores or LMLC specimens.

M_R ratio, HWTT RRP ratio, DSR G* ratio, and FT-IR CA ratio values greater than 1.0 indicate an increase in mixture stiffness and rutting resistance, as well as binder stiffness and oxidation, after long-term aging compared to the short-term aged counterparts. To discriminate asphalt binders or mixtures with different aging characteristics, mixtures with higher property ratios are considered more sensitive to aging and more likely to exhibit an increase in mixture stiffness and rutting resistance as well as binder stiffness and oxidation after a certain level of aging.

To further characterize the evolution of binder or mixture properties with field aging, the exponential function shown in Equation (2-6) was used to correlate the measured ratio values of post-construction cores with their corresponding CDD values. As will be explained in Chapter 3, Equation (2-6) provides a good fit of data for field-aged material properties plotted against CDD.

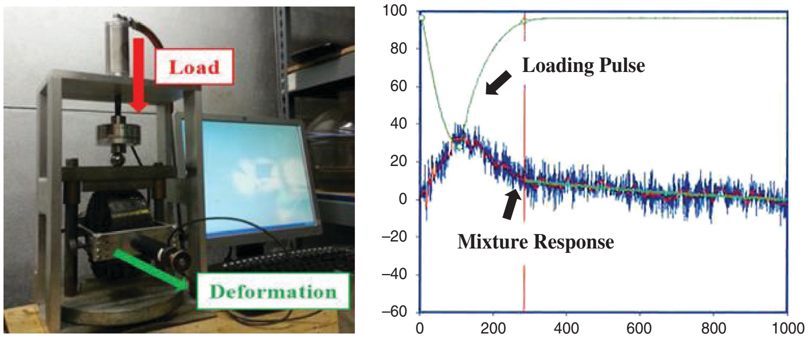
$$\text{Binder or Mixture Property Ratio} = 1 + \alpha * \exp\left[-\left(\frac{\beta}{CDD}\right)^{\gamma}\right] \qquad \text{Eq. (2-6)}$$

Where:

CDD = cumulative degree-days for cores after certain in-service times; and

α, β, and γ = fitting coefficients.

Findings from the Phase I experiment indicated that the laboratory STOA protocol of 2 hours at 275°F (135°C) for HMA and 240°F (116°C) for WMA was representative of cores at construction in terms of mixture stiffness and rutting resistance. Therefore, based on the definitions for binder or



(a) sample setup in loading frame (b) data acquisition system

Figure 2-3. M_R test equipment.

mixture property ratios, the correlation between field aging and laboratory LTOA protocols could be made.

To sum up, the mixture and binder results obtained in the Phase II experiment for cores versus LMLC specimens with STOA plus LTOA protocols were used to evaluate the effect of field and laboratory aging on mixture properties and to develop a correlation between field aging and laboratory LTOA protocols. Comparisons were also performed to evaluate the effect of mixture and production factors on the long-term aging characteristics of asphalt mixtures.

Laboratory Tests

Based on previous experience with laboratory testing, M_R, E*, and HWTT tests were selected in this project to compare the stiffness and rutting resistance of asphalt mixtures with various factors of mixture components, production parameters, and aging stages. DSR, bending beam rheometer (BBR), and FT-IR tests were included in order to evaluate the change in the rheological and chemical properties of asphalt binders with aging. Details of each laboratory test are given in the following subsections.

Resilient Modulus

The M_R test was conducted through repetitive applications of compressive loads in a haversine waveform along a vertical diametral plane of cylindrical asphalt concrete specimens. The resulting horizontal deformations of the specimen were measured by two linear variable differential transducers (LVDTs) aligned along the horizontal diametral plane. An environmentally controlled room at 77°F (25°C) was used for temperature conditioning and testing. The test equipment used to perform the measurements is shown in Figure 2-3. M_R stiffness was measured in accordance with the current ASTM D-7369 with a modification consisting of replacing the on-specimen LVDTs with external LVDTs aligned along the horizontal diametral plane (i.e., gauge length as a fraction of diameter of the specimen = 1.00). As expressed in Equation (2-7), the M_R stiffness was calculated based on vertical load, horizontal deformation, and the asphalt mixture's Poisson ratio.

$$M_R = \frac{P(\upsilon + 0.2732)}{t\Delta} \qquad \text{Eq. (2-7)}$$

Where:

M_R = resilient modulus of asphalt mixture;
P = vertical load;
υ = Poisson's ratio;
t = specimen thickness; and
Δ = horizontal deformation measured by LVDTs.

Dynamic Modulus

The E* test was conducted under unconfined conditions using the Asphalt Mixture Performance Tester, shown in Figure 2-4, following the test procedure specified in AASHTO TP 79-13. Superpave gyratory compactor specimens were

Figure 2-4. Asphalt Mixture Performance Tester.

compacted to a height of 6.7 in. (170 mm) and then cored and trimmed to obtain test specimens with a diameter of 4.0 in. (100 mm) and a height of 6.0 in. (150 mm).

Testing was conducted at 39.2°F (4°C), 68°F (20°C), and 104°F (40°C) and three frequencies of 0.1, 1, and 10 Hz for each temperature. Load levels were determined by a trial and error process to ensure that the amplitude of measured vertical strains was in the range of 50 to 75 microstrains and to prevent damage to the test specimen.

The E* master curve was constructed by fitting the E* values at each temperature/frequency condition to the sigmoidal function described in Equation (2-8), followed by horizontally shifting according to the time–temperature shift factor function expressed in Equation (2-9). To further discriminate E* stiffness of asphalt mixtures due to different binder or mixture aging levels, the E* stiffness at 68°F (20°C) and 10 Hz was used as another indicator for asphalt mixture stiffness in addition to the E* master curve.

$$\log|E^*| = a + \frac{b}{1 + \frac{1}{e^{c+g*\log(f_R)}}} \qquad \text{Eq. (2-8)}$$

Where:

E^* = dynamic modulus of asphalt mixture;
f_R = reduced frequency; and
a, b, c, and g = fitting coefficients of the sigmoidal function.

$$\log a_T = \alpha_1 T^2 + \alpha_2 T^2 + \alpha_3 \qquad \text{Eq. (2-9)}$$

Where:

a_T = time–temperature shift factor; and
α_1, α_2, and α_3 = fitting coefficients of time–temperature shift factor function.

Hamburg Wheel-Tracking Test

The HWTT (AASHTO T 324) is a laboratory test commonly used for evaluating rutting resistance and moisture susceptibility of asphalt mixtures. The test consists of submerging specimens in water at 122°F (50°C) for one hour prior to subjecting them to 52 passes per minute of a loaded steel wheel. Two replicate specimens were loaded for a maximum of 20,000 load cycles or until the deformation at the center of the specimen reached 12.5 mm, per Texas Department of Transportation standard specification Tex-242-F, at which point the test was stopped. The HWTT equipment used to perform the measurements is shown in Figure 2-5.

Typical results of the rut depth versus number of load cycles consist of three phases: (1) post-compaction, (2) creep, and (3) stripping (Solaimanian et al. 2003). The traditional test parameter for evaluating the rutting performance of asphalt mixtures is the rut depth at a specific number of load cycles (i.e., 5,000, 10,000, 15,000, and 20,000). In this study, the rut depth at 5,000 load cycles was selected as a rutting resistance parameter in the HWTT since a significant amount of the short-term aged mixtures (especially those using soft binders, i.e., low high-temperature PG binders) failed to achieve higher levels of loading.

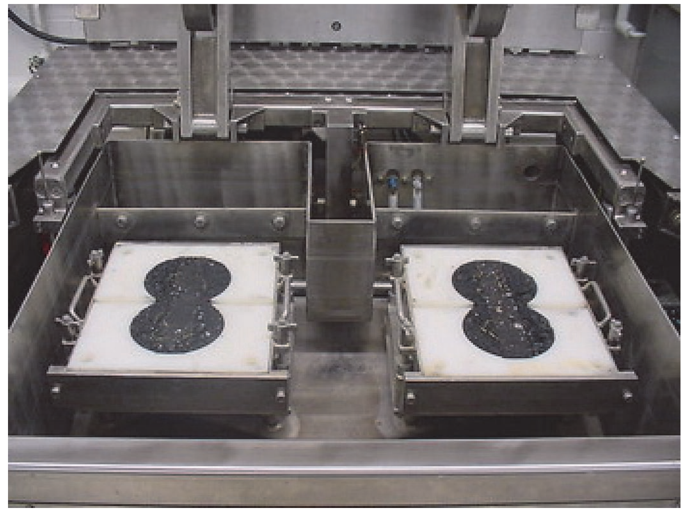


Figure 2-5. HWTT equipment.

In addition to the traditional HWTT rutting analysis, a novel method developed by Yin et al. (2014) was included in the project to discriminate between asphalt mixtures with different rutting resistance. The Yin et al. RRP value (i.e., viscoplastic strain at the stripping number) was employed as an alternative to the HWTT rutting resistance parameter and accuracy was significantly improved by isolating the viscoplastic strain during the creep phase and excluding any contributions from the post-compaction phase due to different specimen AV content or to stripping. The determination of alternative RRP is schematically illustrated in Figure 2-6, and the explanation of the analysis method follows.

The methodology to analyze the HWTT results included curve fitting the entire output of rut depth versus load cycles and defining a test parameter to evaluate mixture rutting resistance, as expressed in Equation (2-10):

$$\mathrm{RD} = \rho * \left[\ln\left(\frac{\mathrm{LC}_{\mathrm{ult}}}{\mathrm{LC}}\right)\right]^{-\frac{1}{\beta}} \qquad \text{Eq. (2-10)}$$

Where:

LC = the number of load cycles at a certain rut depth;
$\mathrm{LC}_{\mathrm{ult}}$ = the maximum number of load cycles;
RD = rut depth at a certain number of load cycles (mm); and
ρ and β = curve fitting coefficients.

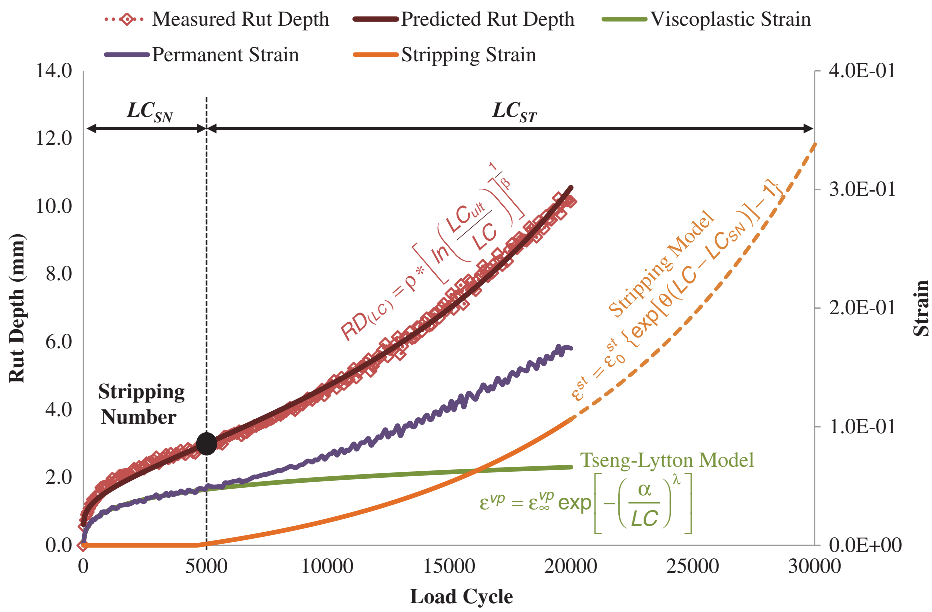


Figure 2-6. Determination of alternative HWTT rutting resistance parameter.

As illustrated in Figure 2-6, the fitted HWTT rut depth curve was composed of one part with negative curvature used to evaluate mixture resistance to rutting due to deformation, followed by a second part with positive curvature used to evaluate mixture resistance to stripping. The critical point where the curvature changed from negative to positive was defined as the stripping number (SN), and the load cycle where SN occurred was labeled as LC_{SN}, which can be determined following Equation (2-11):

$$LC_{SN} = LC_{ult} * e^{-\left(\frac{\beta+1}{\beta}\right)} \qquad \text{Eq. (2-11)}$$

The rut depth accumulated before the SN is related to the viscoplastic deformation of the mixture, and this viscoplastic strain could be calculated as the ratio of the rut depth to the specimen thickness at any given number of load cycles up to LC_{SN}. A typical viscoplastic strain versus load cycle curve is also shown in Figure 2-6. Then, the Tseng–Lytton model (Tseng and Lytton, 1989) was employed to fit the viscoplastic strain data using Equation (2-12):

$$\varepsilon^{vp} = \varepsilon_{\infty}^{vp} \exp\left[-\left(\frac{\alpha}{LC}\right)^{\lambda}\right] \qquad \text{Eq. (2-12)}$$

Where:

$\varepsilon_{\infty}^{vp}$ = saturated viscoplastic strain (treated as a fitting coefficient); and

α and λ = model coefficients.

To better quantify mixture resistance to rutting in the HWTT and compare different mixtures, the RRP (slope of the viscoplastic strain versus load cycle curve at SN) was proposed as an alternative to the traditional parameter (i.e., rut depth at a specific number of load cycles [5,000, 10,000, 15,000, or 20,000]), as expressed in Equation (2-13):

$$\text{RRP}\left(\text{i.e., } \Delta\varepsilon_{LC_{SN}}^{vp}\right) = \alpha^{\lambda}\lambda\varepsilon_{\infty}^{vp} \exp\left[-\left(\frac{\alpha}{LC_{SN}}\right)^{\lambda}\right](LC_{SN})^{-(\lambda+1)} \qquad \text{Eq. (2-13)}$$

Compared to the traditional parameter, the determination of the RRP isolates the viscoplastic strain during the creep phase and does not include contributions due to specimen AV content or stripping. The RRP should be able to better characterize mixture rutting resistance in the HWTT. Specifically, asphalt mixtures with higher RRP values are expected to be more susceptible to rutting than those with lower RRP values.

According to laboratory experience with the analysis method, early stripping had been frequently observed for short-term aged asphalt mixtures using softer asphalt binders and/or recycled materials, with LC_{SN} observed at less than 3,000 load cycles. These mixtures had a limited duration of the creep phase before stripping occurred and, as a consequence, the determination of the viscoplastic strain was not feasible. Therefore, in this project, the evaluation of rutting resistance of asphalt mixture by the RRP was only performed for asphalt mixtures having LC_{SN} greater than 3,000 load cycles.

Dynamic Shear Rheometer

The DSR is commonly used to characterize the viscous and elastic behavior of asphalt binders at medium to high temperatures. As shown in Figure 2-7, the equipment used in this project was a Malvern Bohlin DSR2. During the test, a 0.04 in. (1 mm) or 0.08 in. (2 mm) thick sample of asphalt binders was placed between two parallel circular plates (0.32 in. [8 mm] or 1.0 in. [25 mm] in diameter). The bottom plate was fixed, while the top plate oscillated back and forth across the sample at a given frequency to create shear in the sample. The angular rotation and the applied torque were measured during the test and were then used to calculate G* and phase angle (δ). G* is the asphalt binder's total resistance to deformation in repeated shear, and δ is the time delay between the applied shear stress and the resulting shear strain.

According to the Superpave PG asphalt binder specifications, DSR tests are conducted on unaged, short-term aged, and long-term aged binders. In this project, in order to characterize the unaged and short-term aged asphalt binders, the test was performed at high pavement in-service temperatures, ranging from 137°F (58°C) to 169°F (76°C), in accordance with AASHTO T 315, and the parameter of G*/sin(δ) was used to evaluate mixture resistance to rutting. The high-temperature PG of the asphalt binder was determined based on the G*/sin(δ) values at different temperatures per ASTM D7643.

Figure 2-7. DSR test equipment.

Besides the traditional DSR testing, the DSR frequency sweep test was incorporated in the experimental design as an alternative to the BBR test in order to determine the asphalt binder's low-temperature properties. Replacing the BBR test with the DSR frequency sweep test was advantageous due to the significantly reduced amount of material required and shorter preparation time for the extracted binder. In this project, the DSR frequency sweep test was conducted at 43°F (6°C), which was the lowest achievable test temperature by the available DSR equipment. To predict the BBR test results (i.e., stiffness and *m*-value), Equation (2-14)—provided in *Strategic Highway Research Program (SHRP) Report A-369* (Anderson et al. 1994)—was used to determine the frequency for the DSR testing at the intermediate temperature.

$$T_d = \left[\frac{1}{273+T_s} - \frac{2.303 * R * \log(t_s * f)}{250{,}000}\right]^{-1} - 273 \qquad \text{Eq. (2-14)}$$

Where:

T_d = test temperature for DSR testing at frequency f, °C;
T_s = specified temperature for BBR test, °C;
R = ideal gas constant, 8.31 J/°K-mol;
t_s = specified creep loading time, 60 s; and
f = DSR testing frequency, rad/s.

Considering the DSR frequency sweep test temperature of 43°F (6°C) and the specified creep loading time of 60 seconds in the BBR test, the relationship between BBR test temperature and DSR testing frequency was established and is summarized in Table 2-1. Due to the frequency limitations (0.001 to 100 Hz) of the available DSR equipment at the Texas A&M Transportation Institute, the BBR prediction using the DSR frequency sweep test could not be applied to binders with a low-temperature PG below −28°C.

Figure 2-8 presents a typical DSR frequency sweep test result in terms of G* and δ versus testing frequency. Given the desired temperature for the BBR tests, the corresponding frequency used in the DSR frequency sweep test (f_c) could be found using Table 2-1. Then, the G* and δ at f_c were obtained based on Figure 2-8, referred to as G*(f_c) and δ(f_c),

Table 2-1. Selected frequency for the DSR frequency sweep test.

PG	BBR Test Temperature (°C)	Selected DSR Frequency (Hz)
XX-16	−6	0.34
XX-22	−12	4.49
XX-28	−18	67.66
XX-34	*−24*	*1160.72*

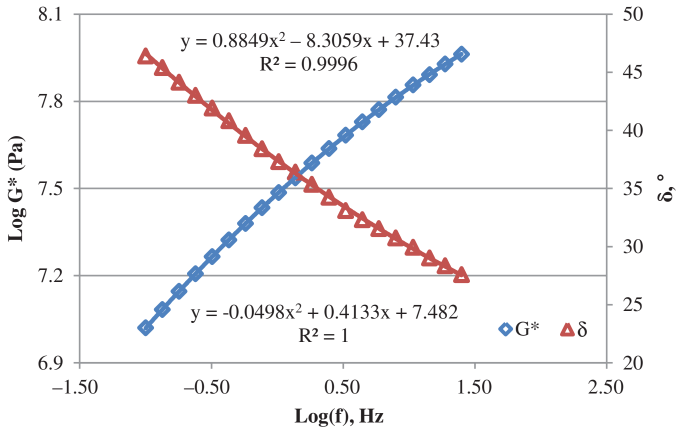


Figure 2-8. DSR frequency sweep test result example.

respectively. Then, the creep stiffness for the BBR test was calculated using Equation (2-15).

$$S_t = \frac{3G^*_{(f_c)}}{\left[1+0.2\sin\left(2\delta_{(f_c)}\right)\right]} \qquad \text{Eq. (2-15)}$$

Additionally, the BBR m-value was calculated by taking the first derivative of the G* versus frequency function (shown in Figure 2-8) at f_c, as expressed in Equation (2-16):

$$m = \frac{\partial[\log(G^*)]}{\partial[\log(f)]_{f=f_c}} \qquad \text{Eq. (2-16)}$$

Bending Beam Rheometer

The BBR test is commonly used to characterize the low-temperature relaxation properties of the original asphalt binders after long-term aging. In this project, the BBR test was performed in accordance with AASHTO T 313, and the test equipment is shown in Figure 2-9. During the test, an asphalt binder beam of 4.9 in. (125 mm) long, 0.5 in. (12.5 mm) wide, and 0.25 in. (6.25 mm) thick was supported in a cooling bath. After the temperature of the beam achieved the designated testing temperature, a vertical load was applied to the center of the beam for 240 seconds. During loading, the deflection at the center of the beam was measured against time. The low-temperature relaxation properties of the asphalt binders in terms of relaxation stiffness and m-value were calculated based on the dimensions, applied loads, and measured deflections of the beam.

According to the Superpave PG asphalt binder specifications, BBR tests should be conducted on long-term aged asphalt binders. In this project, the BBR test was conducted on the original binder after rolling thin-film oven (RTFO) plus pressure aging vessel (PAV) aging in order to characterize their low-temperature rheological properties. Additionally, the stiffness and m-value results measured at different test temperatures were used to determine the continuous low-temperature PG of the asphalt binder in accordance with AASHTO PP 42.

Figure 2-9. BBR test equipment.

Fourier Transform Infrared Spectroscopy

The FT-IR analysis has been proven to be an effective tool to determine the compositional changes occurring in asphalt binders with aging. During the process of oxidation, changes occur in the chemical bonds and molecular structure of the asphalt binder; polar oxygen-containing functional compounds, which contain infrared active carbonyl C=O bonds, are formed (Michalica et al. 2008; Jia et al. 2014). Therefore, the amount of asphalt aging can be quantified by measuring the change in the amount of carbonyl C=O bonds.

In addition to DSR and BBR tests, the infrared spectra analysis was also performed on short-term aged and long-term aged asphalt binders in order to characterize their chemical properties. Figure 2-10 presents the Thermo Scientific

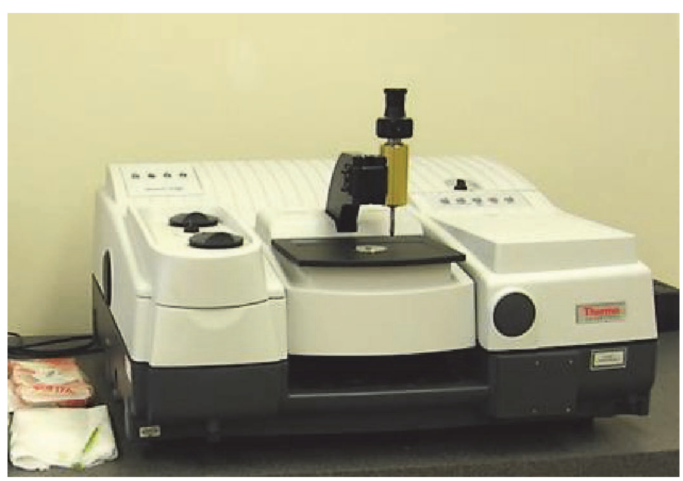


Figure 2-10. FT-IR test equipment.

Nicolet 6700 FT-IR spectrometer used in the project. During the test, a Teflon-coated spatula was used to apply the molten asphalt sample (approximately 0.5 g) to the reflection surface of the prism, which was preheated in an oven at 302°F (150°C). The CA was measured using the attenuated total reflectance method (effective path length was about 4 μm). The CA was defined as the integrated peak area from 1820 to 1650 cm^{-1}, measured in arbitrary units, as a surrogate of asphalt oxidation level (Jemison et al. 1992). Asphalt binders with higher CA values were expected to have experienced a greater level of aging compared to those with lower CA values.

Field Sites

Table 2-2 provides a summary of the field sites used in this project. A detailed construction report for each field site is provided in Appendix A. Based on a literature search on asphalt

Table 2-2. Summary of field sites.

Location & Climate	Date	Plant Type	$T_{Production}$	Asphalt	Aggregate & Additives	Mixtures	Factors
Texas I FM 973 Wet–No Freeze	1/12	CFD	325°F (163°C) HMA 325°F (163°C) HMA+RAP/RAS 275°F (135°C) Foaming 275°F (135°C) Evotherm 270°F (132°C) Evotherm+RAP/RAS	5.2% AC PG 70-22 (Styrene–Butadiene–Styrene [SBS]) PG 64-22 w/ 15%RAP / 3%RAS	Limestone	HMA HMA + 15%RAP + 3%RAS WMA Foaming WMA Evotherm DAT WMA Evotherm + 15%RAP + 3%RAS	HMA vs. WMA RAP/RAS vs. No RAP
New Mexico IH 25 Dry–No Freeze	10/12	CFD	345°F (174°C) HMA 315°F (157°C) HMA + RAP 285°F (141°C) Foaming 275°F (135°C) Evotherm	5.4% AC PG 76-28 (SBS) PG 64-28 w/ 35%RAP	Siliceous Gravel 1% Versabind	HMA HMA + 35%RAP WMA Foaming + 35%RAP WMA Evotherm 3G + 35%RAP	HMA vs. WMA RAP vs. No RAP
Connecticut IH 84 Wet–Freeze	8/12	CFD	322°F (161°C) HMA 312°F (156°C) Foaming	5.0% AC PG 76-22 (SBS) w/ 20%RAP	Basalt	HMA + 20%RAP WMA Foaming + 20%RAP	HMA vs. WMA
Wyoming SR 196 Dry–Freeze	8/12	CFD	315°F (157°C) HMA 255°F & 275°F (124°C & 135°C) Evotherm 275°F & 295°F (135°C & 146°C) Foaming	5.0% PG 64-28 (polymer)	Limestone 1% Lime	HMA WMA Foaming WMA Evotherm 3G	HMA vs. WMA Production Temperature (WMA)
South Dakota SH 262 Dry–Freeze	10/12	CFD	310°F (154°C) HMA 275°F (135°C) Foaming 270°F (132°C) Evotherm 280°F (138°C) Advera	5.3% PG 58-34 (SBS) w/ 20%RAP	Quartzite 1% Lime	HMA + 20%RAP WMA Foaming + 20%RAP WMA Evotherm 3G + 20%RAP WMA Advera + 20%RAP	HMA vs. WMA
Iowa Fairgrounds Wet–Freeze	6/13	CFD	295°F & 325°F (146°C & 163°C) Low Abs HMA 295°F & 310°F (146°C & 154°C) High Abs HMA 265°F & 295°F (129°C & 146°C) Low Abs Foaming 260°F & 290°F (127°C & 143°C) High Abs Foaming	5.0% (0.9% AC Limestone) and 7.0% (3.2% AC Limestone) PG 58-28 w/ 20%RAP	0.9% AC Limestone + Field Sand 3.2% AC Limestone + Field Sand	Low Abs HMA + 20%RAP High Abs HMA + 20%RAP Low Abs WMA Foaming + 20%RAP High Abs WMA Foaming + 20%RAP	HMA vs. WMA Production Temperature Aggregate Absorption
Florida Parking Wet–No Freeze	8/13	CFD	306°F (152°C) Low Abs HMA 308°F (153°C) High Abs HMA 272°F (133°C) Low Abs Foaming 267°F (131°C) High Abs Foaming	5.1% (0.6% AC Granite) and 6.8% (3.7% AC Limestone) PG 58-28 w/ 25%RAP	0.6% AC Granite 3.7% AC Limestone 0.5% Liquid Antistrip	Low Abs HMA + 25%RAP High Abs HMA + 25%RAP Low Abs WMA Foaming + 25%RAP High Abs WMA Foaming + 25%RAP	HMA vs. WMA Aggregate Absorption

(continued on next page)

Table 2-2. (Continued).

Location & Climate	Date	Plant Type	$T_{Production}$	Asphalt	Aggregate & Additives	Mixtures	Factors
Indiana Residential Wet–Freeze	8/13	CFD BMP	300°F (149°C) HMA + RAP (CFD) 305°F (152°C) HMA + RAP (BMP) 271°F (133°C) Foaming + RAP (CFD) 273°F (134°C) Advera + RAP (BMP)	5.8% PG 64-22	Limestone	HMA + 25%RAP (CFD) HMA + 25%RAP (BMP) WMA Foaming + 25%RAP (CFD) WMA Advera + 25%RAP (BMP)	HMA vs. WMA Plant Type
Texas II Local Dry–No Freeze	4/14	CFD BMP	310°F (154°C) HMA (Binder V, CFD) 315°F (157°C) HMA (Binder V, BMP) 310°F (154°C) HMA (Binder A, CFD) 315°F (157°C) HMA (Binder A, BMP)	6.2% PG 64-22	Limestone	HMA (Binder V, CFD) @ 5.9% AC HMA (Binder V, BMP) @ 6.4% AC HMA (Binder A, CFD) @ 5.9% AC HMA (Binder A, BMP) @ 6.4% AC	Plant Type Asphalt Source

plants and a preliminary laboratory experiment (Appendixes B and C, respectively), the following factors—besides climate (wet–freeze, dry–freeze, wet–no freeze, and dry–no freeze)—were considered when selecting field sites that included a wide spectrum of materials and production parameters:

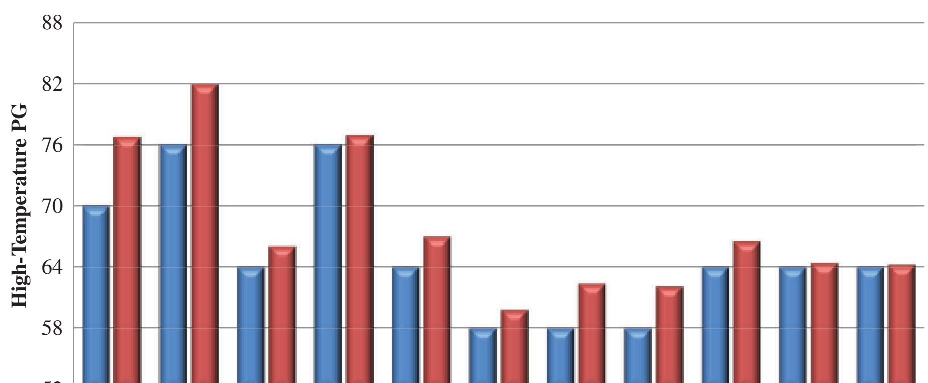
- Aggregate type (high water absorption capacity [AC] and low AC)
- Asphalt source (fast aging and slow aging)
- Recycled materials (RAP and RAS)
- WMA technology (WMA and HMA)
- Plant type (batch plant and drum plant)
- Production temperature (high and low)

Certain factors such as asphalt grade and nominal maximum aggregate size (NMAS) had to be treated as co-variables. All field sites included at least one of the factors listed above.

Figures 2-11, 2-12, and 2-13 summarize the continuous high-temperature performance grades, low-temperature performance grades, and intermediate-temperature performance grades for 11 different binders used in nine field sites, respectively. For each binder type presented in Figures 2-11 and 2-12, two bars describe the results; the bar on the left of each pair represents the specified PG provided by the material supplier, while the bar on the right represents the PG measured in the laboratory.

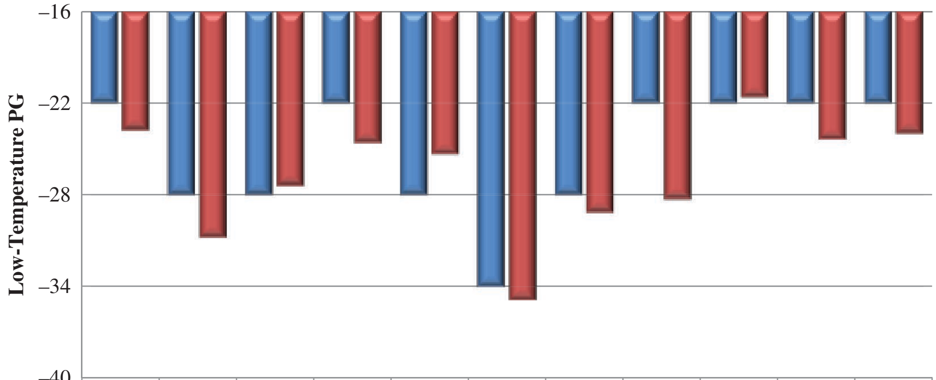
As illustrated in Figure 2-11, for 9 out of 11 binders included in the study, the high-temperature PG results obtained from the DSR testing on unaged and/or RTFO aged binders matched the specified grades. The only two exceptions were Texas I and New Mexico 76 (PG 76-28) binders, which showed a higher measured PG.

A similar trend in the low-temperature PG results from laboratory testing versus specified values is shown in Figure 2-12.



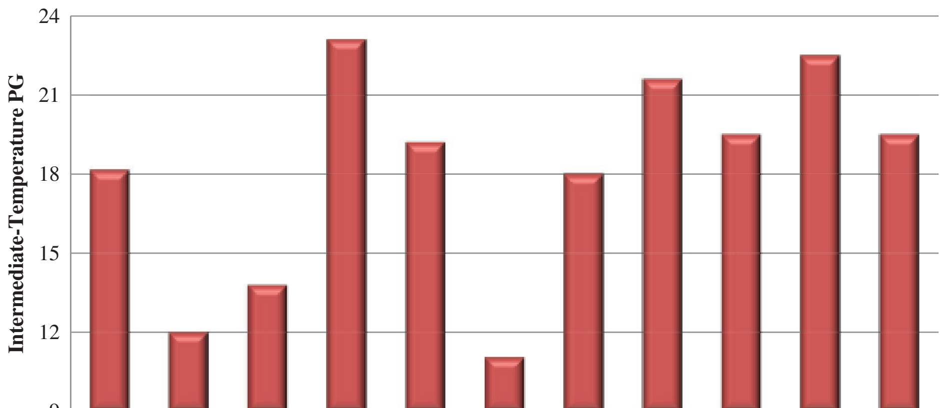
	TXI	NM 76	NM 64	CT	WY	SD	IA	FL	IN	TXII-A	TXII-V
Specified	70.0	76.0	64.0	76.0	64.0	58.0	58.0	58.0	64.0	64.0	64.0
Graded	76.8	82.0	66.0	76.9	67.0	59.8	62.4	62.1	66.5	64.4	64.2

Figure 2-11. Continuous high-temperature PG results.



	TXI	NM 76	NM 64	CT	WY	SD	IA	FL	IN	TXII-A	TXII-V
Specified	–22.0	–28.0	–28.0	–22.0	–28.0	–34.0	–28.0	–22.0	–22.0	–22.0	–22.0
Graded	–23.8	–30.8	–27.4	–24.6	–25.4	–34.9	–29.2	–28.3	–21.6	–24.4	–24.0

Figure 2-12. Continuous low-temperature PG results.



	TXI	NM 76	NM 64	CT	WY	SD	IA	FL	IN	TXII-A	TXII-V
Graded	18.2	12.0	13.8	23.1	19.2	11.1	18.0	21.6	19.5	22.5	19.5

Figure 2-13. Continuous intermediate-temperature PG results.

For most of the binders, the low-temperature PG results obtained from the BBR tests on RTFO plus PAV aged binders matched the grades specified by the supplier. However, a slightly higher PG was exhibited by the laboratory test results for the New Mexico 64 (PG 64-28) binder (–27.4 instead of –28.0) and Wyoming binder (–25.4 instead of –28.0), while the opposite trend was shown for the Florida binder (–28.3 instead of –22.0).

The continuous PG results for the 11 binders used in the nine field sites are summarized in Table 2-3.

Specimen Fabrication

To fabricate LMLC specimens, aggregates and binders were heated to the specified plant mixing temperature (T_m) and then mixed with a portable bucket mixer. Afterwards, the loose mix was conditioned in the oven following the laboratory STOA protocol of 2 hours at 275°F (135°C) for HMA and 240°F (116°C) for WMA prior to compaction with the Superpave gyratory compactor. Trial specimens were fabricated to ensure specimens were obtained with AV contents of 7 ± 0.5 percent. To simulate the intermediate-term mixture aging in the field for the Phase II experiment, LMLC specimens were conditioned following a STOA protocol of 2 hours at 275°F (135°C) for both HMA and WMA and then further aged in accordance with the selected laboratory LTOA protocols in an environmental room or oven prior to being tested. In total, 522 LMLC specimens with 7 ± 0.5 percent AV content were fabricated for the nine field sites that included 40 mixtures.

For PMPC specimens, loose mix was taken from the trucks before leaving the plant and maintained in the oven for 1 to 2 hours at the field compaction temperature prior to compaction (Epps Martin et al. 2014). As mentioned previously, cores were obtained at construction for all nine field sites. Additionally, one or more sets of post-construction cores were acquired

Table 2-3. Continuous PG summary.

Field Site	Specified PG	Continuous PG*
Texas I	70-22	76.8–23.8 (18.2)
New Mexico	76-22	82.0–30.8 (12.0)
New Mexico	64-28	66.0–27.4 (13.8)
Connecticut	76-22	76.9–24.6 (23.1)
Wyoming	64-28	67.0–25.4 (19.2)
South Dakota	58-34	59.8–34.9 (11.1)
Iowa	58-28	62.4–29.2 (18.0)
Indiana	64-22	66.5–21.6 (21.6)
Florida	58-22	62.1–28.3 (19.5)
Texas II (Binder A)	64-22	64.4–24.4 (22.5)
Texas II (Binder V)	64-22	64.2–24.0 (19.5)

**Values in parentheses are intermediate grades.*

from the seven field sites for the Phase II experiment. In total, 352 cores and 160 PMPC specimens were tested in this project.

For LMLC specimens, the total time between fabrication and completion of testing or the beginning of LTOA was approximately 2 weeks. After LTOA of LMLC specimens, testing was completed within an approximately 2-week period. This time frame was also applicable to the cores and PMPC specimens when cores and specimens from one field site (which included three to four mixtures) at a time arrived at the laboratory. When cores and specimens from more than one field site arrived at the same time or if other delays occurred due to equipment availability or testing schedules, these specimens were stored in a controlled-temperature room at 68°F (20°C) and tested within 2 to 5 months.

Since more than one laboratory prepared and tested specimens, round robin testing was conducted to ensure consistency, repeatability, and reproducibility of the laboratory test results. The round robin testing was limited to M_R, E*, and flow number tests, none of which are currently assessed under AASHTO Materials Reference Library (AMRL) accreditation. The HWTT was not included, as all participating laboratories are AMRL accredited for this test. The round robin testing included (1) compaction and testing of specimens using loose mix prepared by the University of California–Davis (UC Davis) and (2) testing of specimens prepared and compacted at UC Davis and sent to the other two participating laboratories. Testing of specimens prepared from supplied loose mix assessed consistency, repeatability, and reproducibility between participating laboratories of mixing, compacting (gyratory), sawing/trimming, AV content testing, and actual performance-related testing. Testing of specimens prepared at UC Davis and sent to participating laboratories assessed performance-related testing only. Details of the round robin study can be found in Appendix D.

CHAPTER 3

Findings and Applications

This chapter provides the mixture test results for the Phase I and Phase II experiments. Mixture volumetrics, stiffness, and HWTT results are summarized and analyzed for simulating plant aging and examining the effects of the selected factors on plant aging.

Phase I

Mixture Volumetrics

Table 3-1 presents the comparison of the volumetrics of LMLC specimens fabricated following the selected laboratory STOA protocols of 2 hours at 275°F (135°C) for HMA and 2 hours at 240°F (116°C) for WMA, along with PMPC specimens, in terms of G_{mm}, percentage of absorbed binder (P_{ba}), percentage of effective binder (P_{be}), and effective binder film thickness (FT_{be}). The volumetrics were calculated using the mix design aggregate gradation and asphalt content per *Superpave Mix Design* (Asphalt Institute 2001).

Figures 3-1 and 3-2 present the volumetric correlations for PMPC specimens versus LMLC specimens in terms of G_{mm} and P_{ba} values, respectively. As illustrated in Figure 3-1, most of the data points fall on the line of equality, indicating that equivalent G_{mm} values were achieved by PMPC specimens and LMLC specimens. The exceptions were mixtures from the Iowa test site that were produced as HMA and foamed WMA with highly absorptive aggregates (3.2 percent AC). A reasonable correlation in terms of P_{ba} values was also observed, as shown in Figure 3-2, when comparing the two specimen types, with the exception of the same subset of the Iowa mixtures. It can be seen here that the absorption in the plant for the Iowa high-absorption mixtures was not as great in the laboratory as in the plant. There may be numerous explanations for this but the high-absorption aggregate mixture was produced at a higher-than-planned temperature in the plant as noted in Appendix A. Based on the data summarized in Table 3-1 and Figures 3-1 and 3-2, practically equivalent mixture volumetrics were observed for PMPC specimens and LMLC specimens for a wide range of asphalt mixtures. Therefore, the selected laboratory STOA protocols of 2 hours at 275°F (135°C) for HMA and 2 hours at 240°F (116°C) for WMA were considered suitable to simulate the asphalt absorption during plant production and construction.

Simulation of Plant Aging

As mentioned previously, the laboratory STOA protocols of 2 hours at 275°F (135°C) for HMA and 2 hours at 240°F (116°C) for WMA were selected and used in the project to simulate asphalt aging during plant production and construction. To explore the correlation in binder or mixture aging induced by the selected laboratory STOA protocols versus that during plant production, M_R stiffness, E* stiffness, and HWTT RRP and rut depth at 5,000 load cycles for LMLC specimens from all nine field sites were plotted against the corresponding results obtained for PMPC specimens and cores at construction. In addition, continuous PG and FT-IR CA results for extracted and recovered binders from LMLC specimens from three field sites (i.e., Indiana, Florida, and Texas II) were plotted against the corresponding results for the binders extracted and recovered from plant loose mix and cores at construction. A detailed discussion of the results of each test is presented in the following subsections.

M_R Test Results

Figures 3-3 and 3-4 present the M_R stiffness correlation of LMLC specimens versus PMPC specimens and cores at construction, respectively. In Figure 3-3, most of the data points fall around the line of equality, which indicates that M_R stiffness for LMLC specimens with the selected laboratory STOA protocols of 2 hours at 275°F (135°C) for HMA and 2 hours at 240°F (116°C) for WMA fairly mimicked the M_R stiffness of

Table 3-1. Mixture volumetrics for PMPC and LMLC specimens.

Field Site	Mixture Type	PMPC				LMLC			
		G_{mm}	P_{ba}	P_{be}	FT	G_{mm}	P_{ba}	P_{be}	FT
Texas I	HMA	2.420	0.53	4.70	9.09	2.397	0.10	5.11	9.88
	Evotherm	2.408	0.30	4.91	9.50	2.399	0.13	5.07	9.81
	Foaming	2.400	0.15	5.06	9.77	2.407	0.28	4.93	9.53
	HMA + RAP/RAS	2.410	0.83	4.42	7.89	2.418	0.98	4.27	7.64
	Evotherm + RAP/RAS	2.420	1.02	4.24	7.58	2.417	0.96	4.29	7.67
New Mexico	HMA	2.342	0.41	5.01	10.21	2.329	0.16	5.25	10.70
	HMA + RAP	2.340	0.66	4.78	9.66	2.339	0.64	4.79	9.70
	Evotherm + RAP	2.343	0.72	4.72	9.55	2.333	0.52	4.91	9.93
	Foaming + RAP	2.335	0.56	4.87	9.85	2.349	0.84	4.61	9.32
Connecticut	HMA + RAP	2.676	1.26	3.71	8.55	2.652	0.90	4.04	9.33
	Foaming + RAP	2.675	1.24	3.72	8.59	2.658	0.99	3.96	9.14
Wyoming	HMA	2.470	0.76	4.28	8.81	2.491	1.13	3.93	8.09
	Evotherm High T	2.479	0.92	4.13	8.50	2.494	1.18	3.88	7.98
	Evotherm Ctrl T	2.487	1.06	3.99	8.22	2.501	1.30	3.76	7.75
	Foaming High T	2.485	1.03	4.03	8.29	2.497	1.24	3.83	7.88
	Foaming Ctrl T	2.470	0.76	4.28	8.81	2.505	1.37	3.70	7.61
South Dakota	HMA + RAP	2.441	0.58	4.75	7.28	2.441	0.58	4.75	7.28
	Evotherm + RAP	2.440	0.56	4.77	7.31	2.440	0.56	4.77	7.31
	Foaming + RAP	2.428	0.35	4.97	7.62	2.440	0.56	4.77	7.31
	Advera + RAP	2.432	0.42	4.90	7.51	2.432	0.42	4.90	7.51
Iowa	High Abs HMA + RAP High T	2.425	2.35	4.82	10.18	2.373	1.35	5.75	12.14
	High Abs HMA + RAP Ctrl T	2.439	2.61	4.57	9.67	2.373	1.35	5.75	12.14
	High Abs Foaming + RAP High T	2.435	2.54	4.64	9.81	2.365	1.19	5.89	12.45
	High Abs Foaming + RAP Ctrl T	2.437	2.57	4.61	9.74	2.373	1.35	5.75	12.14
	Low Abs HMA + RAP High T	2.481	0.61	4.42	9.28	2.482	0.63	4.40	9.25
	Low Abs HMA + RAP Ctrl T	2.476	0.52	4.50	9.46	2.479	0.58	4.45	9.35
	Low Abs Foaming + RAP High T	2.477	0.54	4.49	9.42	2.488	0.73	4.30	9.04
	Low Abs Foaming + RAP Ctrl T	2.474	0.49	4.54	9.53	2.489	0.75	4.29	9.00
Indiana	HMA + RAP BMP	2.451	1.31	4.77	8.12	2.458	1.32	4.65	7.92
	HMA + RAP DMP	2.446	1.48	5.00	8.55	2.443	1.43	5.05	8.64
	Advera + RAP BMP	2.448	1.29	4.84	8.24	2.456	1.43	4.71	8.02
	Foaming + RAP DMP	2.455	1.43	4.73	8.06	2.440	1.16	4.98	8.49
Florida	High Abs HMA + RAP	2.350	2.03	4.66	6.93	2.341	1.86	4.83	7.17
	High Abs Foaming + RAP	2.363	2.18	4.37	6.48	2.365	2.22	4.33	6.43
	Low Abs HMA + RAP	2.537	0.79	3.74	5.61	2.540	0.84	3.70	5.54
	Low Abs Foaming + RAP	2.548	1.09	3.64	5.46	2.540	0.96	3.76	5.65
Texas II	HMA BMP Binder A	2.402	1.39	5.06	7.97	2.393	1.11	5.11	8.10
	HMA DMP Binder A	2.415	1.34	4.65	7.28	2.393	1.11	5.11	8.10
	HMA BMP Binder V	2.395	1.26	5.19	8.17	2.392	1.16	5.21	8.20
	HMA DMP Binder V	2.411	1.26	4.71	7.38	2.392	1.16	5.21	8.20

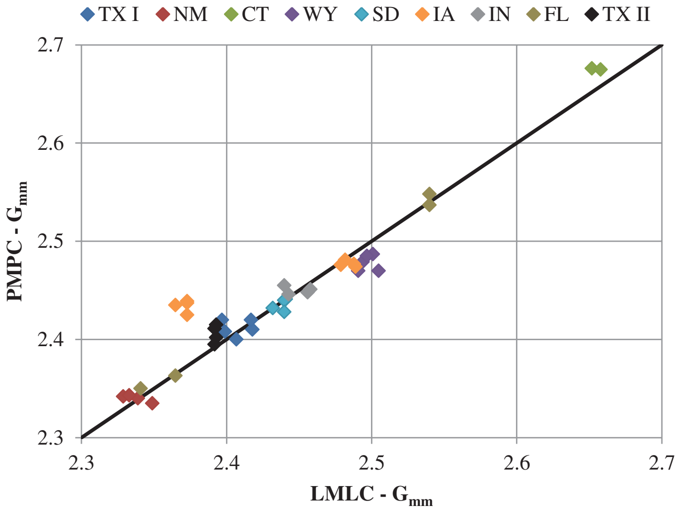


Figure 3-1. Theoretical maximum specific gravity correlation for LMLC specimens versus PMPC specimens.

the PMPC specimens. The biggest explainable exceptions are for the high-absorption mixes from Iowa, which were affected by the laboratory conditioning; the high-RAP mixture from New Mexico; and the rapidly aging asphalt mixture from Texas II. Although a reasonable correlation is observed in Figure 3-4 between the cores at construction and the LMLC specimens, the cores exhibited lower M_R stiffness, possibly due to the higher AV content and perhaps different aggregate orientation in the construction cores. Previous studies have shown that flatter aggregate orientation in field cores can lead to anisotropic behavior resulting in lower mixture-stiffness values measured in the M_R tests (Yin et al. 2013; Zhang et al. 2011).

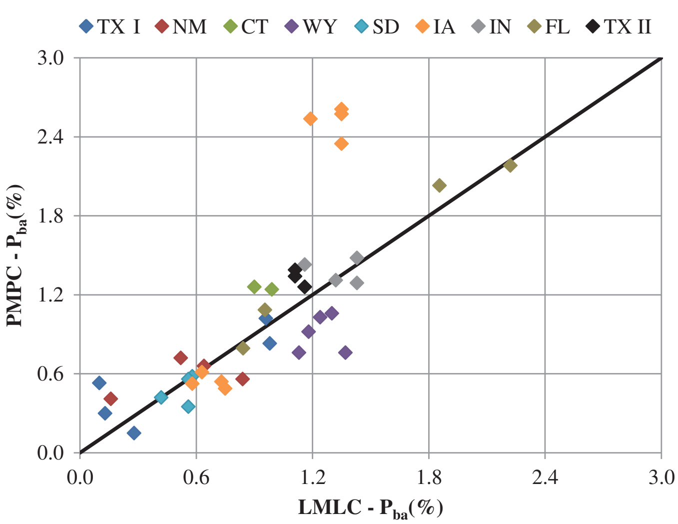


Figure 3-2. Percentage of absorbed binder correlation for LMLC specimens versus PMPC specimens.

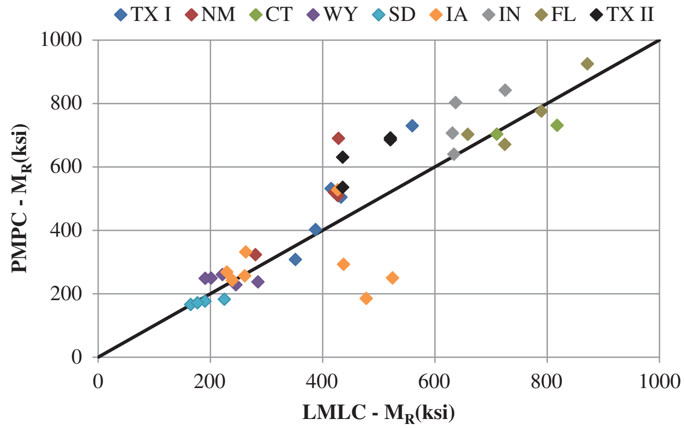


Figure 3-3. Resilient modulus stiffness correlation for LMLC specimens versus PMPC specimens.

E Test Results*

Figure 3-5 presents the correlation of E* stiffness results at 68°F (20°C) and 10 Hz for LMLC specimens versus PMPC specimens of asphalt mixtures from the Connecticut, Indiana, and Texas II field sites. Consistent with the results shown in Figure 3-3, a good correlation in E* stiffness is observed for LMLC specimens versus PMPC specimens in Figure 3-5. Therefore, the laboratory STOA protocols of 2 hours at 275°F (135°C) for HMA and 2 hours at 240°F (116°C) for WMA were able to produce laboratory asphalt mixtures with an E* stiffness equivalent to that of plant-produced asphalt mixtures. The outlier shown in Figure 3-5 is the BMP PMPC specimen of HMA from the Indiana field

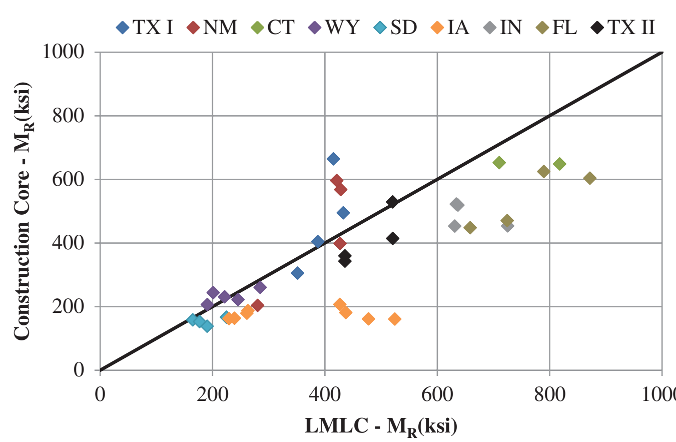


Figure 3-4. Resilient modulus stiffness correlation for LMLC specimens versus cores at construction.

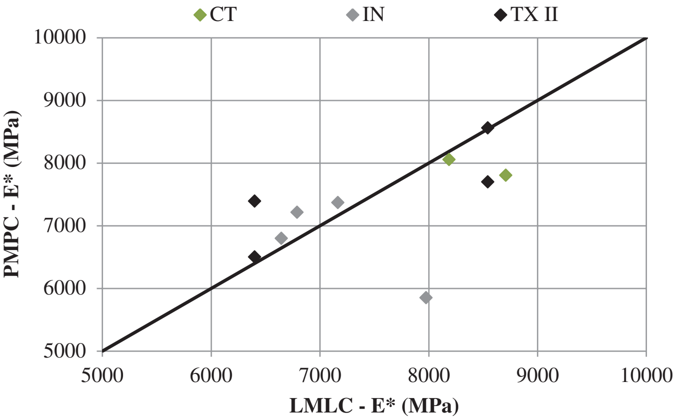


Figure 3-5. Dynamic modulus stiffness at 20°C/10 Hz correlation for LMLC specimens versus PMPC specimens.

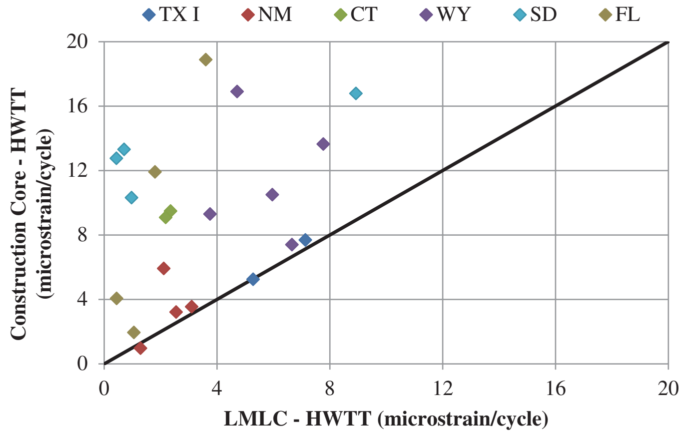


Figure 3-7. HWTT RRP correlation for LMLC specimens versus cores at construction.

site, which showed a significantly lower E* stiffness compared to its corresponding LMLC counterpart. The authors could not find a reasonable explanation for this occurrence.

HWTT Results

Figures 3-6 and 3-7 present the HWTT RRP results for LMLC specimens versus PMPC specimens and cores at construction, respectively. The asphalt mixtures included in this evaluation did not show early stripping during the tests and had LC_{SN} values greater than 3,000 load cycles. As illustrated in Figure 3-6, a reasonable correlation, indicated by values scattered about the line of equality, in HWTT RRP values between LMLC specimens and PMPC specimens was obtained, indicating the selected LTOA protocols were able to produce laboratory asphalt mixtures with rutting resistance in the HWTT equivalent to that of the mixture produced in the plant. However, a distinct trend is shown in Figure 3-7, where almost all of the data points are above the line of equality. Thus, cores at construction exhibited a higher rutting susceptibility in the HWTT compared to their corresponding LMLC specimens. The degradation and debonding of the plaster needed to fit the cores into the testing mold was likely a significant contributor to higher rut depths for cores at construction and a consequent poor correlation with the LMLC results. Therefore, there is a need for appropriate modifications to the HWTT procedure for testing field cores in the future. It is also possible that anisotropy in the field cores could have contributed to this behavior.

In addition to the HWTT RRP results, the traditional rutting resistance parameter of rut depth at 5,000 load cycles was also used to evaluate the simulation of plant aging by the selected laboratory STOA protocols. The results for LMLC specimens versus PMPC specimens and cores at construction are presented in Figures 3-8 and 3-9, respectively. Although a substantial variability in the rut depth measurements is exhibited

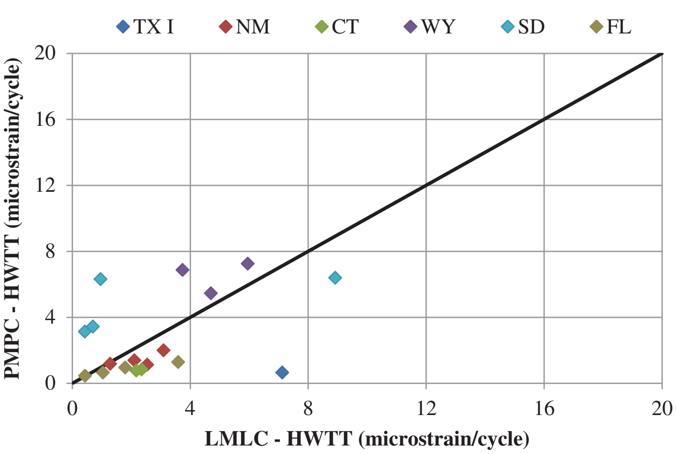


Figure 3-6. HWTT RRP correlation for LMLC specimens versus PMPC specimens.

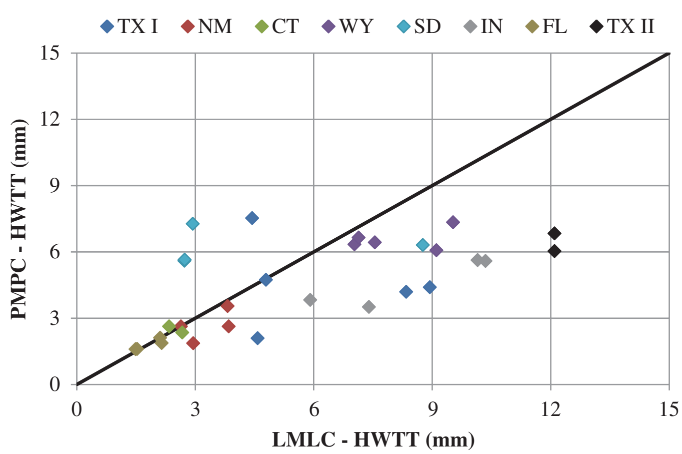


Figure 3-8. HWTT rut depth at 5,000 load cycle correlation of LMLC specimens versus PMPC specimens.

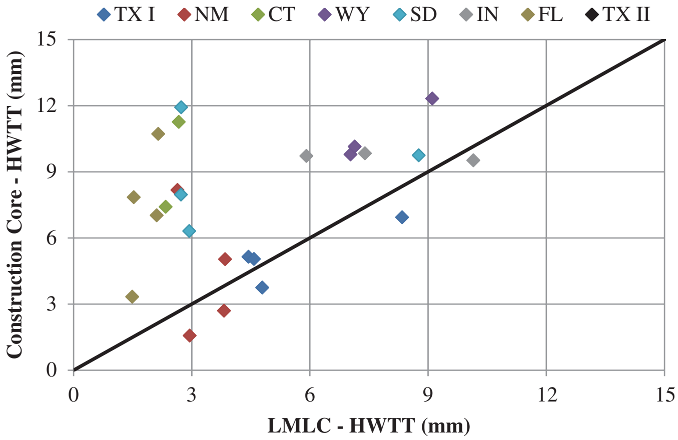


Figure 3-9. HWTT rut depth at 5,000 load cycle correlation of LMLC specimens versus cores at construction.

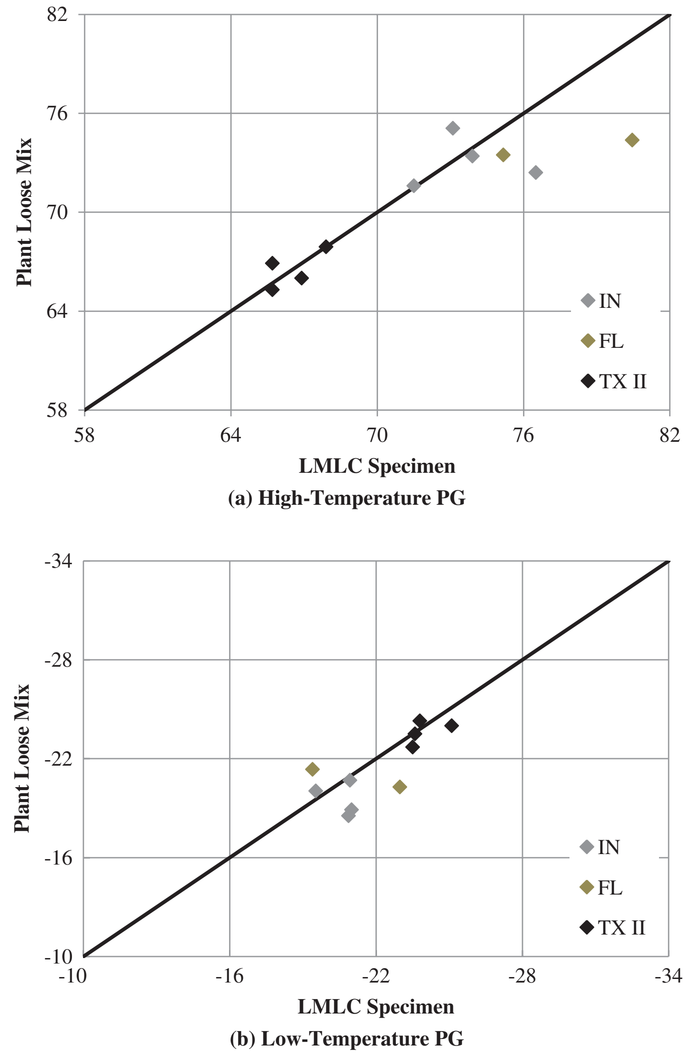


Figure 3-10. Continuous PG correlation for extracted and recovered binders from LMLC specimens versus plant loose mix.

in Figure 3-8, there is a reasonable correlation, as indicated by a scattering around the line of equality, in terms of rutting resistance between LMLC specimens and their corresponding PMPC specimens. The reduced correlation observed for specimens with high rut-depth values (i.e., 6 to 12 mm at 5,000 load cycles) is possibly due to the rut depth induced by stripping. Similar to the results shown in Figure 3-7, a higher rutting susceptibility as indicated by higher rut depths at 5,000 load cycles for cores at construction versus their corresponding LMLC specimens can also be observed in Figure 3-9. Again, HWTT results for cores at construction were probably affected by the disintegration and debonding of the plaster needed to properly fit the cores into the molds during the test as well as possible anisotropy in the field cores.

Continuous PG Results

Figure 3-10 presents the continuous high-temperature and low-temperature PG correlations for extracted and recovered binders from plant loose mix versus those from LMLC specimens. As illustrated, most of the data points fall on the line of equality, indicating equivalent continuous PG results. This correspondence indicates that the plant production and the laboratory STOA protocols of 2 hours at 275°F (135°C) for HMA and 2 hours at 240°F (116°C) for WMA produced an equivalent effect on binder stiffening. The correlations for the binders extracted from the Indiana and Florida mixtures were slightly lower than those extracted from the Texas II mixtures, possibly due to the incorporation of RAP.

Figure 3-11 presents the continuous high-temperature and low-temperature PG correlations of extracted and recovered binders from cores at construction versus those from LMLC specimens, respectively. A similar trend was shown in that most of the data points fall near the line of equality, indicating equivalent continuous PG results. The similarity in the correlation results, as shown in Figure 3-10 versus Figure 3-11, indicates that most short-term aging of asphalt binders and mixtures occurs during plant production, while the aging induced by the construction process (i.e., transportation, laydown, and compaction) may be insignificant.

FT-IR Test Results

Figures 3-12 and 3-13 present the FT-IR CA correlations of extracted and recovered binders from LMLC specimens versus those from plant loose mix and cores at construction, respectively. As illustrated, most of the data points fall near the line of equality, despite a few outliers. This correspondence indicates that the extracted and recovered binders from the plant loose mix and cores at construction experienced an

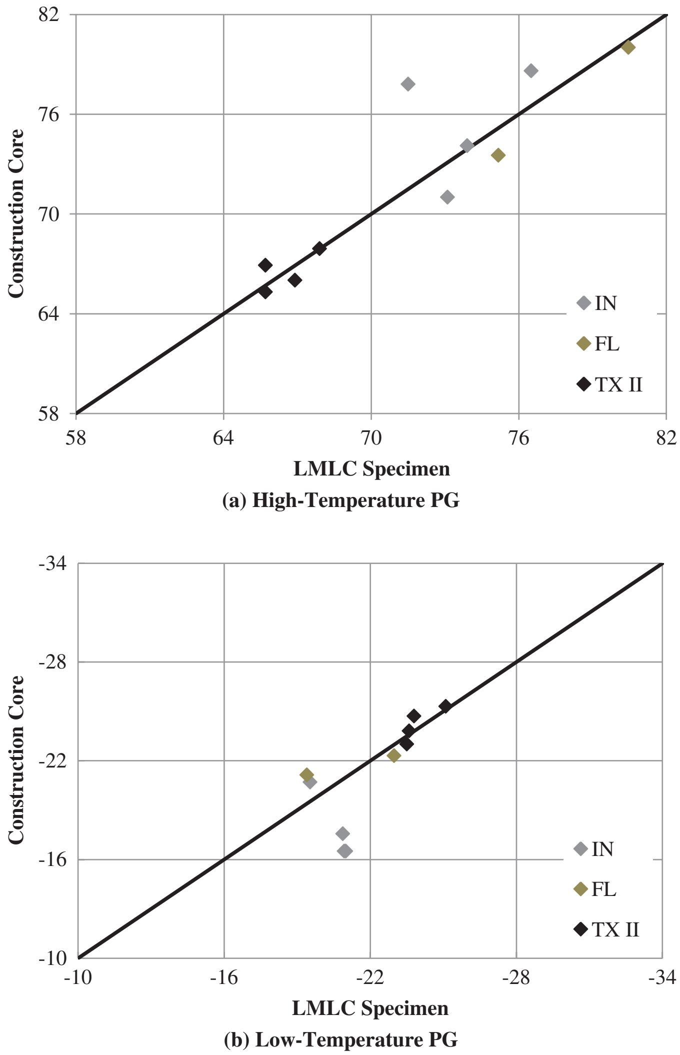


Figure 3-11. Continuous PG correlation for extracted and recovered binders from LMLC specimens versus cores at construction.

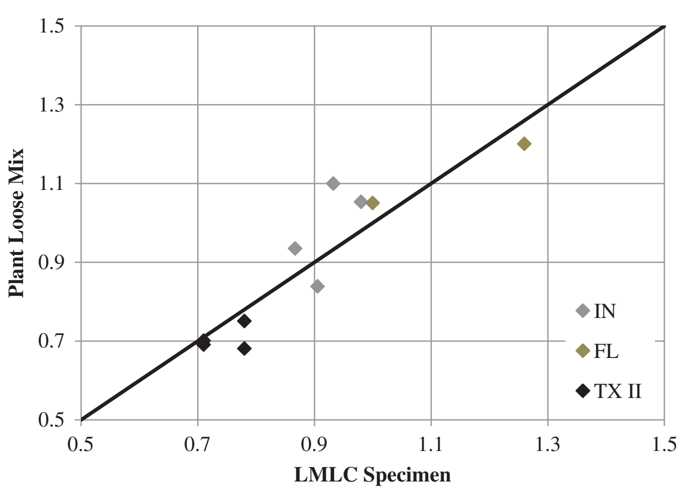


Figure 3-12. FT-IR carbonyl area correlation for extracted and recovered binders from LMLC specimens versus plant loose mix.

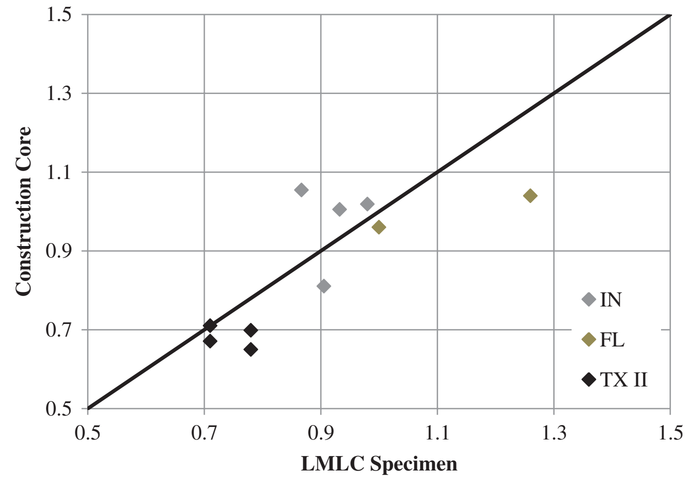


Figure 3-13. FT-IR carbonyl area correlation for extracted and recovered binders from LMLC specimens versus cores at construction.

equivalent level of oxidation during the plant production and construction as those binders extracted and recovered from the LMLC specimens fabricated using the STOA protocols of 2 hours at 275°F (135°C) for HMA and 2 hours at 240°F (116°C) for WMA.

Summary

According to the M_R and E* test results, good correlations in mixture stiffness between LMLC specimens with the selected laboratory STOA protocols and PMPC specimens and cores at construction were obtained for a wide range of asphalt mixtures from nine field sites. In addition, an approximately equivalent rutting resistance was observed for LMLC specimens and PMPC specimens in terms of HWTT RRP values and rut depths at 5,000 load cycles. A higher rutting susceptibility in the HWTT was shown for cores at construction than for the corresponding LMLC specimens, which was possibly caused by the need to plaster the cores to fit the height of the HWTT molds and subsequent disintegration of this plaster.

Although it was not an experiment design factor, different binder grades and the use of polymers were included in the study. Across a wide spectrum of grades (PG 58-22 to 76-22), the LMLC M_R test results were fairly consistent with the PMPC values. One of the most interesting observations concerning polymer modification can be seen in the New Mexico data where a virgin polymer-modified (i.e., PG 76-22) mixture had a lower stiffness than a high-RAP content mixture made with a straight PG 64-28 binder.

Good correlations in binder stiffening and oxidation were observed for the extracted and recovered binders from LMLC specimens versus plant loose mix and cores at construction, as indicated by the continuous PG and the FT-IR CA results.

Thus, this study verified the simulation of binder or mixture aging during plant production and construction by the laboratory STOA protocols of 2 hours at 275°F (135°C) for HMA and 240°F (116°C) for WMA for asphalt mixtures with a wide spectrum of materials and production parameters (aggregate type, asphalt source, recycled materials, WMA technology, plant type, and production temperature).

Identification of Factors Affecting the Performance of Short-Term Aged Asphalt Mixtures

This section presents the results of laboratory experiments to identify mixture components and production parameters—i.e., factors—with significant effects on the performance of short-term aged asphalt mixtures. These factors included (1) WMA technology, (2) production temperature, (3) plant type, (4) recycled material inclusion, (5) aggregate absorption, and (6) binder source. Detailed discussions for each factor are presented in the following subsections.

A statistical analysis was also performed to identify which factors had a significant effect on M_R stiffness. Separate statistical experiments and analyses were performed to assess the effects of each of the six factors of interest while incorporating information on field site, specimen type (i.e., cores at construction, PMPC, and LMLC), NMAS (i.e., 9.5 mm, 12.5 mm, and 19 mm), and AV content as variables. Results of the statistical analyses are presented in Appendix E.

WMA Technology (HMA versus WMA)

The M_R and HWTT results for LMLC specimens, PMPC specimens, and cores at construction from eight field sites with both HMA and WMA are shown in Figures 3-14 through 3-16; M_R stiffness and HWTT RRP and rut depths at 5,000 load cycles for HMA mixtures are plotted against those of corresponding WMA mixtures. The *x*-axis coordinate represents HMA test results and the *y*-axis coordinate represents the corresponding WMA test results. The black solid line is the line of equality, and the red dashed line illustrates the shift from the line of equality for M_R stiffness or rutting resistance parameters in the HWTT.

The M_R stiffness comparison for HMA versus WMA shown in Figure 3-14 illustrates that most of the data points are below the line of equality, indicating a higher M_R stiffness for HMA compared to WMA. According to the shift from the line of equality, the average ratio of WMA M_R stiffness over HMA M_R stiffness is approximately 0.85. Thus, the inclusion of WMA technology in the asphalt mixture was likely to produce asphalt mixtures with an approximately 15 percent lower stiffness than HMA.

Figure 3-15 presents the comparison of HWTT RRP values for HMA versus WMA. As illustrated, most of the data points

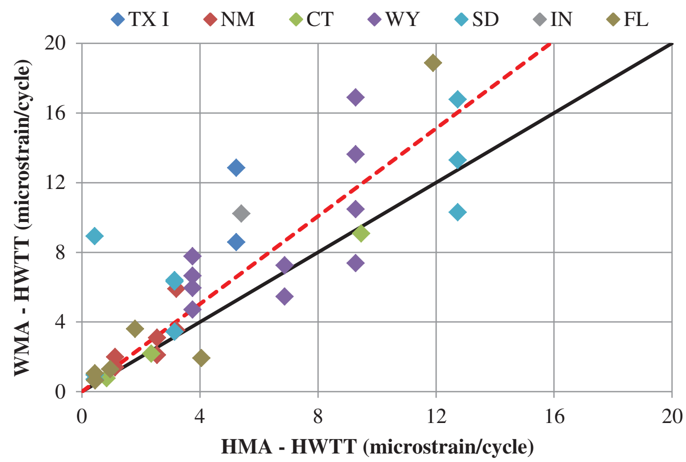


Figure 3-15. HWTT RRP comparison for HMA versus WMA.

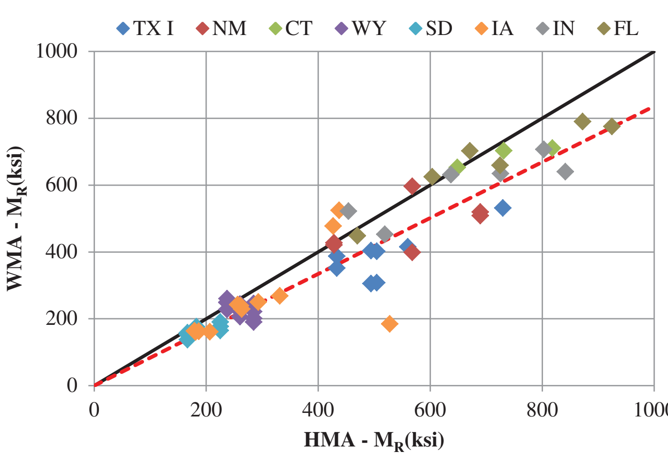


Figure 3-14. M_R stiffness comparison for HMA versus WMA.

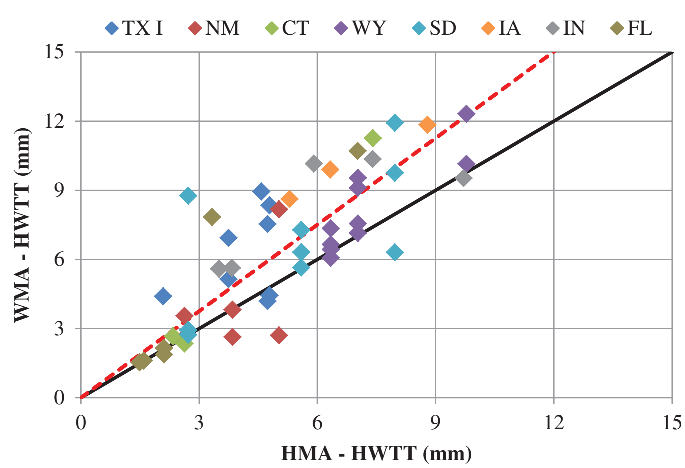


Figure 3-16. HWTT rut depth at 5,000 load cycle comparison for HMA versus WMA.

were above the line of equality, indicating a better rutting resistance in the HWTT for HMA than WMA. It may also be noticed that the data scatter tends to increase with larger values. Figure 3-16 presents the HWTT traditional rutting resistance parameter of rut depth at 5,000 load cycles. Similar to Figure 3-15, most of the data points in Figure 3-16 aligned above the line of equality. According to the slope of the line that illustrates the shift from the line of equality, the rut depth after the first 5,000 load cycles in the HWTT for WMA mixtures is approximately 1.26 times higher than that for HMA. Thus, the inclusion of WMA technology is likely to produce early-life asphalt mixtures with a higher rutting susceptibility, though the difference has not been observed in any of the field projects.

Figures 3-17 and 3-18 present the E* master curve comparisons for HMA versus WMA from the Connecticut and Indiana field sites, respectively. The comparison was performed for each specimen type available (i.e., PMPC and LMLC for Connecticut and BMP PMPC, DMP PMPC, BMP LMLC, and DMP LMLC for Indiana). For all comparisons in terms

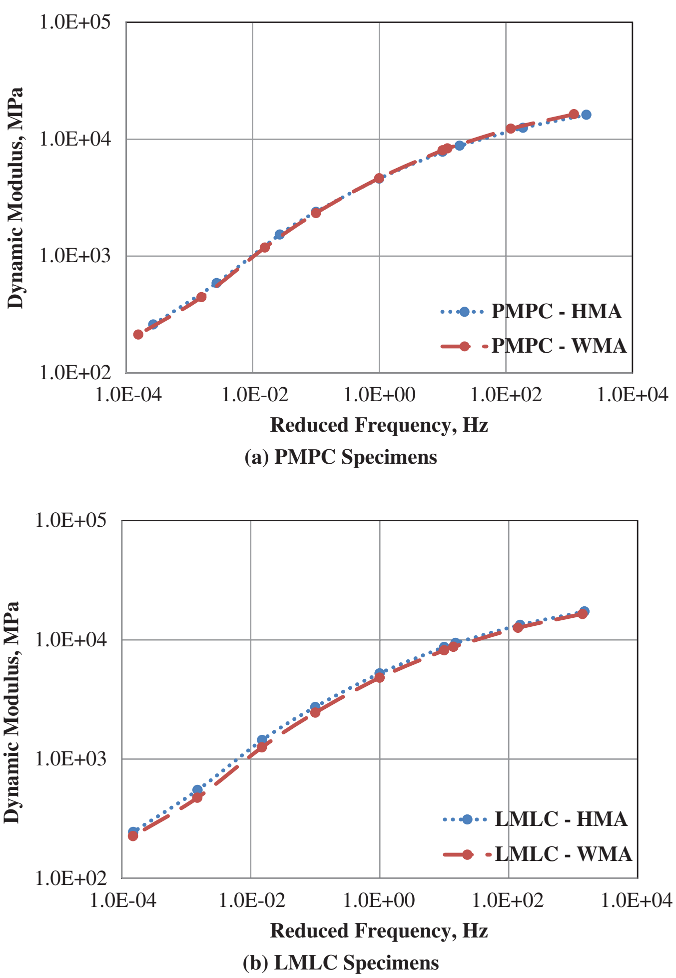


Figure 3-17. E master curve comparison for HMA versus WMA for the Connecticut field site.*

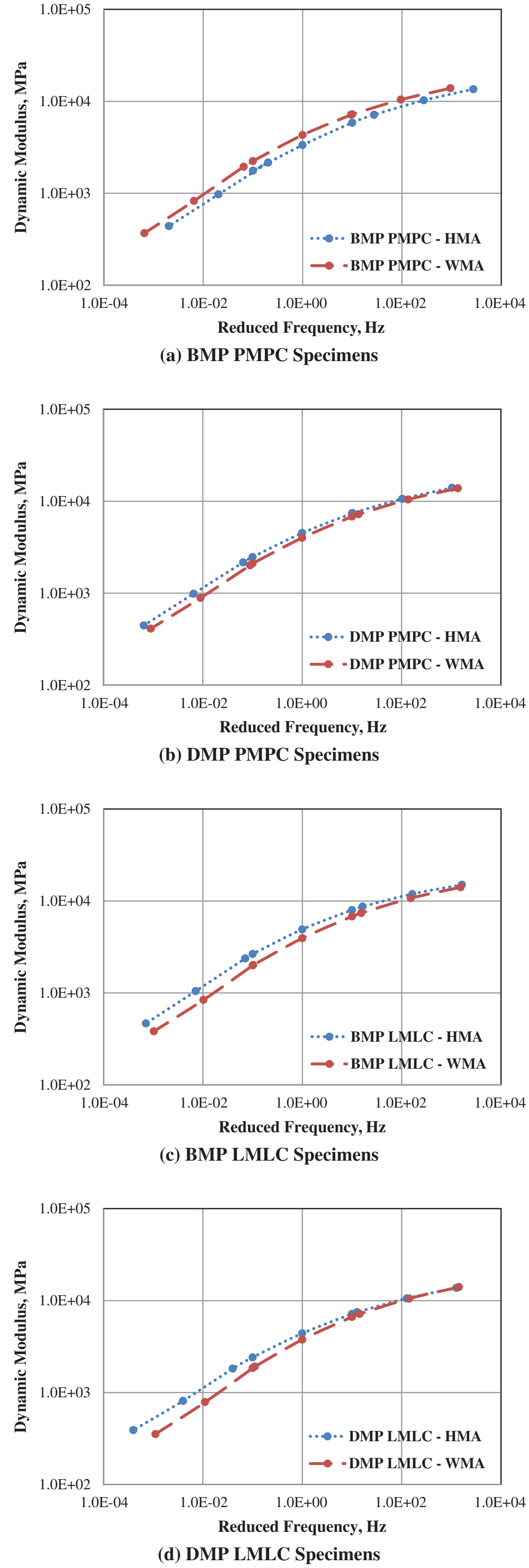


Figure 3-18. E master curve comparison for HMA versus WMA for the Indiana field site.*

of E* stiffness for HMA versus WMA, with the exception of BMP PMPC specimens from the Indiana field site, the E* master curves for HMA mixtures were above or overlapping with those for WMA counterpart mixtures, indicating higher or equivalent E* stiffness values over a wide range of testing temperatures and frequencies. For the exceptional case of BMP PMPC specimens from the Indiana field site, slightly lower E* stiffness values were observed for HMA mixtures compared to their WMA counterparts. This was the same data anomaly that was mentioned earlier (Figure 3-5).

For the statistical analysis, M_R stiffness measurements obtained from eight field sites (Connecticut, Florida, Indiana, Iowa, New Mexico, South Dakota, Texas I, and Wyoming)—with information about specimen type, WMA technology, NMAS, and AV content—were used. The analysis of covariance (ANCOVA)—having WMA technology and specimen type as main effects along with a two-way interaction effect between them (specimen type*WMA technology), NMAS and AV content as covariates, and field site as a random effect—was fitted to the data. The analysis outputs are listed in Appendix E.

The fixed-effect test results indicated that specimen type, WMA technology, AV content, and specimen type*WMA technology were statistically significant at $\alpha = 0.05$, while the effect of NMAS was not significant. When there is a significant interaction effect, the effect of each factor involved in the interaction needs to be assessed against the levels of the other factor because the effect might be different for each level of the other factor. The effect of WMA technology was assessed for each level of specimen type. Except for cores at construction, the predicted M_R stiffness was lower for WMA than for HMA. In addition, the ANCOVA model without the two-way interaction term was also fitted to the data for comparison purposes. As before, the effects of specimen type, WMA technology, and AV content were statistically significant, while the effect of NMAS was not.

Production Temperature (High vs. Control)

The MR test results for LMLC specimens, PMPC specimens, and cores at construction from the Wyoming and Iowa field sites with different production temperatures are presented in Figure 3-19, with M_R stiffness for mixtures produced at high temperatures and those at control temperatures plotted against each other. The evaluation of rutting resistance in the HWTT by RRP value and rut depth at 5,000 load cycles was not available for this factor since early stripping was observed for the majority of Iowa mixtures when tested at 122°F (50°C), with LC_{SN} values less than 3,000 load cycles and rut depths greater than the failure criteria of 12.5 mm at 5,000 load cycles. The *x*-axis coordinate represents the M_R stiffness for mixtures produced at control temperatures, and the *y*-axis coordinate represents M_R stiffness for mixtures produced at high temperatures. The black solid line is the line of equality, and the red dashed line illustrates the shift from the line of equality for M_R stiffness.

As illustrated in Figure 3-19, most of the data points fell on the line of equality, indicating equivalent M_R stiffness for those asphalt mixtures. Therefore, an increase in production temperature during mixing followed by a conditioning protocol that consisted of heating the PMPC to compaction temperature had no significant effect on mixture stiffness.

For the statistical analysis, the ANCOVA model, including production temperature, WMA technology, and specimen type as main effects along with all possible two-way interactions, AV content as a covariate, and field site as a random effect, was first fitted to the data, but none of the two-way interaction effects were statistically significant. Thus, the two-way interaction effects were removed from the model, and the ANCOVA model was fitted again to the data. The results showed that the effects of specimen type and WMA technology were statistically significant at $\alpha = 0.05$, while the effect of production temperature and AV content were not (see Appendix E).

Plant Type (BMP vs. DMP)

The M_R test and HWTT results for LMLC specimens, PMPC specimens, and cores at construction from the Indiana and Texas II field sites with different plant types are presented in Figures 3-20 and 3-21 for M_R stiffness and the HWTT traditional rutting resistance parameter of rut depth at 5,000 load cycles, respectively, with BMP-produced mixtures and DMP-produced mixtures plotted against each other. The evaluation of rutting resistance in the HWTT by RRP value was not available for this factor since early stripping was observed for the majority of Indiana and Texas II asphalt mixtures. The *x*-axis coordinate represents the test results for DMP-produced

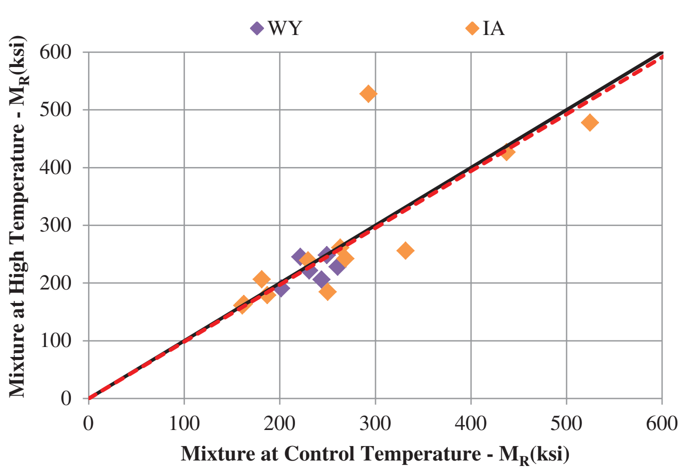


Figure 3-19. M_R stiffness comparison for asphalt mixtures produced at different temperatures.

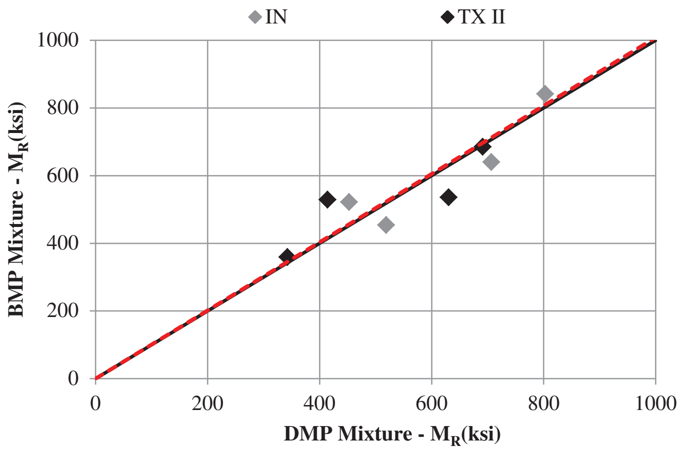


Figure 3-20. M_R stiffness comparison for asphalt mixtures produced at BMP versus DMP.

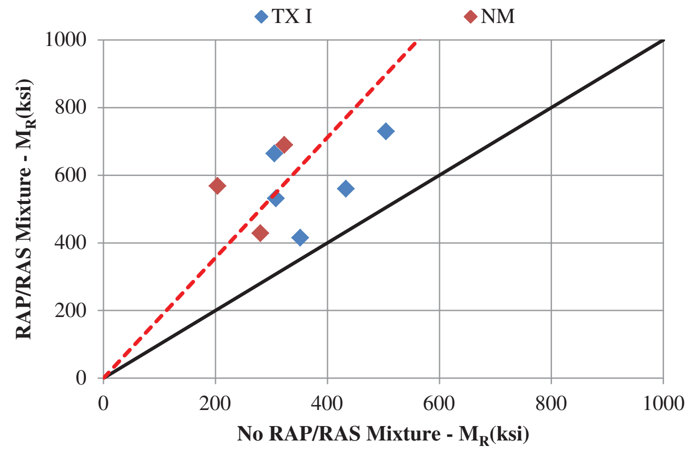


Figure 3-22. Resilient modulus stiffness comparison for asphalt mixtures with and without RAP and RAS.

mixtures, and the *y*-axis coordinate represents corresponding test results for BMP-produced mixtures. The black solid line is the line of equality, and the red dashed line illustrates the shift from the line of equality for M_R stiffness or rut depth measurements in the HWTT.

The M_R test results in Figure 3-20 show equivalent mixture stiffness was achieved by asphalt mixtures produced in a BMP and the corresponding mixtures produced in a DMP. Similar to the M_R test results, most of the data points shown in Figure 3-21 fall on the line of equality. Therefore, equivalent mixture stiffness and rutting resistance was observed for asphalt mixtures produced in a BMP and a DMP.

For the statistical analysis, the ANCOVA model including plant type and specimen type as main effects, plant type* specimen type as a two-way interaction effect, AV content as a covariate, and field site as a random effect was first fitted to the data. However, the two-way interaction effect was not statistically significant, and the ANCOVA model without the two-way interaction effect was fitted again to the data. The results (see Appendix E) show that the effects of specimen type and AV content were statistically significant at $\alpha = 0.05$, while the effect of plant type was not.

Inclusion of Recycled Material (RAP/RAS vs. No RAP/RAS)

The M_R test and HWTT results for LMLC specimens, PMPC specimens, and cores at construction from the Texas I and New Mexico field sites with mixtures with and without RAP/RAS are presented in Figures 3-22 through 3-24; M_R stiffness and HWTT RRP and rut depth at 5,000 load cycles for RAP/RAS mixtures and control mixtures without recycled materials are plotted against each other. The control mixtures from the Texas I field site were produced using a PG 70-22 binder, while

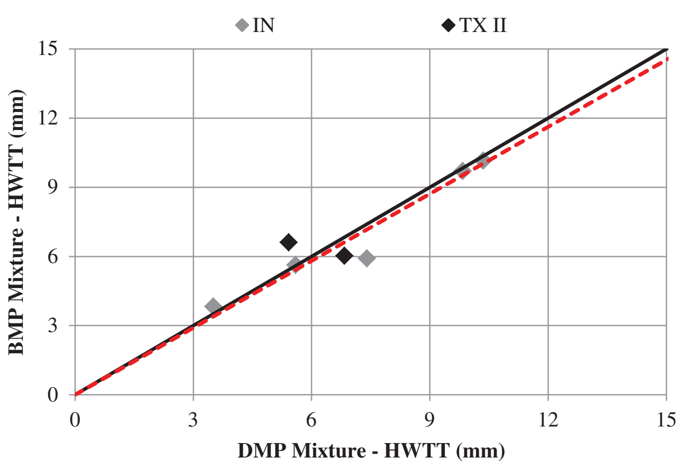


Figure 3-21. HWTT rut depth at 5,000 load cycle comparison for asphalt mixtures produced at BMP versus DMP.

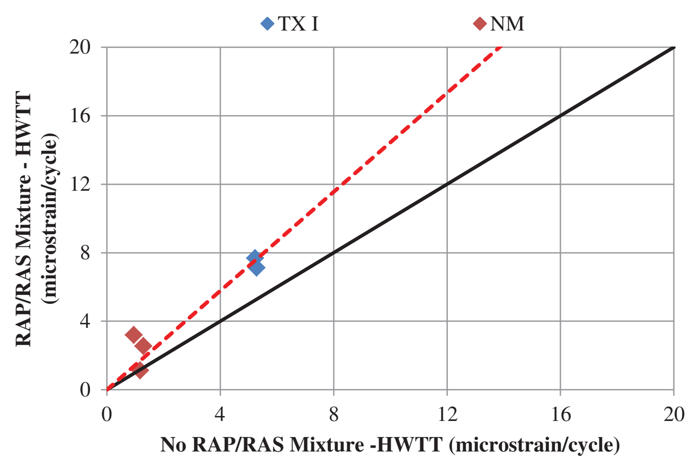


Figure 3-23. HWTT RRP comparison for asphalt mixtures with and without RAP and RAS.

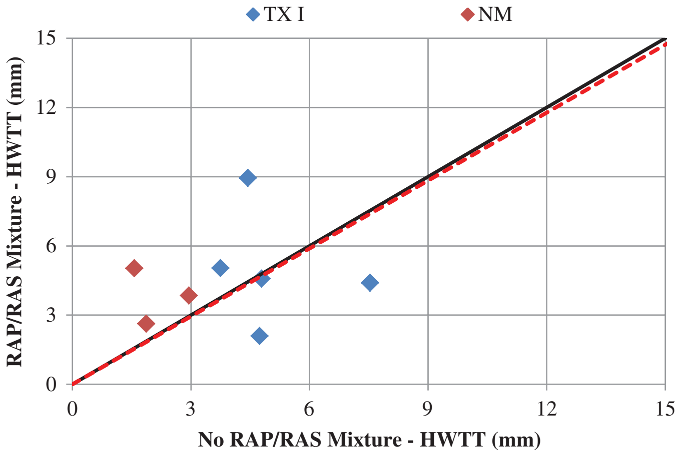


Figure 3-24. HWTT rut depth at 5,000 load cycle comparison for asphalt mixtures with and without RAP and RAS.

the RAP/RAS mixtures were produced using a softer PG 64-22 binder in conjunction with 15 percent RAP and 3 percent RAS. The RAP came from a stockpile at Ramming Paving in Austin, Texas, and the RAS was from tear-off shingles ground to 100 percent passing the 0.5-in. (12.5 mm) sieve.

The control mixtures from the New Mexico mixtures were produced using a PG 76-28 binder, while the RAP mixtures were produced using a softer PG 64-28 binder in conjunction with 35 percent RAP. The *x*-axis coordinate represents test results for the control mixtures, and the *y*-axis coordinate represents corresponding test results for RAP/RAS mixtures. The black solid line is the line of equality, and the red dashed line illustrates the shift from the line of equality for M_R stiffness or rutting resistance parameters in the HWTT.

The M_R stiffness results in Figure 3-22 indicate, based on the shift from the line of equality, that the RAP/RAS mixtures have higher stiffness as compared to the control mixtures. However, the scatter around the red dashed line indicates that the increase in mixture stiffness induced by adding recycled materials was inconsistent. Recycled materials have a profound effect on the properties of WMA and HMA as they tend to stiffen mixtures appreciably. A large difference in behavior of RAS-bearing mixtures will occur depending upon the origin of the RAS, whether it comes from manufacturing waste or tear-offs. Likewise, the character of RAP will vary depending upon its age and climatic exposure. This suggests that recycled materials (i.e., RAP and RAS) from different sources utilized in different field sites should be treated as unique materials whose properties are related to the original asphalt mixtures and in-service times and climatic conditions.

Figure 3-23 presents the comparison of HWTT RRP values for RAP/RAS mixtures versus control mixtures. As illustrated, most points were above the line of equality, indicating decreased rutting resistance in the HWTT for RAP/RAS mixtures compared to the control counterpart mixtures. This was possibly due to the softer virgin asphalt binders used in the RAP/RAS mixtures and the degree of blending with the recycled binders. According to the shift from the line of equality, the average ratio of the RRP value for RAP/RAS mixtures over the RRP value for the control mixtures was approximately 1.44. Figure 3-24 presents the traditional HWTT rutting resistance parameter of rut depth at 5,000 load cycles. No consistent trend in the comparison of RAP/RAS mixtures versus control mixtures was observed for this parameter.

For the statistical analysis, the ANCOVA model including recycled materials, specimen type, and WMA technology as main effects along with all possible two-way interaction effects—among them, AV content as a covariate and field site as a random effect—was first fitted to the data. Because the WMA technology*recycled materials interaction effect was not statistically significant at $\alpha = 0.05$, it was removed, and the ANCOVA model was fitted again to the data. The results (see Appendix E) show that the only effect that was not statistically significant at $\alpha = 0.05$ was AV content. A multiple comparison procedure (Tukey's honestly significant difference) carried out to test which of those factor levels were statistically different showed that the difference between no RAP/RAS and RAP/RAS mixtures was statistically significant for cores at construction, PMPC, and LMLC, although the amount of difference varied with specimen type. The conclusion from the statistical analysis is that, in general, mixtures with RAP/RAS had higher M_R stiffness than mixtures with no RAP/RAS although there is considerable variability due to the origin, age, and nature of the recycled materials.

Aggregate Absorption (High- vs. Low-Absorptive Aggregate)

The M_R test and HWTT results (rut depth at 5,000 load cycles) for LMLC specimens, PMPC specimens, and cores at construction from the Iowa and Florida field sites with aggregates of different absorption are presented in Figures 3-25 and 3-26, respectively. In the figures, results from asphalt mixtures using high-absorptive aggregates versus low-absorptive aggregates are plotted against each other. The evaluation of rutting resistance using the HWTT RRP value was not available for this factor since early stripping was observed for the majority of Iowa and Florida asphalt mixtures, with LC_{SN} values less than 3,000 load cycles, most likely due to the use of PG 58-28 binder at both locations. The *x*-axis coordinate represents test results for mixtures using high-absorptive aggregates, and the *y*-axis coordinate represents corresponding test results for mixtures using low-absorptive aggregates. The black solid line is the line of equality, and the red dashed line illustrates the shift from the line of equality for the M_R stiffness or rut depth measurements in the HWTT.

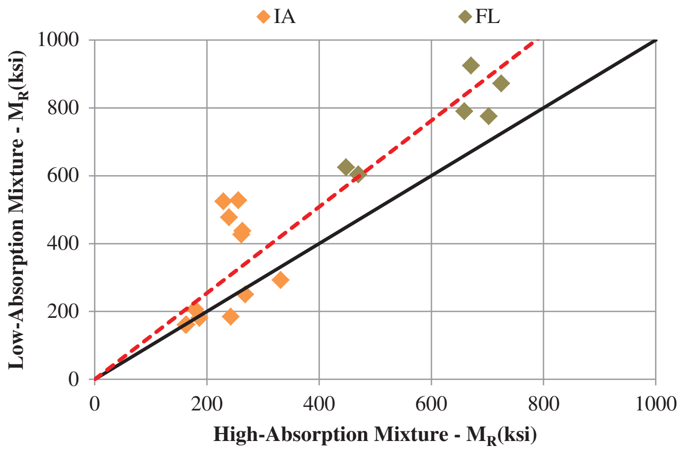


Figure 3-25. ***Resilient modulus stiffness comparison for asphalt mixtures using high-absorptive versus low-absorptive aggregates.***

The M_R stiffness comparison for mixtures using high- versus low-absorptive aggregates in Figure 3-25 shows that most of the data points are above the line of equality, indicating a higher M_R stiffness for mixtures using low-absorptive aggregates compared to those with high-absorptive aggregates. As evident in Figure 3-25, mixtures with highly absorptive aggregates may produce results that are variable when they are subjected to the recommended STOA. Figure 3-26 presents the HWTT results in terms of rut depth at 5,000 load cycles. Consistent with Figure 3-25, most of the data points in Figure 3-26 are below the line of equality, indicating better rutting resistance in the HWTT for mixtures using low-absorptive aggregates. The reduced stiffness and rutting resistance observed for the mixtures with high-absorptive aggregates could be due to the thicker FT_{be} that resulted from incorporating higher binder contents, as indicated by the higher P_{be} and FT values listed in Table 3-1.

For the statistical analysis, the ANCOVA model—including aggregate absorption, specimen type, and WMA technology as main effects; AV content as a covariate; and field site as a random effect—was fitted to the M_R stiffness measurements obtained from the Iowa and Florida field sites. The results show that the effects of specimen type, WMA technology, and aggregate absorption were statistically significant at $\alpha = 0.05$, while the effect of AV was not (see Appendix E for details).

Binder Source (Binder A vs. Binder V)

The M_R test results for LMLC specimens, PMPC specimens, and cores at construction from the Texas II field site with different binder sources are presented in Figure 3-27, with M_R stiffness for mixtures using Binder A plotted against those of corresponding mixtures using Binder V. The evaluation of rutting resistance in the HWTT by RRP value and rut depth at 5,000 load cycles was not available for this factor since early stripping in the HWTT was observed for all Texas II mixtures. The *x*-axis coordinate represents M_R stiffness for mixtures using Binder A, and the *y*-axis coordinate represents M_R stiffness for mixtures using Binder V. The black solid line is the line of equality, and the red dashed line illustrates the shift from the line of equality for M_R stiffness.

As shown in Figure 3-27, most of the data points are below the line of equality, indicating a higher M_R stiffness for asphalt mixtures using Binder A than the mixtures using Binder V.

Figure 3-28 presents the E* master curve comparisons for PMPC specimens and LMLC specimens of asphalt mixtures with Binder A versus Binder V. The comparison was performed for each specimen type (i.e., BMP PMPC, DMP PMPC, and

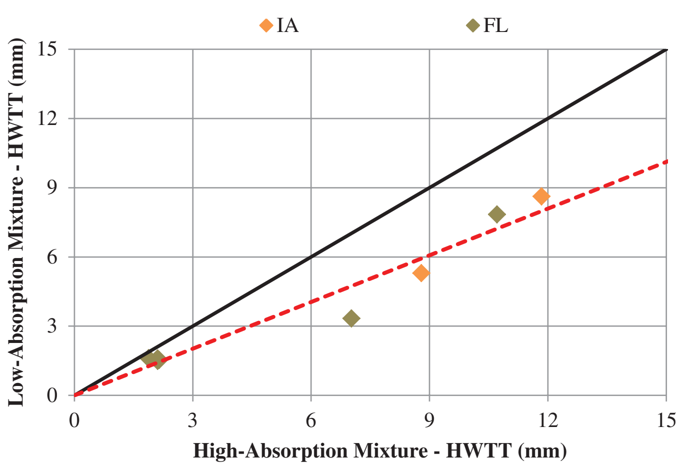


Figure 3-26. ***HWTT rut depth at 5,000 load cycle comparison for asphalt mixtures using high-absorptive versus low-absorptive aggregates.***

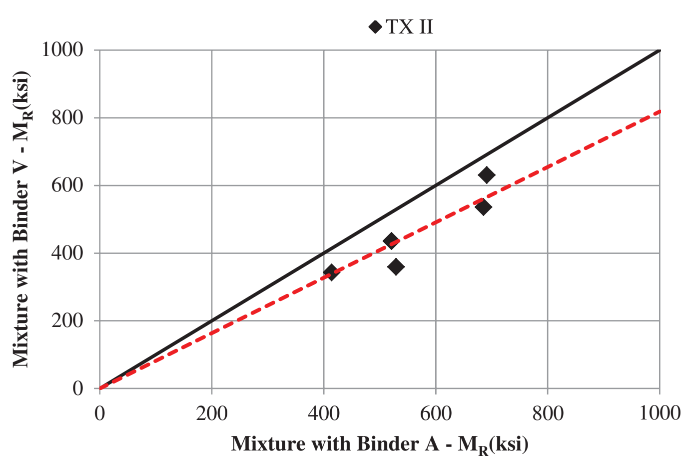
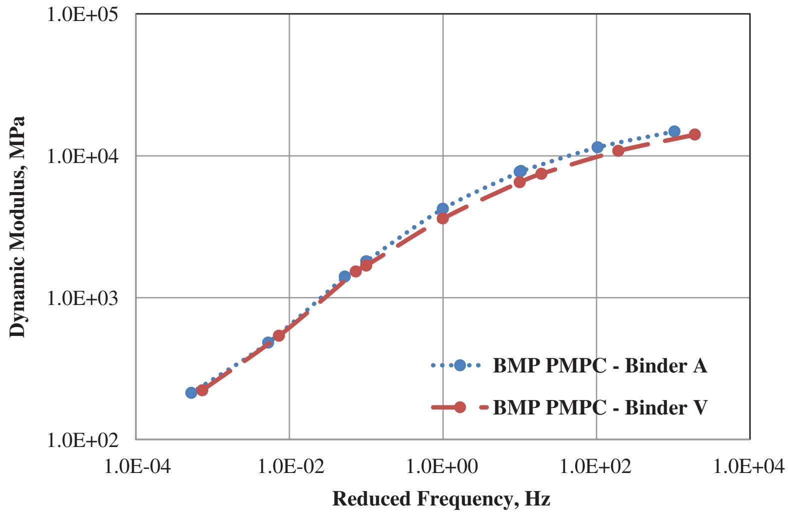
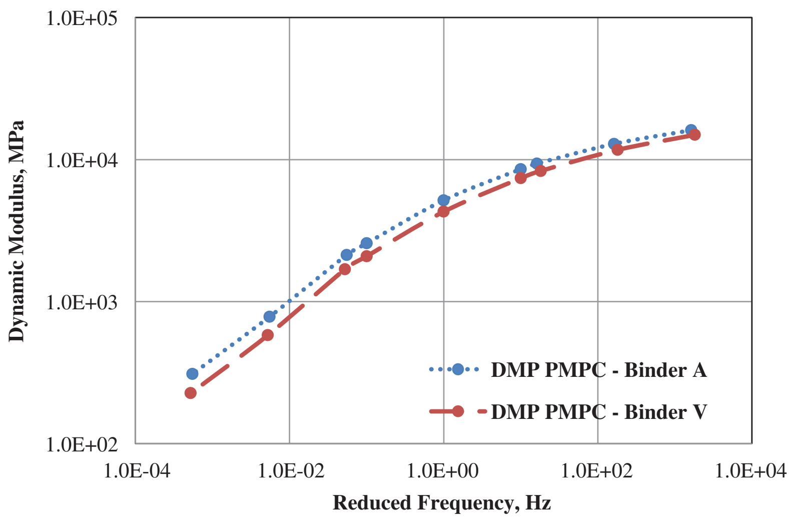


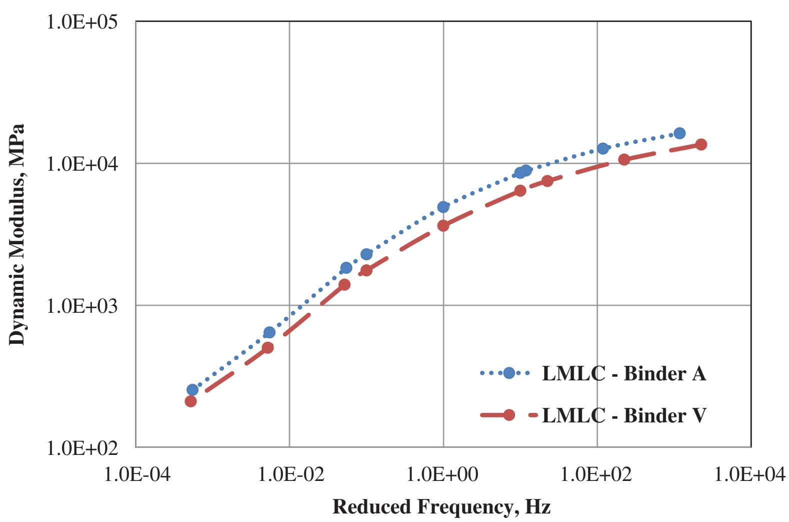
Figure 3-27. ***Resilient modulus stiffness comparison for asphalt mixtures using different binder sources.***



(a) BMP PMPC Specimens



(b) DMP PMPC Specimens



(c) LMLC Specimens

Figure 3-28. E* master curve comparison for asphalt mixtures using different binder sources for the Texas II field site.

LMLC). As illustrated, the E* master curves for mixtures with Binder A were consistently above the curves obtained for mixtures with Binder V. Therefore, binder source exhibited a significant effect on the performance of short-term aged asphalt mixtures and, consequently, asphalt mixtures using the same performance-graded binders from different sources could exhibit substantially different mixture performance in terms of stiffness and rutting resistance.

For the statistical analysis, the ANCOVA model included binder source and specimen type as main effects and AV content as a covariate; originally, a model including a two-way interaction—binder source*specimen type—was used, but the interaction was not statistically significant. The results detailed in Appendix E show that the effects of binder source, specimen type, and AV content were all statistically significant at $\alpha = 0.05$. Specifically, Binder A yielded significantly higher M_R stiffness than Binder V.

Summary

In this section, the effects of various mixture components and production parameters including WMA technology, production temperature, plant type, recycled material inclusion, aggregate absorption, and binder source on the performance of short-term aged asphalt mixtures were evaluated. The correlations in terms of M_R stiffness and HWTT rutting resistance parameters including RRP and rut depth at 5,000 load cycles were performed for each factor, and the results are summarized in Table 3-2. Mixture components and production parameters with significant effects were identified based on the magnitude of the slope of the shifted line with respect to the line of equality being greater than 1.05 or smaller than 0.95 (i.e., 5 percent off from the line of equality) and corroborated via statistical analysis.

As shown in Table 3-2, binder source, aggregate absorption, WMA technology, and inclusion of recycled materials had significant effects on stiffness and rutting resistance of short-term aged asphalt mixtures. However, no significant effect from plant type and production temperature was observed.

Based on the results of Phase I, guidance for conducting short-term aging studies for different factors affecting asphalt mixtures was prepared. This guidance is found in Appendix F, Proposed AASHTO Recommended Practice for Conducting Plant Aging Studies. The guidance is applicable to the experiment design, mix design, asphalt mix sampling, plant sample compaction, performance testing, and data analysis.

Phase II

Quantification of Field Aging

CDD (32°F [0°C] base) is proposed to provide a basis for field aging and to account for the differences in construction dates and climates for various field sites. The CDD values for the seven field sites included in the Phase II experiment are presented in Figure 3-29; they are noticeably different and are able to provide a distinct indication of the individual climatic characteristics. Specifically, the average slopes (i.e., secant slopes) of the curves for the Texas I, New Mexico, and Florida field sites are significantly steeper than those located

Table 3-2. Summary of the effects of mixture components and production parameters on the performance of short-term aged asphalt mixtures.

Factor	M_R Stiffness			HWTT RRP		HWTT Rut Depth	
	Slope Magnitude	Significant Effect	Statistically Significant	Slope Magnitude	Significant Effect	Slope Magnitude	Significant Effect
WMA Technology	0.836	Yes	Yes	1.259	Yes	1.251	Yes
Production Temperature	0.985	No	No	N/A			
Plant Type	1.008	No	No	N/A		0.968	No
Inclusion of Recycled Materials	1.779	Yes	Yes	1.44	Yes	0.981	No
Aggregate Absorption	1.271	Yes	Yes	N/A		0.675	Yes
Binder Source	0.818	Yes	Yes	N/A			

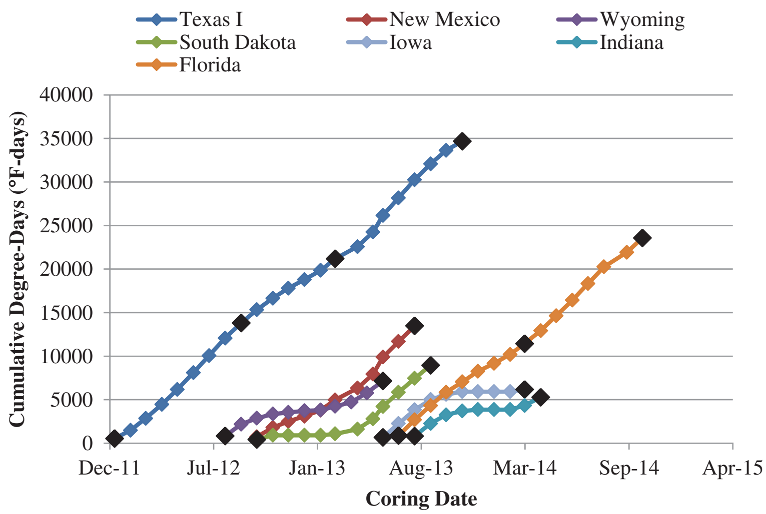


Figure 3-29. Cumulative degree-days for various field sites.

in colder climatic zones, including Wyoming, South Dakota, Iowa, and Indiana, due to differences in pavement in-service temperatures.

Additionally, the construction date has a significant effect on the CDD value and a subsequent effect on field aging of asphalt mixtures. For example, the South Dakota field site shown in Figure 3-29 was constructed in October 2012, and the pavement went through the 2012 winter prior to the 2013 summer. Consequently, the CDD curve was flat for the first several months (corresponding to the winter season); afterwards, the slope of the curve increased due to the high in-service temperatures during the summer. On the basis of these considerations, field sites with different construction dates and climates will have different CDD values for a given pavement in-service time. Specifically, field sites located in warmer climates and constructed in the spring or summer are likely to experience more severe initial field aging due to higher CDD values compared to those located in colder climates and constructed in the fall or winter.

In Figure 3-29, data points highlighted in black represent the time when field cores were acquired. It was the intent to have, at a minimum, construction cores and, depending upon the time of construction, one or more subsequent cores acquired 8 to 12 months apart. The mixture test results of these cores were used to evaluate the mixture stiffness and rutting resistance evolution with field aging.

Figures 3-30 and 3-31 present the plots of the CDD values for post-construction cores versus their associated M_R ratios and HWTT RRP ratios, respectively; the data points represent the average property ratio values for each field site, and the adjusted line represents the exponential function from Equation (2-6). As illustrated, both M_R ratio and HWTT RRP ratio exhibit a significant increase with CDD values. According to the coefficient of determination (R^2) values shown in

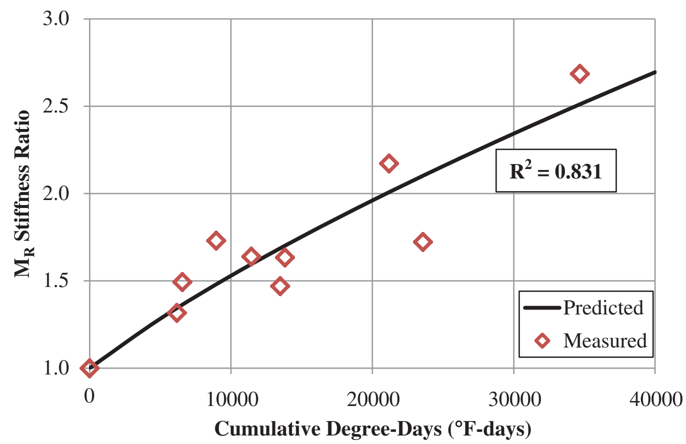


Figure 3-30. Resilient modulus ratio versus cumulative degree-days.

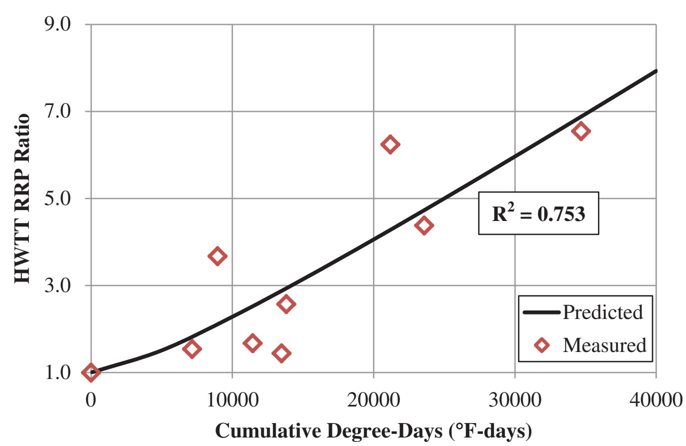


Figure 3-31. HWTT RRP ratio versus cumulative degree-days.

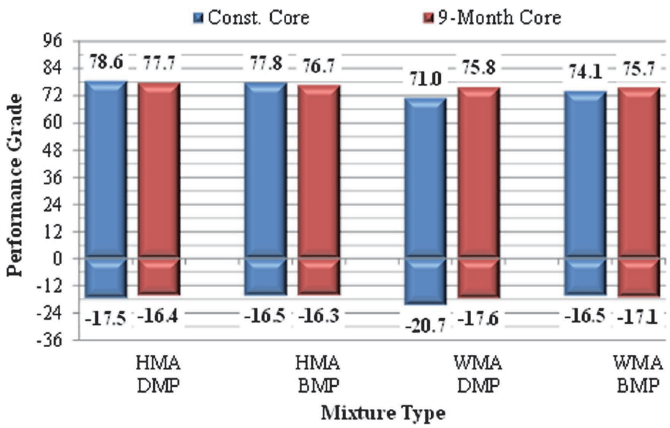


Figure 3-32. Continuous PG results for Indiana post-construction cores versus cores at construction.

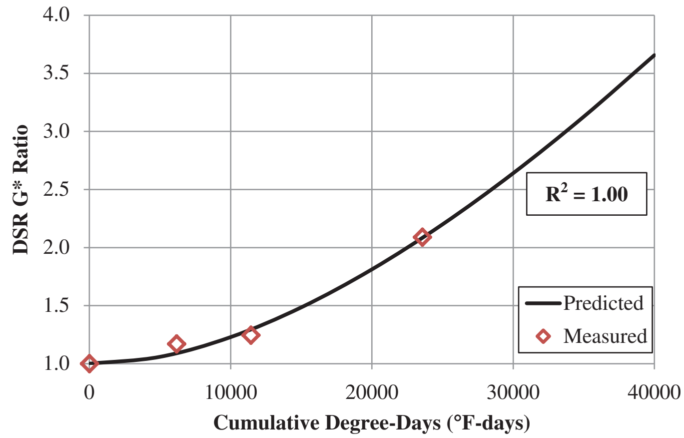


Figure 3-34. DSR complex modulus ratio versus cumulative degree-days.

Figures 3-30 and 3-31, it is reasonable to use the M_R ratio or HWTT RRP ratio as a function of CDD values to quantify mixture aging in the field.

Figures 3-32 and 3-33 present the continuous PG results for extracted and recovered binders from cores at construction and post-construction from the Indiana and Florida field sites, respectively. As illustrated, for most of the extracted and recovered binders, post-construction cores (i.e., 9-month cores from the Indiana field site, and 9-month and 15-month cores from the Florida field site) show higher continuous performance grades than the corresponding cores at construction, indicating increased binder stiffness but a reduced ductility with field aging.

Figures 3-34 and 3-35 present the plots of the CDD values for Indiana and Florida post-construction cores versus their associated DSR G* ratios and FT-IR CA ratios, respectively. The data points represent the average DSR G* ratio and FT-IR CA ratio for each field site and pavement in-service time, and the predicted lines represent the exponential function from Equation (2-6). As illustrated, both the DSR G* ratio and the FT-IR CA ratio exhibit an increase with CDD values, indicating that a significant level of binder stiffening and oxidation occurred during pavement in-service life. Although limited binder DSR G* at 77°F (25°C) and FT-IR CA results were used to determine these correlations, the results are promising.

Correlation of Field Aging with Laboratory LTOA Protocols

As previously introduced, binder or mixture property ratios (M_R ratio, HWTT RRP ratio, DSR G* ratio, and FT-IR CA ratio) were proposed to quantify the evolution of mixture stiffness, rutting resistance, and binder oxidation with field

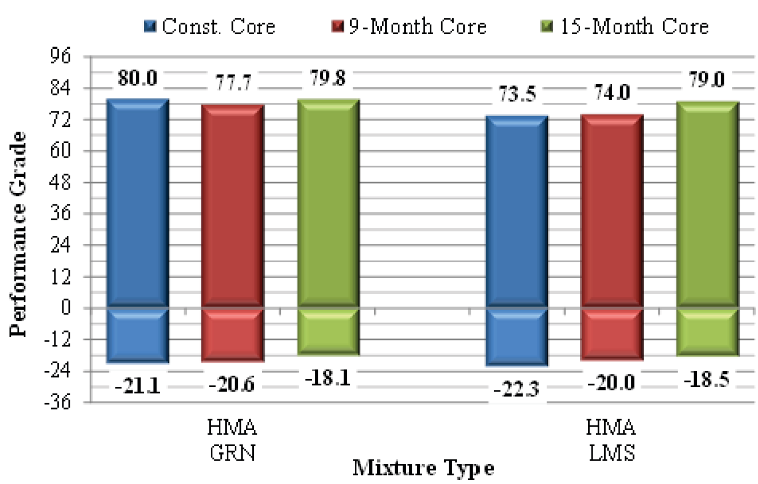


Figure 3-33. Continuous PG results for Florida post-construction cores versus cores at construction.

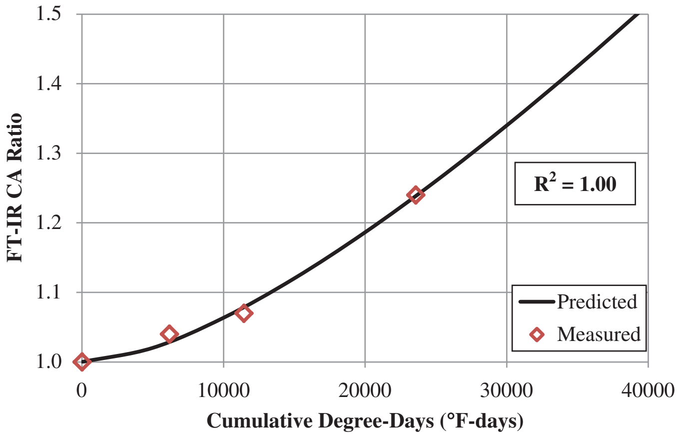


Figure 3-35. FT-IR carbonyl area ratio versus cumulative degree-days.

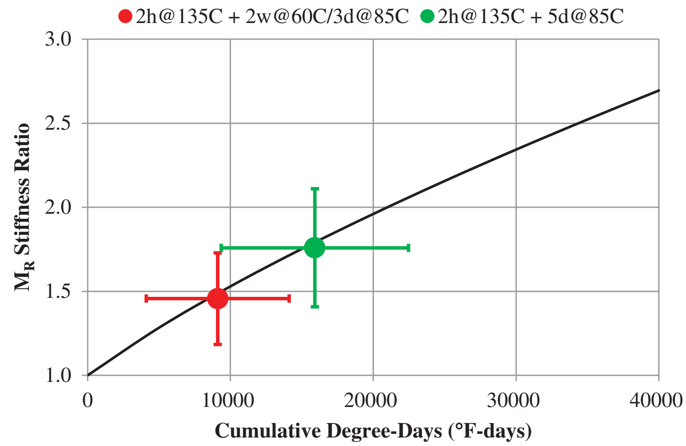


Figure 3-36. Resilient modulus ratio correlation between field aging and laboratory LTOA protocols (based upon Figure 3-30).

aging. They were defined as the ratios of binder or mixture properties of short-term aged specimens to those properties of long-term aged specimens. For specimens fabricated in the laboratory, short-term aged mixtures include LMLC specimens with STOA protocols of 2 hours at 275°F (135°C) for HMA and 2 hours at 240°F (116°C) for WMA, and long-term aged mixtures refer to HMA and WMA LMLC specimens with STOA protocol of 2 hours at 275°F (135°C) followed by LTOA protocols of 5 days at 185°F (85°C) or 2 weeks at 140°F (60°C). For field specimens, short-term aged mixtures correspond to construction cores, while long-term aged specimens refer to cores acquired several months after construction.

Findings from Phase I of this project indicated that the selected STOA protocols of 2 hours at 275°F (135°C) for HMA and 2 hours at 240°F (116°C) for WMA were representative of PMPC specimens as well as construction cores in terms of mixture stiffness and rutting resistance, and binder stiffening and oxidation. Thus, the correlation between the long-term field aging and the selected laboratory LTOA protocols in terms of mixture stiffness and binder oxidation could also be made on the basis of M_R ratio, DSR G* ratio, and FT-IR CA ratio values for long-term aged cores and LMLC specimens and their corresponding extracted and recovered binders.

Mixture test results showed that the average M_R ratio values for all LMLC specimens with STOA protocol of 2 hours at 275°F (135°C) plus LTOA protocols of 2 weeks at 140°F (60°C) and 5 days at 185°F (85°C) from the selected field sites were approximately 1.46 and 1.76, respectively. The higher M_R ratio value indicated that the LTOA protocol at 185°F (85°C) produced a greater level of mixture aging compared to that at 140°F (60°C), though associated with a shorter aging time period (i.e., 5 days versus 2 weeks). This suggests that mixture aging was more sensitive to aging temperature than to aging time in the LTOA protocol, which is consistent with the findings from NCHRP 9-49 (Epps Martin et al. 2014).

Figure 3-36 illustrates the correlation of field aging and laboratory LTOA protocols on M_R stiffness. The average M_R ratio values for LMLC specimens with STOA protocol of 2 hours at 275°F (135°C) plus LTOA protocols of 2 weeks at 140°F (60°C) and 5 days at 185°F (85°C) were plotted as markers by crossing the curve of the M_R ratio versus CDD values, as presented in Figure 3-30. The vertical and horizontal error bars represent one standard deviation from the average M_R ratio values and corresponding CDD values, respectively. As illustrated, laboratory LTOA protocols of 2 weeks at 140°F (60°C) and 5 days at 185°F (85°C) were able to produce mixture aging equivalent to an average of 9,100 and 16,000 CDD, respectively, in the field.

A subset of LMLC specimens from the Indiana and Florida field sites was aged using an additional LTOA protocol of 3 days at 185°F (85°C), because the aging induced by the standard laboratory LTOA protocol of 5 days at 185°F (85°C) per AASHTO R35 was more significant than that of 2 weeks at 140°F (60°C). The property ratios for the Indiana and Florida LMLC specimens with different LTOA protocols were compared against each other to determine if an equivalent level of laboratory aging could be produced by LTOA protocols of 3 days at 185°F (85°C) and 2 weeks at 140°F (60°C). Figure 3-37 presents the comparison of different LTOA protocols in terms of CDD after equating their M_R ratio values to the curve presented in Figure 3-30. For both the Indiana and Florida mixtures, equivalent CDD values were achieved for LTOA protocols of 3 days at 185°F (85°C) and 2 weeks at 140°F (60°C), which were significantly lower than those from 5 days at 185°F (85°C). Therefore, laboratory STOA protocol of 2 hours at 275°F (135°C) plus LTOA protocols of 3 days at 185°F (85°C) and 2 weeks at 140°F (60°C) were able to produce an equivalent level of mixture aging, which was less than the 5 days at 185°F (85°C) aging protocol.

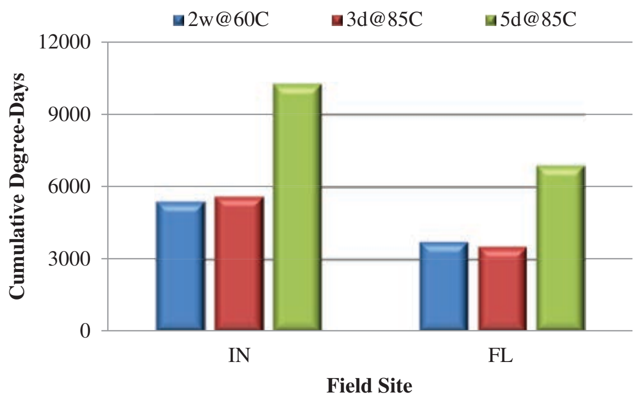


Figure 3-37. Comparison of cumulative degree-days achieved by different LTOA protocols for the Indiana and Florida field sites.

The DSR G* results measured at 77°F (25°C) indicate that the average DSR G* ratio values for the Indiana extracted and recovered binders and Florida recovered binders and LMLC specimens with LTOA protocols of 2 weeks at 140°F (60°C) and 5 days at 185°F (85°C) were approximately 1.24 and 1.47, respectively. In addition, the binder oxidation results from the FT-IR CA ratio values for the two LTOA protocols were approximately 1.11 (2 weeks at 140°F [60°C]) and 1.15 (5 days at 185°F [85°C]), which may not be practically different. Consistent with the mixture-stiffness results discussed previously, the LTOA protocol of 5 days at 185°F (85°C) produced a greater level of binder stiffening and oxidation compared to that of 2 weeks at 140°F, indicating a greater sensitivity of binder aging characteristics to LTOA temperature than to LTOA time.

Figure 3-38 presents the correlation of field aging and laboratory LTOA protocols in terms of the binder stiffening effect. The average DSR G* ratio values for extracted and recovered binders from LMLC specimens with the two LTOA protocols were plotted as markers by crossing the curve of the DSR G* ratio versus CDD values, as shown in Figure 3-34. The vertical and horizontal error bars represent one standard deviation from the average DSR G* ratio values and the corresponding CDD values for six different mixtures (i.e., four Indiana mixtures and two Florida mixtures). As illustrated by the DSR G* results, laboratory LTOA protocols of 2 weeks at 140°F (60°C) and 5 days at 185°F (85°C) were able to produce binder stiffening equivalent to an average of 10,200 and 14,300 CDD, respectively. These CDD values were close to those determined using the M_R ratio results shown in Figure 3-36. Figure 3-39 presents the correlation of binder G* stiffness at 77°F (25°C) and mixture M_R stiffness at 77°F (25°C) for 32 Indiana and Florida field cores and LMLC specimens, as well as the corresponding extracted and recovered binders. As illustrated, similarity between binder stiffness and mixture stiffness was observed. Note that M_R is influenced by more factors than just binder aging, such as air voids and aggregate absorption.

Figure 3-40 illustrates the correlation of field aging and laboratory LTOA protocols in terms of the effect on binder oxidation (i.e., FT-IR CA). The average FT-IR CA ratio values for extracted and recovered binders from LMLC specimens with STOA and LTOA protocols were plotted as markers by crossing the curve of the FT-IR CA ratio versus CDD values, as presented in Figure 3-35. The vertical and horizontal error bars represent one standard deviation from the average FT-IR CA ratio values and the corresponding CDD values for six different mixtures (i.e., four Indiana mixtures and two Florida mixtures). As illustrated by the binder oxidation results, laboratory LTOA protocols of 2 weeks at 140°F (60°C) and 5 days at 185°F (85°C) were able to produce binder oxidation equivalent to an average of 14,100 and 16,900 CDD, respectively,

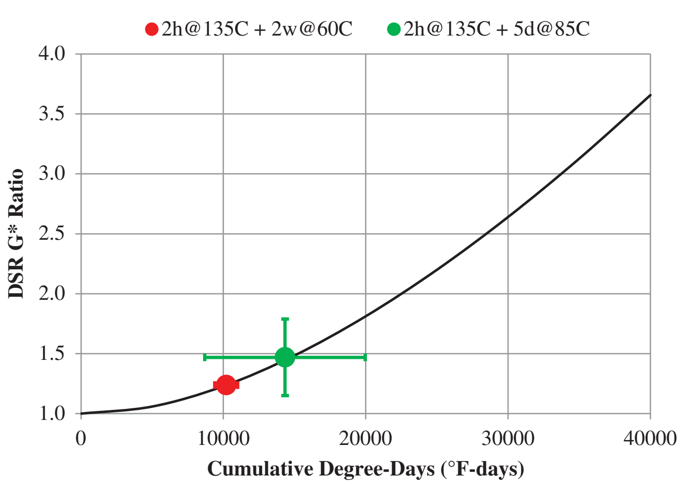


Figure 3-38. DSR complex modulus ratio correlation between field aging and laboratory LTOA protocols.

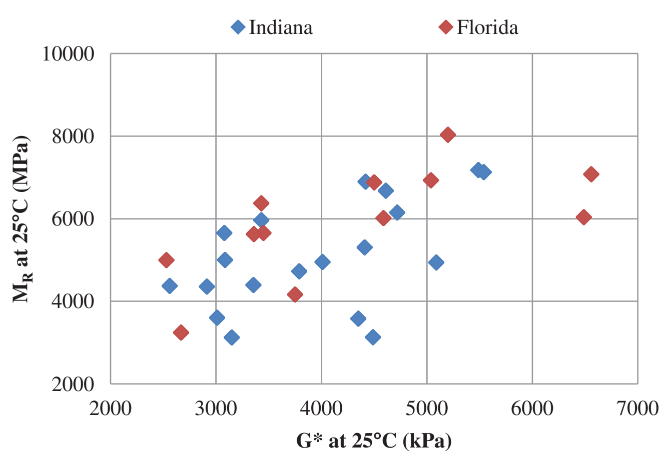


Figure 3-39. Correlation between binder complex modulus stiffness and LMLC mixture resilient modulus stiffness.

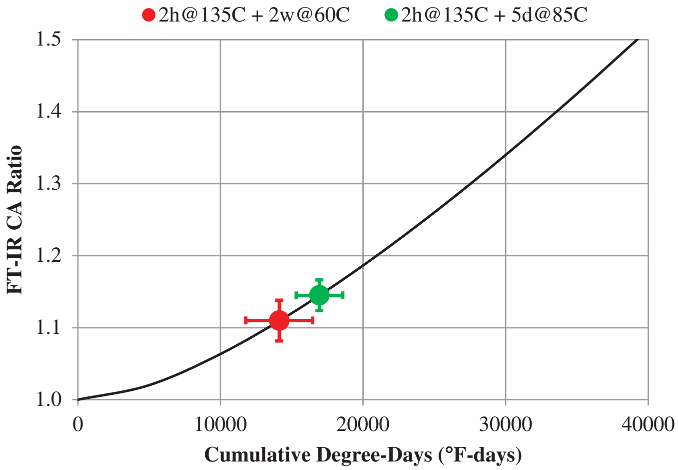


Figure 3-40. FT-IR carbonyl area ratio correlation between field aging and laboratory LTOA protocols.

in the field. Note that the correlation illustrated in Figure 3-40 was determined based on a limited amount of binder FT-IR CA results and therefore the cross points for LTOA protocols of 2 weeks at 140°F (60°C) and 5 days at 185°F (85°C) in Figure 3-40 would likely change if more binder results were included.

Based on the mixture M_R stiffness results, LTOA protocols of 3 days at 185°F (85°C) or 2 weeks at 140°F (60°C) and 5 days at 185°F (85°C) were representative of field aging of approximately 9,100 and 16,000 CDD, respectively. Using the information shown in Figure 3-29, the in-service time for each field site corresponding to 9,100 and 16,000 CDD was determined and is summarized in Table 3-3. As shown, the laboratory LTOA protocols of 3 days at 185°F (85°C) or 2 weeks at 140°F (60°C) were equivalent to approximately 7 months in service in warmer climates and 12 months in service in colder climates. As for the aging induced by the laboratory LTOA protocol of 5 days at 185°F (85°C), approximately 11 months and 22 months in service were required for warmer climates and colder climates, respectively.

Identification of Factors with Significant Effects on Mixture Aging Characteristics

This section presents the laboratory test results for identifying factors with significant effects on long-term mixture aging characteristics. These factors include WMA technology, production temperature, plant type, recycled material inclusion, and aggregate absorption. Detailed discussions for each factor are presented in the following subsections.

WMA Technology (HMA vs. WMA)

The M_R test and HWTT results for long-term aged mixtures including cores and LMLC specimens with LTOA protocols from the seven Phase II field sites are shown in Figures 3-41 and 3-42. The *x*-axis coordinate represents HMA test results and the *y*-axis coordinate represents the corresponding WMA test results. The solid line is the line of equality, and the dashed line illustrates the shift from the line of equality for the M_R ratio or HWTT RRP ratio.

The M_R ratio comparison for HMA versus WMA shown in Figure 3-41 illustrates that most of the data points aligned above the line of equality, indicating a greater increase in M_R stiffness after long-term aging for WMA compared to HMA. According to the shift from the line of equality, the average difference in M_R ratio for WMA versus HMA is approximately 11 percent greater. Although it exhibited more data scatter, a similar trend in the HWTT RRP ratio comparison for WMA versus HMA is observed in Figure 3-42, with most of the data points falling above the line of equality. This indicates that the increase in mixture rutting resistance in the HWTT after

Table 3-3. Correlation of field aging in terms of in-service time and laboratory LTOA protocols.

Field Site	Climate	M_R Ratio	
		2 weeks at 140°F (60°C) or 3 days at 185°F (85°C)	5 days at 185°F (85°C)
Texas I	Warmer Climate	6 months	10 months
New Mexico		8 months	12 months
Florida		6 months	11 months
Average		**7 months**	**11 months**
Wyoming	Colder Climate	12 months	22 months
South Dakota		12 months	22 months
Iowa		12 months	22 months*
Indiana		11 months	21 months*
Average		**12 months**	**22 months**

** Projected in-service time based on historical climatic information.*

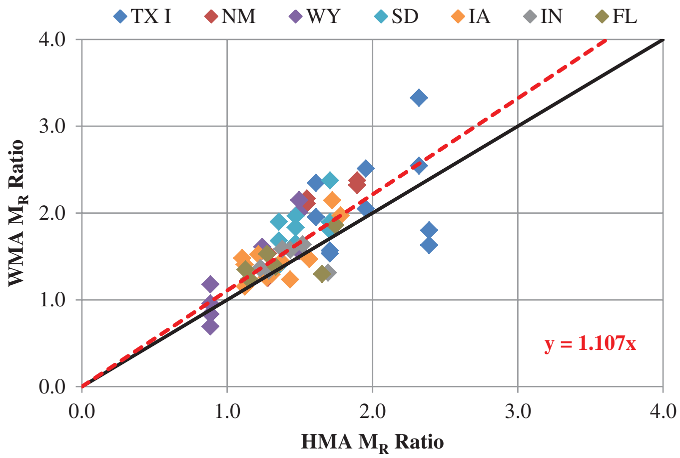


Figure 3-41. Resilient modulus ratio comparison for HMA versus WMA.

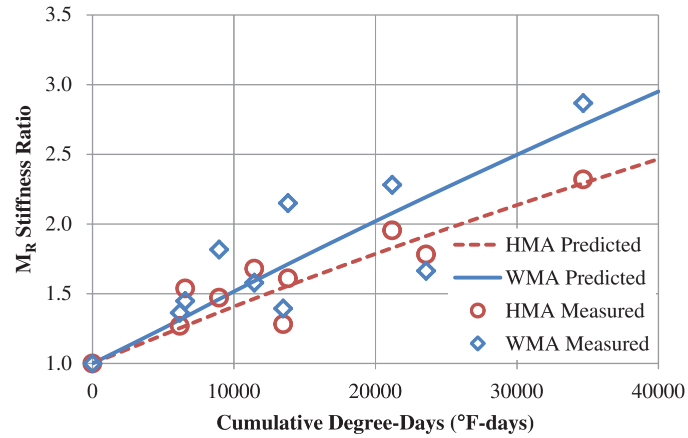


Figure 3-43. Resilient modulus ratio versus cumulative degree-days for HMA and WMA post-construction cores.

the long-term aging for WMA was more significant than that for HMA.

As the results of Phase I showed, in general, WMA mixtures begin their service lives with lower stiffness and tend to age in place more rapidly, as shown in Figures 3-41 and 3-42, but ultimately their stiffness and rutting resistance are comparable to HMA. The lower stiffness in WMA was possibly due to the use of WMA technologies as well as lower production and short-term aging temperatures. It is reasonable to hypothesize that there is a critical in-service time for WMA at which an equivalent mixture property as HMA is achieved.

To determine the critical in-service time when WMA equals HMA, CDD values for WMA and HMA post-construction cores and their associated M_R ratio values were fitted separately using Equation (2-6), as shown in Figure 3-43. In the figure, the data points represent the average HMA and WMA

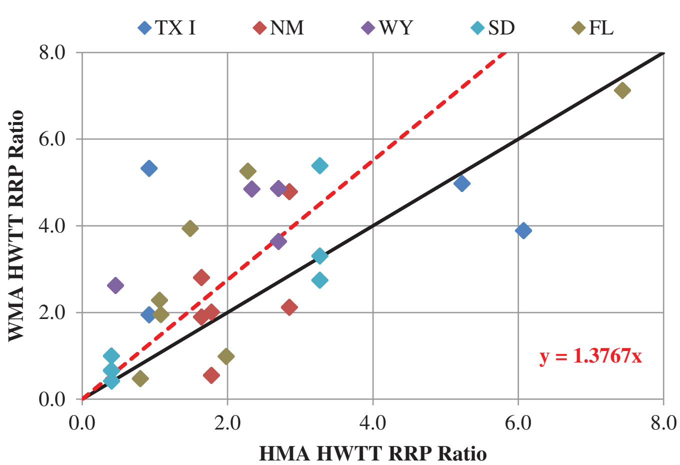


Figure 3-42. HWTT RRP ratio comparison for HMA versus WMA.

M_R ratio values for each field site, and the curves represent the exponential functions as expressed in Equation (2-6) for the M_R ratio versus CDD values. As illustrated, the WMA curve was above the HMA curve, verifying a greater increase in mixture stiffness after long-term aging for WMA versus HMA.

Based on the definition of the mixture property ratio, the M_R stiffness of WMA and HMA cores at a given in-service time or CDD value can be determined using Equations (3-1) and (3-2), respectively. The fitted exponential functions for HMA and WMA post-construction cores shown in Figure 3-43 are denoted as f_{HMA} and f_{WMA}.

$$M_{R\,WMA} = M_{R\,WMA0} * f_{WMA}$$

$$= M_{R\,WMA0} * \left\{1 + \alpha * \exp\left[-\left(\frac{\beta}{CDD}\right)^{\gamma}\right]\right\}_{WMA} \quad \text{Eq. (3-1)}$$

Where:

$M_{R\,WMA}$ = M_R stiffness of WMA post-construction cores at a given CDD value; and
$M_{R\,WMA0}$ = M_R stiffness of WMA construction cores.

$$M_{R\,HMA} = M_{R\,HMA0} * f_{HMA}$$

$$= M_{R\,HMA0} * \left\{1 + \alpha * \exp\left[-\left(\frac{\beta}{CDD}\right)^{\gamma}\right]\right\}_{HMA} \quad \text{Eq. (3-2)}$$

Where:

$M_{R\,HMA}$ = M_R stiffness of HMA post-construction cores at a given CDD value; and
$M_{R\,HMA0}$ = M_R stiffness of HMA construction cores.

By making Equations (3-1) and (3-2) equal, the critical CDD value for achieving equivalent M_R stiffness by WMA and HMA ($CDD_{WMA=HMA}$) can be determined, as expressed in Equation (3-3).

$$M_{R\,WMA0} * \left\{1+\alpha * \exp\left[-\left(\frac{\beta}{CDD}\right)^{\gamma}\right]\right\}_{WMA} = M_{R\,HMA0} * \left\{1+\alpha * \exp\left[-\left(\frac{\beta}{CDD}\right)^{\gamma}\right]\right\}_{HMA} \qquad \text{Eq. (3-3)}$$

In addition to $CDD_{WMA=HMA}$, the determination of the CDD value at which the M_R stiffness of WMA post-construction cores equaled that of HMA construction cores ($CDD_{WMA=HMA0}$) was also essential in order to understand the performance evolution of WMA in the field compared to HMA. $CDD_{WMA=HMA0}$ can be calculated in accordance with Equation (3-4).

$$M_{R\,WMA0} * \left\{1+\alpha * \exp\left[-\left(\frac{\beta}{CDD}\right)^{\gamma}\right]\right\}_{WMA} = M_{R\,HMA0} \qquad \text{Eq. (3-4)}$$

Depending on the difference in the magnitude of M_R stiffness for WMA and HMA construction cores, the mixture stiffness evolution of WMA and HMA with field aging can be categorized into three different scenarios, as shown in Figure 3-44. Scenario I in Figure 3-44(a) illustrates the case where the M_R stiffness of the HMA cores was always higher than their WMA counterparts, but the difference in stiffness between these two mixtures decreased with field aging. Scenario II in Figure 3-44(b) indicates the case where HMA had higher mixture stiffness compared to WMA at the initial aging stage (i.e., construction cores). It should be noted that beyond the catch-up point shown in Figure 3-44(b) is a projection from a regression beyond the limit of most of the data. Hence, it should not be concluded that WMA will age at a faster rate than HMA beyond this point. Scenario III represents the case where equivalent mixture stiffness was shown for HMA and WMA construction cores, but higher stiffness for post-construction cores was observed for WMA versus HMA, as shown in Figure 3-44(c).

Table 3-4 summarizes the $CDD_{WMA=HMA}$ and $CDD_{WMA=HMA0}$ values for each field site. For the majority of the field sites (four out of seven), the M_R stiffness evolution with field aging followed the trend illustrated in Scenario II, indicating that the stiffness of WMA was initially lower than that of HMA but it was able to catch-up to the stiffness of HMA after a certain amount of time in the field. The average $CDD_{WMA=HMA}$ and $CDD_{WMA=HMA0}$ values for those four field sites were approximately 23,000 and 3,000 CDD, respectively. Thus, field aging of approximately 3,000 CDD is necessary for the stiffness of WMA to equal the **initial** stiffness of HMA, and equivalent WMA and HMA mixture stiffness is likely to be achieved after 23,000 CDD of field aging.

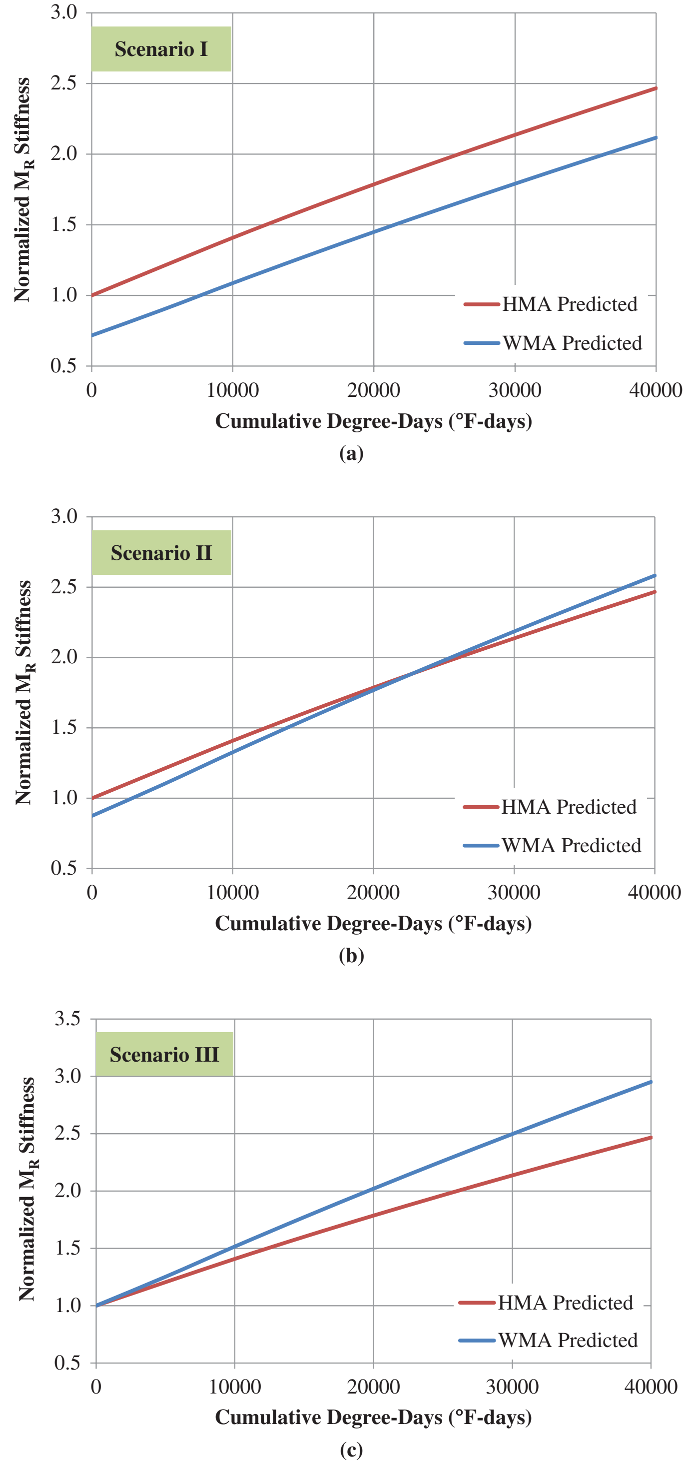


Figure 3-44. Evolution of mixture resilient modulus stiffness with field aging for HMA versus WMA.

Referring to Figure 3-29, the in-service time for each field site corresponding to 23,000 and 3,000 CDD was determined and is summarized in Table 3-5. As shown, approximately 17 months in service in warmer climates and 30 months in service in colder climates were needed in order to achieve equivalent mixture stiffness for WMA versus HMA. As for the

Table 3-4. $CDD_{WMA=HMA}$ and $CDD_{WMA=HMA0}$ values for each field site.

Field Site	Scenario	WMA_0/HMA_0	CDD Values	
			WMA = HMA	WMA = HMA_0
Texas I	I	0.717	N/A	7,672
New Mexico	II	0.876	22,421	2,929
Wyoming	II	0.867	25,486	3,152
South Dakota	II	0.897	16,543	2,426
Iowa	II	0.860	28,156	3,328
Indiana	III	1.002	0	0
Florida	III	0.999	0	0

in-service time corresponding to $CDD_{WMA=HMA0}$ of 3,000 CDD, approximately 2 months and 3 months were required in warmer climates and colder climates, respectively.

For the statistical analysis (see Appendix E), the analysis of variance (ANOVA) having WMA technology and aging level as main effects along with field site as a random effect was fitted to the data. The results showed that the effects of WMA technology and aging level were statistically significant at $\alpha = 0.05$. Also, WMA mixtures showed a higher predicted M_R ratio value than HMA mixtures. As expected, the predicted M_R ratio seemed to increase as aging level increased.

Production Temperature (High vs. Control)

The M_R test results for long-term aged mixtures including cores after certain in-service times and LMLC specimens with STOA protocol of 2 hours at 275°F (135°C) plus LTOA protocols from the Wyoming and Iowa field sites are shown in Figure 3-45, with the M_R ratio for mixtures produced at high temperatures and those at control temperatures plotted against each other. The evaluation of mixture rutting resistance evolution with long-term aging on the basis of the HWTT RRP ratio was not available for this factor since early stripping during the HWTT was observed for the majority of Wyoming and Iowa mixtures, with LC_{SN} values less than 3,000 load cycles, most likely due to the high test temperature and the softer binders from these sites. The *x*-axis coordinate represents the test results for control temperature mixtures, and the *y*-axis coordinate represents the results for mixtures at high temperature. The solid line is the line of equality, and the dashed line illustrates the shift from the line of equality for the M_R ratio.

The M_R ratio results shown in Figure 3-45 illustrate that most of the data points aligned along the line of equality, indicating an equivalent increase in M_R stiffness induced by long-term aging of mixtures produced at high versus control temperatures. Therefore, production temperature differences for these two sites had no significant effect on the sensitivity of mixture stiffness to long-term aging.

For the statistical analysis, an ANOVA model including production temperature, WMA technology, and aging level as fixed effects and field site as a random effect was fitted to the data. Results showed that the effect of the factor of interest—production temperature—was not statistically significant at $\alpha = 0.05$. Details of the analysis can be found in Appendix E.

Plant Type (BMP vs. DMP)

The MR test results for long-term aged mixtures including cores after 10 months in service and LMLC specimens with

Table 3-5. Field in-service time corresponding to $CDD_{WMA=HMA}$ and $CDD_{WMA=HMA0}$ values for each field site.

Field Site	Climate	CDD Values	
		WMA = HMA	WMA = HMA_0
Texas I	Warmer Climate	16 months	2 months
New Mexico		19 months	3 months
Florida		15 months	1 months
Average		**17 months**	**2 months**
Wyoming	Colder Climate	32 months*	2 months
South Dakota		32 months*	7 months
Iowa		28 months*	2 months
Indiana		26 months*	2 months
Average		**30 months**	**3 months**

* *Projected in-service time based on historical climatic information.*

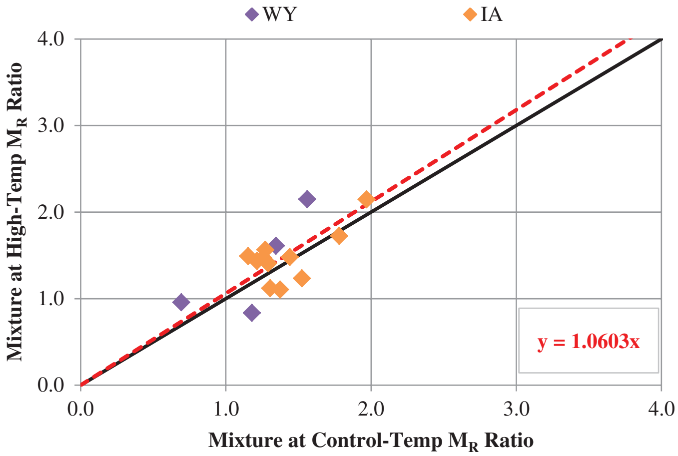


Figure 3-45. Resilient modulus ratio comparison for mixtures produced at high versus control temperature.

LTOA protocols of 5 days at 185°F (85°C) and 2 weeks at 140°F (60°C) from the Indiana field site are shown in Figure 3-46, with the M_R ratio for BMP-produced mixtures and DMP-produced mixtures plotted against each other. The evaluation of mixture rutting resistance evolution with long-term aging on the basis of the HWTT RRP ratio was not available for this factor since early stripping was observed for the majority of Indiana mixtures, with LC_{SN} values less than 3,000 load cycles. The *x*-axis coordinate represents the test results for BMP-produced mixtures, and the *y*-axis coordinate represents corresponding results for DMP-produced mixtures. The solid line is the line of equality, and the dashed line illustrates the shift from the line of equality for the M_R ratio.

The M_R ratio results shown in Figure 3-46 illustrate that most of the data points align along the line of equality, indicating an equivalent increase in M_R stiffness induced by long-term aging for the BMP- and DMP-produced mixtures. Therefore, plant type had no significant effect on the sensitivity of mixture stiffness to long-term aging.

For the statistical analysis, the ANOVA model including plant type, aging level, and WMA technology as main effects (since all possible two-way interactions were statistically insignificant) showed that none of the factor effects (as well as the factor of interest—plant type) was statistically significant at $\alpha = 0.05$. Additional details can be found in Appendix E.

Inclusion of Recycled Material (RAP/RAS vs. No RAP/RAS)

The M_R test results for long-term aged mixtures including cores after certain in-service times and LMLC specimens with LTOA protocols of 5 days at 185°F (85°C) and 2 weeks at 140°F (60°C) from the Texas I and New Mexico field sites are shown in Figure 3-47, with the M_R ratio for control mixtures without recycled materials and RAP/RAS mixtures plotted against each other. The evaluation of mixture rutting resistance evolution with long-term aging on the basis of the HWTT RRP ratio was not available for this factor since early stripping was observed for the majority of Texas I and New Mexico mixtures, with LC_{SN} values less than 3,000 load cycles. The control mixtures from the Texas I field site were produced using a PG 70-22 binder, while the RAP/RAS mixtures were produced using a softer PG 64-22 binder in conjunction with 15 percent RAP and 3 percent RAS; the control mixtures from the New Mexico site were produced using a PG 76-28 binder, while the RAP mixtures were produced using a softer PG 64-28 binder in conjunction with 35 percent RAP. The *x*-axis coordinate represents the test results for the control mixtures, and the *y*-axis coordinate represents corresponding results for RAP/RAS mixtures. The solid line is the line of equality, and the dashed line illustrates the shift from the line of equality for the M_R ratio.

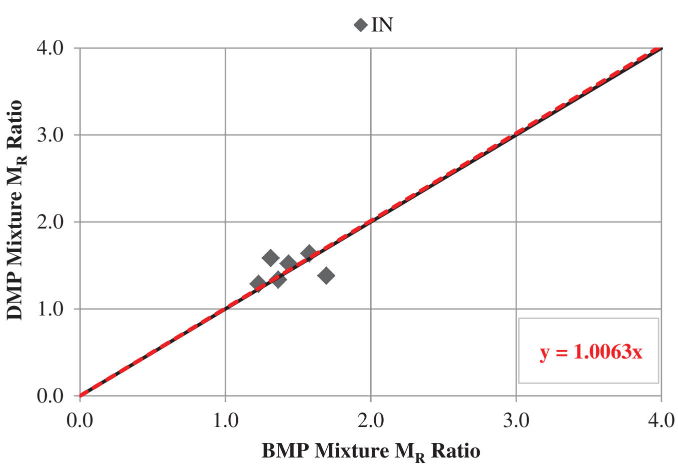


Figure 3-46. Resilient modulus ratio comparison for mixtures produced at BMP versus DMP.

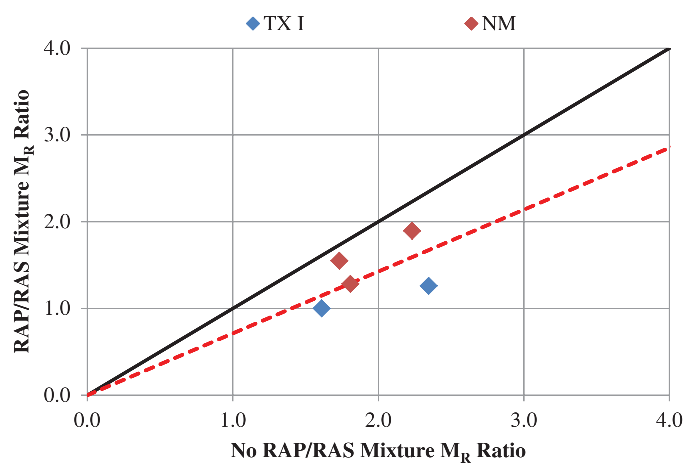


Figure 3-47. Resilient modulus ratio comparison for mixtures produced with RAP/RAS versus no RAP/RAS.

The M_R ratio results shown in Figure 3-47 illustrate that the data points align below the line of equality, indicating a significantly higher increase in M_R stiffness after long-term aging for the control mixtures compared to the RAP/RAS mixtures. The greater sensitivity to aging exhibited by the control mixtures can possibly be attributed to the larger amount of virgin binder in the mixture, which is likely more susceptible to aging. Therefore, the inclusion of recycled materials had a significant effect on mixture aging characteristics in this study.

For the statistical analysis (see Appendix E), the ANOVA model included recycled materials, aging level, and WMA technology as main effects (since all possible two-way interactions were statistically insignificant at $\alpha = 0.05$). Because field site was confounded with aging level for this dataset, field site could not be included as a random effect in the ANOVA model. The results showed that the effects of recycled materials, aging level, and WMA technology were all statistically significant at $\alpha = 0.05$. The conclusion of the statistical analysis was that mixtures with no RAP/RAS had a higher M_R ratio compared to mixtures with RAP/RAS.

Aggregate Absorption (High- vs. Low-Absorptive Aggregate)

The M_R test and HWTT results for long-term aged mixtures including cores after certain in-service times and LMLC specimens with LTOA protocols of 5 days at 185°F (85°C) and 2 weeks at 140°F (60°C) from the Iowa and Florida field sites are shown in Figures 3-48 and 3-49; the M_R ratio and HWTT RRP ratio for mixtures using high-absorptive aggregates versus low-absorptive aggregates are plotted against each other. The evaluation of rutting resistance evolution with long-term aging on the basis of the HWTT RRP ratio was not available for Iowa mixtures since early stripping was observed, with

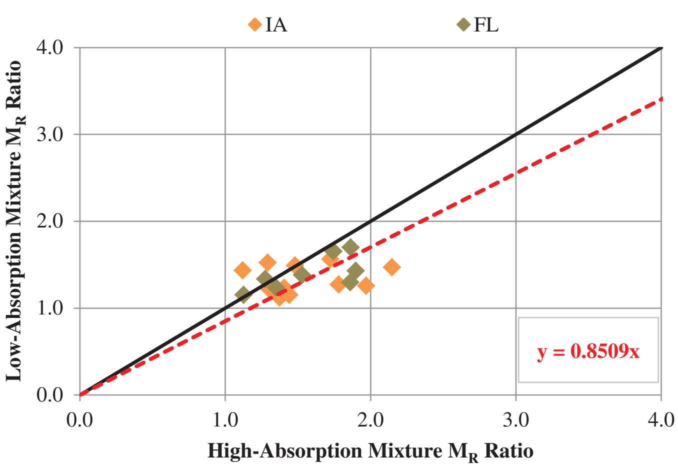


Figure 3-48. Resilient modulus ratio comparison for mixtures produced using high- versus low-absorptive aggregates.

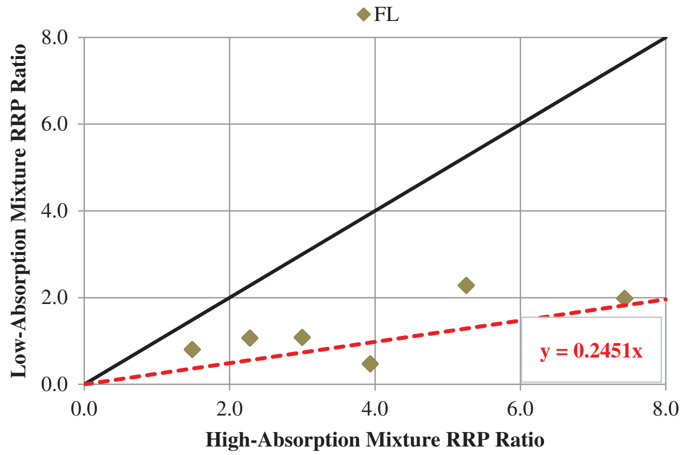


Figure 3-49. HWTT RRP ratio comparison for mixtures produced using high- versus low-absorptive aggregates.

LC_{SN} values less than 3,000 load cycles. The *x*-axis coordinate represents test results for mixtures using high-absorptive aggregates, and the *y*-axis coordinate represents corresponding test results for mixtures using low-absorptive aggregates. The solid line represents the line of equality, and the dashed line illustrates the shift from the line of equality for the M_R ratio or HWTT RRP ratio.

The M_R ratio comparison for mixtures using high-absorptive versus low-absorptive aggregates shown in Figure 3-48 illustrates that most of the data points align below the line of equality, indicating a greater increase in M_R stiffness induced by long-term aging for mixtures using high-absorptive aggregates compared to the mixtures using low-absorptive aggregates. A similar trend for the HWTT RRP ratio is shown in Figure 3-49, indicating that mixtures using high-absorptive aggregates exhibited a greater increase in rutting resistance than those using low-absorptive aggregates. The greater sensitivity of mixture stiffness and rutting resistance to aging for mixtures using high-absorptive aggregates was likely due to the higher volume of effective binder in these mixtures that were available for aging (as indicated by higher P_{be} values in Table 3-1), the continuous asphalt absorption by the aggregates with time, or both (West et al. 2014). Therefore, aggregate absorption and, more specifically, the amount of effective binder had a significant effect on mixture aging characteristics in this study.

For the statistical analysis, an ANOVA model including aggregate absorption, aging level, and WMA technology as main effects, aging level*aggregate absorption and WMA technology*aggregate absorption as two-way interaction effects (WMA technology*aging level interaction was not statistically significant), and field site as a random effect was fitted to the data. The results showed that the effects of aggregate absorption, aging level, aging level*aggregate absorption, and WMA

Table 3-6. Summary of the effects of mixture components and production parameters on mixture aging characteristics.

<table>
<tr><th rowspan="2">Factor</th><th colspan="3">M_R Ratio</th><th colspan="2">HWTT RRP Ratio</th></tr>
<tr><th>Slope Magnitude</th><th>Significant Effect</th><th>Statistically Significant</th><th>Slope Magnitude</th><th>Significant Effect</th></tr>
<tr><td>WMA Technology</td><td>1.107</td><td>Yes</td><td>Yes</td><td>1.377</td><td>Yes</td></tr>
<tr><td>Production Temperature</td><td>1.060</td><td>No</td><td>No</td><td colspan="2" rowspan="3">N/A</td></tr>
<tr><td>Plant Type</td><td>1.006</td><td>No</td><td>No</td></tr>
<tr><td>Inclusion of Recycled Materials</td><td>0.713</td><td>Yes</td><td>Yes</td></tr>
<tr><td>Aggregate Absorption</td><td>0.851</td><td>Yes</td><td>Yes</td><td>0.245</td><td>Yes</td></tr>
</table>

technology*aggregate absorption were statistically significant at $\alpha = 0.05$.

The difference between mixtures using high-absorptive and low-absorptive aggregates was statistically significant for WMA but not for HMA. Also, although mixtures with high-absorptive aggregates had a higher M_R ratio for each level of aging, in general (except for Field Aging 1_IA [10-month field aging for the Iowa field project]), the difference between mixtures with high-absorptive and low-absorptive aggregates was statistically significant only for laboratory LTOA 2 (i.e., 5 days at 185°F [85°C]). Further details of the statistical analysis can be found in Appendix E.

Summary

In this subsection, the effects of various mixture components and production parameters including WMA technology, production temperature, plant type, recycled material inclusion, and aggregate absorption on the mixture aging characteristics were evaluated based on the change in mixture stiffness and rutting resistance after long-term aging. The correlations in terms of M_R ratio and HWTT RRP ratio were performed for each factor, and the results are summarized in Table 3-6. Factors with a significant effect on mixture aging characteristics were identified based on the magnitude of the slope of the shifted line with respect to the line of equality being greater than 1.05 or smaller than 0.95 (i.e., 5 percent off the line of equality) and were corroborated via statistical analysis. According to Table 3-6, WMA technology, recycled material inclusion, and aggregate absorption showed significant effects on mixture aging characteristics, while no significant effects from production temperature and plant type were observed. For aggregate absorption, the results from Phase I indicated that mixtures with high-absorptive aggregates had a reduced M_R stiffness and rutting resistance compared to those with low-absorptive aggregates, likely due to the higher binder content and thus thicker FT_{be} used in the mixtures with high-absorptive aggregates. However, after long-term aging, a greater increase in M_R and rutting resistance was observed for mixtures using high-absorptive aggregates.

CHAPTER 4

Conclusions and Suggested Research

Asphalt mixtures have been traditionally designed on the basis of volumetric parameters in which the optimum asphalt content for a given aggregate gradation was dependent upon the compaction effort, AV content, voids in mineral aggregate, and asphalt absorption into aggregate voids. These volumetric procedures worked well for agencies and contractors in an era when the components for asphalt mixtures were relatively constant. However, in the last three decades, changes in asphalt mixture components, production parameters, and plant design have occurred and, consequently, raised questions of the validity of the current mix design procedures in adequately assessing the volumetric needs of asphalt mixtures and the physical characteristics required to meet performance expectations. This project investigated the need for mix design procedures to consider the impact of these recent changes including binder source, aggregate absorption, WMA technology, recycled material inclusion, plant type, and production temperature on the volumetric and performance characteristics of asphalt mixtures during production and construction.

Phase I Experiment

The objectives of Phase I of this project were to (1) develop a laboratory STOA protocol for asphalt loose mix prior to compaction to simulate the aging and asphalt absorption by the aggregate as it is produced in a plant and then loaded into a truck for transport, and (2) identify factors with significant effects on the performance-related properties of short-term aged asphalt mixtures. Laboratory STOA protocols of 2 hours at 275°F (135°C) for HMA and 2 hours at 240°F (116°C) for WMA, as previously recommended in *NCHRP Report 763* (Epps Martin 2014), were used to fabricate LMLC specimens for volumetric analysis and performance testing. The simulations of asphalt aging and absorption during plant production and construction by the selected laboratory STOA protocols were evaluated by comparing the binder or mixture performance of LMLC specimens to the corresponding PMPC specimens and cores at construction. Additionally, the laboratory test results were used to identify mixture components and production parameters with significant effects on the performance of short-term aged asphalt mixtures. A guide for conducting experiments to assess the short-term aging of asphalt mixtures produced in the field is presented in Appendix F based upon the techniques developed in this research. Recommended changes to the current AASHTO R 30 short-term aging protocol are given in Appendix G, which shows the modifications in "track changes" format. The basic changes resulting from this project include (1) fixing the compaction temperatures for WMA at 240°F (116°C) and HMA at 275°F (135°C) and (2) conditioning the sample for 2 hours at the compaction temperature regardless of whether the sample is being prepared for volumetric mix design or performance testing.

The following conclusions pertain to Phase I of this project, in which 522 LMLC specimens, PMPC specimens, and cores at construction from nine field sites were evaluated.

Simulation of Plant Aging

- The correlation between LMLC and PMPC specimens in terms of volumetric parameters (i.e., theoretical maximum specific gravity and percentage of absorbed binder) indicated that the selected STOA protocols for LMLC specimens were able, to a large extent, to simulate the asphalt absorption that took place during production at the plant.
- The correlations between LMLC and PMPC specimens and cores at construction in terms of resilient modulus stiffness and dynamic modulus stiffness for a wide range of mixtures indicated that the laboratory STOA protocols used for fabricating LMLC specimens were able to simulate plant aging.
- The correlations between LMLC and PMPC specimens for the HWTT resistance parameters (i.e., RRP and rut depth at

5,000 load cycles) also provided evidence that the laboratory STOA protocols produce representative specimens for performance testing. HWTT results from construction cores did not correlate well with those for LMLC specimens, possibly due to testing difficulties of thin lifts and the required use of plaster to fit the cores into the HWTT molds.
- The correlation in terms of binder continuous performance grades and Fourier transform infrared spectroscopy carbonyl area values between extracted and recovered binders from LMLC specimens versus PMPC specimens and cores at construction indicated that the selected laboratory STOA protocols produced equivalent binder stiffening and oxidation effects as the plant production and construction processes.
- Although it was not an experiment design factor, different binder grades and the use of polymers were included in the study. Across a wide spectrum of grades (PG 58-22 to 76-22), the LMLC M_R test results were consistently in line with the PMPC values. One of the most interesting observations concerning polymer modification can be seen in the New Mexico data where a virgin polymer-modified (PG 76-22) mixture had a lower stiffness than a high-RAP content mixture made with a straight PG 64-28 binder.

Factor Analysis

- Laboratory test results indicated a significant effect on the performance of short-term aged asphalt mixtures from *WMA technology.* Lower mixture stiffness and decreased rutting resistance were observed for WMA mixtures compared to HMA mixtures, possibly due to the reduced production temperature.
- Laboratory test results on the effect of *production temperature* and *plant type* indicated no significant change in stiffness and rutting resistance of short-term aged asphalt mixtures.
- Laboratory test results on the effect of *recycled materials* indicated a significant increase in mixture stiffness even with the use of a softer asphalt binder, although the effect was inconsistent due to the variability of recycled materials in terms of original asphalt mixtures and in-service times and climatic conditions.
- A significant effect on the performance of short-term aged asphalt mixtures was observed for *aggregate absorption.* Asphalt mixtures using low-absorptive aggregates exhibited higher stiffness and better rutting resistance than the mixtures using high-absorptive aggregates, which was attributed to the thicker effective film thickness in the high-absorptive aggregate mixtures from volumetric compensation in the mixture design process. High aggregate absorption in mixtures produced greater variability in results than other variables that were studied.
- *Binder source* had a significant effect on the performance of short-term aged asphalt mixtures. Different mixture performance in terms of stiffness and rutting resistance should be expected from asphalt mixtures using the same PG binders from different sources.

Phase II Experiment

The objectives of Phase II of this project were to (1) evaluate the evolution of performance-related properties of asphalt mixtures through their initial period of field performance, (2) develop a correlation between long-term field aging and laboratory LTOA protocols, and (3) identify mixture components and production parameters with significant effects on the intermediate- or long-term aging characteristics of asphalt mixtures. To simulate long-term aging in the laboratory, WMA and HMA LMLC specimens were subjected to a STOA protocol of 2 hours at 275°F (135°C) plus an LTOA protocol of either 5 days at 185°F (85°C) in accordance with AASHTO R 30 or 2 weeks at 140°F (60°C).

The following conclusions pertain to Phase II of this project, in which 512 LMLC specimens and cores acquired at construction and 8 to 22 months after construction from seven field sites were evaluated.

Quantification of Field Aging and Correlation to Laboratory LTOA Protocols

- Cumulative degree-days (32°F [0°C] base) was proposed as a metric to quantify field aging and was able to account for the differences in construction dates and climates for various field sites.
- Binder or mixture property ratios (i.e., M_R ratio, HWTT RRP ratio, DSR complex modulus ratio, and FT-IR CA ratio), defined as the relative differences of binder or mixture properties of short-term aged versus long-term aged specimens, were used to quantify the evolution of mixture stiffness and rutting resistance and binder oxidation with field aging.
- An exponential function was proposed to correlate the binder or mixture property ratios of post-construction cores versus their corresponding CDD values, and a desirable correlation resulted.
- Mixture stiffness results showed that the average M_R ratio values for all LMLC specimens with STOA protocol of 2 hours at 275°F (135°C) on loose mixture plus LTOA protocols of 2 weeks at 140°F (60°C) and 5 days at 185°F (85°C) on compacted specimens from the selected field sites were approximately 1.46 and 1.76, respectively. The higher M_R ratio values associated with the LTOA protocol of 5 days at 185°F (85°C) versus that of 2 weeks at 140°F (60°C) indicated that mixture aging was more sensitive to aging temperature than to aging time in the LTOA protocol.

- Based on the mixture stiffness results, the laboratory STOA protocol of 2 hours at 275°F (135°C) on loose mixture plus LTOA protocols of 2 weeks at 140°F (60°C) or 5 days at 185°F (85°C) on compacted specimens were able to produce mixture aging equivalent to an average of 9,100 and 16,000 CDD, respectively, in the field. This resulted in no change to AASHTO R 30 for the LTOA procedure (i.e., 5 days at 185°F [50°C]), and this is reflected in Appendix G.
- The pavement in-service time for each field site corresponding to 9,100 and 16,000 CDD was determined. The laboratory STOA protocol of 2 hours at 275°F (135°C) on loose mixture plus LTOA protocol of 2 weeks at 140°F (60°C) on compacted specimens was equivalent to approximately 7 months in service in warmer climates and 12 months in service in colder climates. As for the same laboratory STOA protocol on loose mixture plus LTOA protocol of 5 days at 185°F (85°C) on compacted specimens, approximately 11 months and 22 months in service were required for warmer climates and colder climates, respectively.

Factor Analysis

- The effect of *WMA technology* on mixture stiffness evolution with field aging compared to HMA was categorized into three different scenarios:
  - Scenario I: The stiffness of HMA cores was always higher than WMA cores, but the difference in stiffness between these two mixtures decreased with field aging.
  - Scenario II: HMA had higher mixture stiffness compared to WMA at the initial aging stage (i.e., construction cores), but the WMA stiffness eventually equaled that of HMA after some time in the field. In other words, there was a catch-up point in the stiffness values of HMA and WMA.
  - Scenario III: Equivalent mixture stiffness was shown for cores at construction between HMA and WMA, but higher stiffness for post-construction cores was observed for WMA versus HMA.

  For the majority of the field sites (four out of seven), the M_R stiffness evolution with field aging followed Scenario II, indicating that the stiffness of WMA was initially lower than that of HMA, but it was able to catch-up to the stiffness of HMA after a certain in-service time in the field. Ultimately, the stiffness and rutting resistance of WMA mixtures was comparable to HMA.
- The critical in-service time when WMA equaled HMA ($CDD_{WMA=HMA}$) was achieved after 23,000 CDD, which was equivalent to approximately 17 months in service in warmer climates and 30 months in service in colder climates. Field aging of approximately 3,000 CDD was necessary for the stiffness of WMA to equal the initial stiffness of HMA ($CDD_{WMA=HMA0}$), which was equivalent to approximately 2 months for warmer climates and 3 months for colder climates.
- Laboratory test results indicated no significant effect of *production temperature* and *plant type* on the stiffness and rutting resistance of long-term aged asphalt mixtures.
- Laboratory test results for the effect of *recycled materials* indicated a higher increase in M_R stiffness after long-term aging for the control mixtures compared to the RAP/RAS mixtures. The greater sensitivity to aging exhibited by the control mixtures was attributed to the larger amount of virgin binder in the mixture, which was likely more susceptible to aging.
- *Aggregate absorption*, specifically the effective binder content in the mix, also had a significant effect on the long-term aging characteristics of asphalt mixtures. A greater rate of stiffening as indicated by the increase in M_R stiffness induced by long-term aging for mixtures using high-absorptive aggregates was observed compared to the mixtures using low-absorptive aggregates. In addition, mixtures prepared with high-absorptive aggregates exhibited a greater increase in rutting resistance than those using low-absorptive aggregates. The greater sensitivity of mixture stiffness and rutting resistance to aging for mixtures using high-absorptive aggregates was likely due to the higher volume of effective binder in these mixtures that was available for aging, and the continuous asphalt absorption by the aggregates with time.

Suggested Research

The following recommendations are made based on the results of this study:

- There is a need to further monitor the long-term behavior of the field mixtures from this study to more accurately model their aging. It is recommended that cores be obtained from as many of these field sites as possible for at least one more point in time. The testing could be restricted to M_R at 77°F (25°C).
- While this research focused on characterizing the aging characteristics of asphalt mixtures over a wide range of factors, only mixture stiffness and rutting resistance were utilized in this study to discriminate asphalt mixtures with different short-term aging characteristics and testing was conducted at one set of temperatures for each test for most of the data. Additional mixture properties such as moisture susceptibility, fatigue cracking resistance, and thermal cracking resistance need to be considered for further validation and to examine how aging affects the long-term cold-temperature behavior of asphalt mixtures in order to quantify possible embrittlement with time.

- Differences in volumetric properties were observed for mixtures using highly absorptive aggregates when comparing LMLC specimens with the laboratory STOA protocols of 2 hours at 275°F (135°C) for HMA and 2 hours at 240°F (116°C) for WMA and the corresponding PMPC specimens. Therefore, the proposed STOA may not be fully applicable to those mixtures, and additional STOA protocols should be explored in future research for achieving equivalent volumetric properties.
- Based on the limited amount of M_R stiffness results measured for Indiana and Florida LMLC specimens with different LTOA protocols, 3 days at 185°F (85°C) and 2 weeks at 140°F (60°C) produced an equivalent level of mixture aging. This 3-day LTOA protocol might be more practical for simulating the field aging in colder climates. Therefore, there is a need to further explore the LTOA protocol of 3 days at 185°F (85°C) based on additional mixture results.
- Early stripping in the HWTT was observed for a substantial portion of short-term aged asphalt mixtures in this study. As a consequence, the rut depth measurements for these mixtures were possibly biased from the stripping of the binder from aggregates. Therefore, future research on a more accurate rutting evaluation for softer asphalt mixtures by performing the HWTT at a lower temperature is necessary. Additionally, there is a need for appropriate modifications to the HWTT procedure for testing field cores, such as finding a firmer substrate than plaster.
- There needs to be a research effort to define a balanced mix design procedure incorporating performance testing. It is recommended that the STOA defined herein should be used for rutting and moisture susceptibility performance testing as this would provide the most severe conditions for those distresses and STOA and LTOA from this project should be used in conditioning asphalt mixtures for cracking performance testing.

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Abbreviations, Acronyms, and Symbols

AASHTO	American Association of State Highway and Transportation Officials
AC	Absorption Capacity
AMPT	Asphalt Mixture Performance Tester
AMRL	AASHTO Materials Reference Laboratory
ANCOVA	Analysis of Covariance
ANOVA	Analysis of Variance
ASTM	American Society for Testing and Materials
AV	Air Void
BBR	Bending Beam Rheometer
BMP	Batch Mix Plant
CA	Carbonyl Area
CDD	Cumulative Degree-Day
CFD	Counter-Flow Drum
DMD	Dryer Mixing Drum
DMP	Drum Mix Plant
DOT	Department of Transportation
DSR	Dynamic Shear Rheometer
E^*	Dynamic Modulus
FAD	Forced-Air Distillation
FHWA	Federal Highway Administration
FT	Film Thickness
FT_{be}	Effective Binder Film Thickness
FT-IR	Fourier Transform Infrared Spectroscopy
G^*	Complex Modulus
G_{mm}	Theoretical Maximum Specific Gravity
HMA	Hot Mix Asphalt
HSD	Honest Significant Difference
HWTT	Hamburg Wheel-Tracking Test
IADOT	Iowa Department of Transportation
IDT	Indirect Tensile
ISU	Iowa State University
LMLC	Laboratory-Mixed, Laboratory-Compacted
LC_{SN}	Load Cycle Stripping Number
LTA	Long-Term Aged
LTOA	Long-Term Oven Aging
LVDT	Linear Variable Differential Transducers
M_R	Resilient Modulus

MTD	Material Transfer Device
NCAT	National Center for Asphalt Technology
NCHRP	National Cooperative Highway Research Program
NMAS	Nominal Maximum Aggregate Size
NMDOT	New Mexico Department of Transportation
ODOT	Oregon Department of Transportation
PAV	Pressure Aging Vessel
P_{ba}	Percentage of Absorbed Binder
P_{be}	Percentage of Effective Binder
PFD	Parallel-Flow Drum
PG	Performance Grading
PMFC	Plant-Mixed, Field-Compacted
PMPC	Plant-Mixed, Plant-Compacted
QC	Quality Control
RAP	Reclaimed Asphalt Pavement
RAS	Recycled Asphalt Shingle
RFAD	Rolling Forced-Air Distillation
RTFO	Rolling Thin-Film Oven
RRP	Rutting Resistance Parameter
SBS	Styrene-Butadiene-Styrene
SDDOT	South Dakota Department of Transportation
SGC	Superpave Gyratory Compactor
SN	Stripping Number
SSD	Small Steam Distillation
STA	Short-Term Aged
STOA	Short-Term Oven Aging
T_c	Compaction Temperature
T_m	Mixing Temperature
TTI	Texas A&M Transportation Institute
TxDOT	Texas Department of Transportation
UC Davis	University of California—Davis
UCPRC	University of California Pavement Research Center
UDM	Unitized Drum Mixer
VMA	Void in Mineral Aggregates
WMA	Warm Mix Asphalt
WYDOT	Wyoming Department of Transportation
δ	Phase Angle

APPENDIX A

Construction Reports

Table A-1 provides a summary of the field sites used in this project. Besides climate (wet–freeze, dry–freeze, wet–no freeze, and dry–no freeze), the following factors were considered when selecting field sites that included a wide spectrum of materials and production parameters:

- Aggregate type (high AC and low AC)
- Asphalt source (fast aging and slow aging)
- Recycled materials (RAP and RAS)
- WMA technology (WMA and HMA)
- Plant type (batch plant and drum plant)
- Production temperature (high and low)

Texas Farm-to-Market Highway 973 (TX I FM 973)

General Description

The Texas Department of Transportation (TxDOT) set up an experimental overlay on FM 973 in Travis County, in the Austin District, in order to conduct testing and long-term performance monitoring for several research projects. This experimental construction project (Project ID STP 1102 [371]) was planned to explore the different aspects of WMA, as well as the effect of RAP and RAS on the performance of HMA and WMA mixtures. Researchers involved in various state and federal studies actively participated in testing and monitoring these test sections. The overlay construction started on December 1, 2011, and took 1.5 months to complete due to inclement weather and holidays. J. D. Ramming Paving Company was the general contractor for this project.

The project site was located just north of the Austin Bergstrom International Airport (see Figure A-1). The length of the project was approximately 2.9 mi. Within the project limits, there was an aggregate quarry and a concrete plant that generated very high-volume truck traffic.

Nine test sections were laid out as shown in Figure A-2. This portion of FM 973 experiences moderate- to high-volume traffic; 2011 traffic data reported 11,000 and 11,300 annual average daily traffic for the north and south ends, respectively. Truck traffic was reported to be from 4.2 to 4.3 percent.

Mixtures and Materials

A TxDOT Type C (0.5-in. [12.5-mm] NMAS) surface mixture was used in the project. The aggregate structure was the same for all mixtures used in the various test sections. Figure A-3 presents the mix design used in Section 1.

The differences between test sections are listed in Table A-2, namely the type of mixture (i.e., HMA vs. WMA), the type of asphalt binder (i.e., PG 70-22, PG 64-22, and PG 58-28), and the amount of RAP/RAS added to the mixture (i.e., 0/15/30 percent RAP and 0/3/5 percent RAS). Binder contents of the mixtures were verified during quality control through ignition oven analysis. The asphalt binders classified as PG 70-22 (SBS-modified binder) and PG 58-28 were supplied by Binder V Asphalt Company from its Corpus Christi, Texas, refinery. Pelican Refining Company supplied the PG 64-22 (unmodified) asphalt binder from its Channelview, Texas, facility. All mixtures used virgin limestone from Cemex Aggregate located just across from the asphalt mixture plant. RAP and RAS came from various sources.

Table A-1. Summary of field sites.

Location & Climate	Date	Plant Type	$T_{Production}$	Asphalt	Aggregate & Additives	Mixtures	Factors
Texas I FM 973 Wet–No Freeze	1/12	CFD	325°F (163°C) HMA 325°F (163°C) HMA + RAP/RAS 275°F (135°C) Foaming 275°F (135°C) Evotherm® 270°F (132°C) Evotherm + RAP/RAS	5.2% PG 70-22 (Styrene-Butadiene-Styrene [SBS]) PG 64-22 w/ 15%RAP/ 3%RAS	Limestone	HMA HMA + 15%RAP + 3%RAS WMA Foaming WMA Evotherm DAT WMA Evotherm + 15%RAP + 3%RAS	HMA vs. WMA RAP/RAS vs. No RAP
New Mexico IH 25 Dry–No Freeze	10/12	CFD	345°F (174°C) HMA 315°F (157°C) HMA + RAP 285°F (141°C) Foaming 275°F (135°C) Evotherm	5.4% PG 76-28 (SBS) PG 64-28 w/35%RAP	Siliceous Gravel 1% Versabind	HMA HMA + 35%RAP WMA Foaming + 35%RAP WMA Evotherm 3G + 35%RAP	HMA vs. WMA RAP vs. No RAP
Connecticut IH 84 Wet–Freeze	8/12	CFD	322°F (161°C) HMA 312°F (156°C) Foaming	5.0% PG 76-22 (SBS) w/20%RAP	Basalt	HMA + 20%RAP WMA Foaming + 20%RAP	HMA vs. WMA
Wyoming SR 196 Dry–Freeze	8/12	CFD	315°F (157°C) HMA 255 & 275°F (124 & 135°C) Evotherm 275 & 295°F (135 & 146°C) Foaming	5.0% PG 64-28 (polymer)	Limestone 1% Lime	HMA WMA Foaming WMA Evotherm 3G	HMA vs. WMA Production Temperature (WMA)
South Dakota SH 262 Dry–Freeze	10/12	CFD	310°F (154°C) HMA 275°F (135°C) Foaming 270°F (132°C) Evotherm 280°F (138°C) Advera®	5.3% PG 58-34 (SBS) w/20%RAP	Quartzite 1% Lime	HMA + 20%RAP WMA Foaming + 20%RAP WMA Evotherm 3G + 20%RAP WMA Advera + 20%RAP	HMA vs. WMA
Iowa Fairgrounds Wet–Freeze	6/13	CFD	295 & 325°F (146 & 163°C) Low Abs HMA 295 & 310°F (146 & 154°C) High Abs HMA 265 & 295°F (129 & 146°C) Low Abs Foaming 260 & 290°F (127 & 143°C) High Abs Foaming	5.0% (0.9%AC Limestone) and 7.0% (3.2% AC Limestone) PG 58-28 w/ 20%RAP	0.9%AC Limestone + Field Sand 3.2%AC Limestone + Field Sand	Low Abs HMA + 20%RAP High Abs HMA + 20%RAP Low Abs WMA Foaming + 20%RAP High Abs WMA Foaming + 20%RAP	HMA vs. WMA Production Temperature Aggregate Absorption
Florida Parking Wet–No Freeze	8/13	CFD	306°F (152°C) Low Abs HMA 308°F (153°C) High Abs HMA 272°F (133°C) Low Abs Foaming 267°F (131°C) High Abs Foaming	5.1% (0.6%AC Granite) and 6.8% (3.7%AC Limestone) PG 58-28 w/ 25%RAP	0.6%AC Granite 3.7%AC Limestone 0.5% Liquid Anti-strip	Low Abs HMA + 25%RAP High Abs HMA + 25%RAP Low Abs WMA Foaming + 25%RAP High Abs WMA Foaming + 25%RAP	HMA vs. WMA Aggregate Absorption
Indiana Residential Wet–Freeze	8/13	CFD BMP	300°F (149°C) HMA + RAP (CFD) 305°F (152°C) HMA + RAP (BMP) 271°F (133°C) Foaming + RAP (CFD) 273°F (134°C) Advera + RAP (BMP)	5.8% PG 64-22	Limestone	HMA + 25%RAP (CFD) HMA + 25%RAP (BMP) WMA Foaming + 25%RAP (CFD) WMA Advera + 25%RAP (BMP)	HMA vs. WMA Plant Type
Texas II Local Dry–No Freeze	4/14	CFD BMP	310°F (154°C) HMA (Binder V, CFD) 315°F (157°C) HMA (Binder V, BMP) 310°F (154°C) HMA (Binder A, CFD) 315°F (157°C) HMA (Binder A, BMP)	6.2% PG 64-22	Limestone	HMA (Binder V, CFD) @ 5.9%AC HMA (Binder V, BMP) @ 6.4%AC HMA (Binder A, CFD) @ 5.9%AC HMA (Binder A, BMP) @ 6.4%AC	Plant Type Asphalt Source

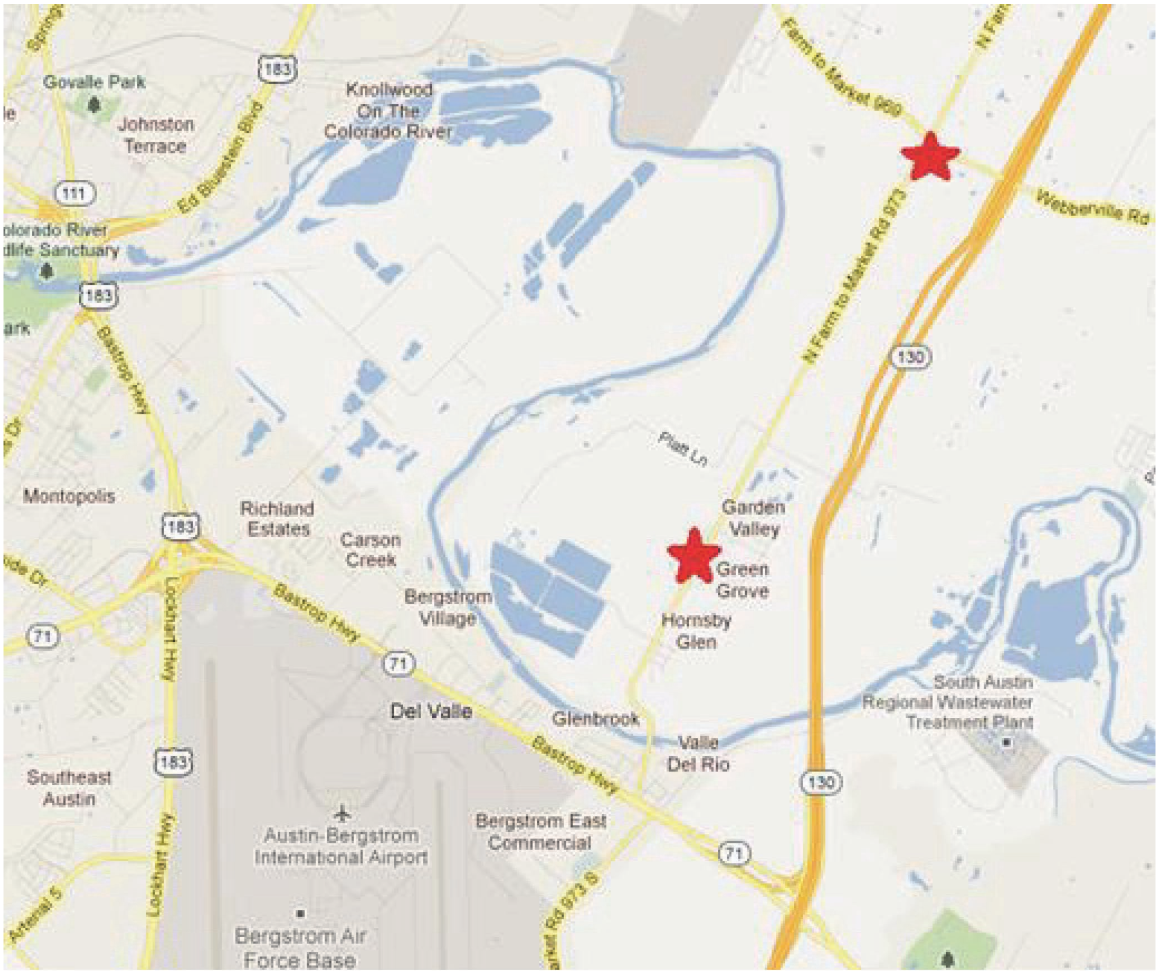

Figure A-1. Project limits for the Texas field site.

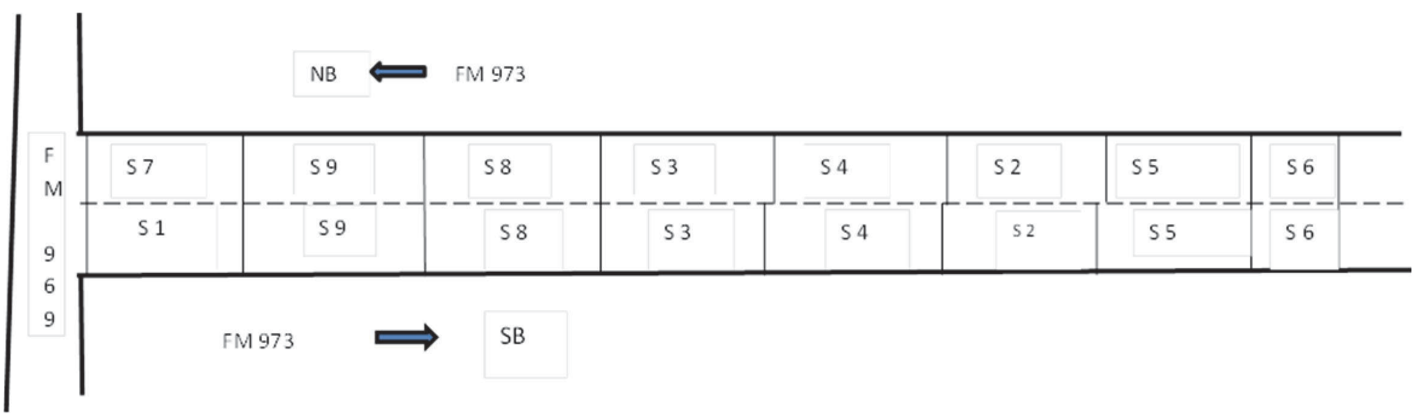

Figure A-2. Schematic layout diagram of the Texas field site test sections (not to scale).

TEXAS DEPARTMENT OF TRANSPORTATION

HMACP MIXTURE DESIGN : COMBINED GRADATION

Refresh Workbook

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SAMPLE ID:		SAMPLE DATE:	
LOT NUMBER:		LETTING DATE:	
SAMPLE STATUS:		CONTROLLING CSJ:	
COUNTY:	Jimmy Whited #652	SPEC YEAR:	2004
SAMPLED BY:		SPEC ITEM:	
SAMPLE LOCATION:		SPECIAL PROVISION:	
MATERIAL CODE:	Type C	MIX TYPE:	SS3224_C_Coarse_Surface
MATERIAL NAME:	70SC1000 #1	WMA Included in Design?	No
PRODUCER:	RTI Hot Mix, Ltd.	WMA TECHNOLOGY:	
AREA ENGINEER:		PROJECT MANAGER:	WMA RATE: UNITS:
COURSE\LIFT: Surface	STATION:	DIST. FROM CL:	CONTRACTOR DESIGN #: 70SC1000 #1

Recycled Binder, %	
Bin No.8 :	0.0
Bin No.9 :	0.0
Bin No.10 :	0.0
Total	***0.0***

Use this value in the QC/QA template>>

Ratio of Recycled to Total Binder, % *(based on binder percent (%) entered below in this worksheet)*
0.0

	AGGREGATE BIN FRACTIONS							"RECYCLED MATERIALS"			
	Bin No.1	Bin No.2	Bin No.3	Bin No.4	Bin No.5	Bin No.6	Bin No.7	Bin No.8	Bin No.9	Bin No.10	
Aggregate Source:	Limestone_Dolom	Limestone_Dolom	Limestone_Dolom	Limestone_Dolom	Gravel			Fractionated RAP	RAS		Material Type
Aggregate Pit:	Ruby	Ruby	Ruby	Ruby					Ramming LTD-Buda		Material Source
Aggregate Number:											RAS Type
Sample ID:	Type C	Type D	Type F	Mfg. Sand	Field Sand						Sample ID
Recycled Material?:								Recycled Asphalt Binder (%)			
Asphalt%:								5.0	20.0		
Hydrated Lime?:								0.0 %ofTot Mix	0.0 %ofTot Mix	%ofTot Mix	Total Bin
Individual Bin (%):	25.0 Percent	20.0 Percent	22.0 Percent	24.0 Percent	9.0 Percent	Percent	Percent	%of Aggreg	%of Aggreg	%of Aggreg	100.0%

Combined Gradation

Sieve Size:	Bin 1 Cum.% Passing	Bin 1 Wtd Cum. %	Bin 2 Cum.% Passing	Bin 2 Wtd Cum. %	Bin 3 Cum.% Passing	Bin 3 Wtd Cum. %	Bin 4 Cum.% Passing	Bin 4 Wtd Cum. %	Bin 5 Cum.% Passing	Bin 5 Wtd Cum. %	Bin 6 Cum.% Passing	Bin 6 Wtd Cum. %	Bin 7 Cum.% Passing	Bin 7 Wtd Cum. %	Bin 8 Cum.% Passing	Bin 8 Wtd Cum. %	Bin 9 Cum.% Passing	Bin 9 Wtd Cum. %	Bin 10 Cum.% Passing	Bin 10 Wtd Cum. %	Total Bin Cum. % Passing	Lower	Upper	Within Spec's	Restricted Zone Lower	Restricted Zone Upper	Restricted Zone Within Spec's	Individual % Retained	Cumulative % Retained	Sieve Size
1" 25.000	100.0	25.0	100.0	20.0	100.0	22.0	100.0	24.0	100.0	9.0					100.0	0.0	100.0	0.0			100.0	100.0	100.0	Yes				0.0	0.0	1"
3/4" 19.000	100.0	25.0	100.0	20.0	100.0	22.0	100.0	24.0	100.0	9.0					100.0	0.0	100.0	0.0			100.0	95.0	100.0	Yes				0.0	0.0	3/4"
3/8" 9.500	56.4	14.1	70.7	14.1	100.0	22.0	100.0	24.0	100.0	9.0					92.4	0.0	100.0	0.0			83.2	70.0	85.0	Yes				16.8	16.8	3/8"
No. 4 4.750	10.9	2.7	14.3	2.9	76.4	16.8	99.9	24.0	99.8	9.0					71.1	0.0	99.7	0.0			55.4	43.0	63.0	Yes				27.9	44.6	No. 4
No. 8 2.360	4.7	1.2	6.3	1.3	20.6	4.5	89.8	21.6	98.1	8.8					53.0	0.0	98.9	0.0			37.3	32.0	44.0	Yes				18.0	62.7	No. 8
No. 30 0.600	3.3	0.8	3.7	0.7	6.2	1.4	40.3	9.7	90.5	8.1					33.2	0.0	62.8	0.0			20.7	14.0	28.0	Yes				16.6	79.3	No. 30
No. 50 0.300	2.6	0.7	3.3	0.7	4.8	1.1	24.2	5.8	66.9	6.0					21.9	0.0	53.7	0.0			14.2	7.0	21.0	Yes				6.6	85.8	No. 50
No. 200 0.075	2.2	0.6	2.7	0.5	3.9	0.9	7.6	1.8	3.7	0.3					6.4	0.0	23.4	0.0			4.1	2.0	7.0	Yes				10.1	95.9	No. 200
0.000																														
0.000																														

(Bold Italic) Not within specifications ***(Bold Italic)*** Not within specificaitons- Restricted Zone *(Italic)* Not cumulative

Lift Thickness, in:	2.00	Binder Substitution?	No	Binder Originally Specified: PG 70-22	Substitute Binder: PG 64-22
Asphalt Source & Grade:	Valero	Binder Percent, (%):	5.2	Asphalt Spec. Grav.:	1.033
Antistripping Agent:		Percent, (%):			

Dry Rodded Unit Weight of Coarse Agg. (pcf):

Figure A-3. Mixture design used in Test Section 1 (HMA without RAP or RAS, PG 70-22).

Table A-2. List of test sections with construction date for the Texas I field site.

Section No.	Lot No.	Mixture Description				Date of Paving
		Type	Binder	RAP %	RAS %	
1	1	HMA	PG 70-22	0	0	12/01/11
7	2	WMA Foaming	PG 70-22	0	0	12/01/11
9	3	WMA Evotherm DAT™	PG 64-22	15	3	12/13/11
8	4	WMA Evotherm DAT™	PG 70-22	0	0	01/04/12
3	5	HMA	PG 64-22	15	3	01/05/12
4	6	HMA	PG 64-22	0	5	01/06/12
2	7	HMA	PG 64-22	30	0	01/16/12
5	8	HMA	PG 58-28	30	0	01/17/12
6	9	HMA	PG 58-28	15	3	01/18/12

Plant and Mixture Production

The Ramming Paving Company's RTI division (referred to as RTI) hot mix plant located in Buda, Texas, supplied the asphalt mixture. This plant was approximately 30 mi away from the jobsite. The driving time between the asphalt plant and job site was between 30 to 40 minutes. RTI had an approximately 10-year-old Astec double-barrel unitized drum mixer (counter-flow) with a capacity of 350 tons/hour production rate (see Figure A-4). The dimension of the drum was 35 ft in length and 8 ft in diameter. The plant also had seven cold-feed bins in addition to one RAP bin and one RAS bin. There were three storage silos, with each having a capacity of 200 tons. The plant had a conventional baghouse fines collection system, and part of the baghouse fines was reintroduced into the drum. A drag slat conveyor carried the mixture from the drum to the storage silo. The plant was equipped with one lime silo and two vertical Heatec binder storage tanks. An Astec green foaming system was added to the plant approximately 2 years prior to this construction job. The Evotherm® DAT™ WMA additive was pumped from temporary tanks and added to the asphalt line during production.

Typically, the plant was initially operated at higher-than-normal production temperatures and was brought down to the target temperature after a few truckloads. The moisture content of the aggregate was somewhere between 4 and 5 percent. The average silo storage time was between 10 and 12 minutes. RTI employed 12 belly dump trucks to haul the loose mixtures to the construction site. The trucks had tarps on them to reduce heat loss.

Figure A-4. RTI hot mix plant located in Buda, Texas.

Construction

The contractor repaired and patched some areas of existing surface distress (especially at the north end of the project) in November before the start of the overlay construction. Two mixtures (i.e., Test Section 1—HMA control, and Test Section 7—WMA foaming) were produced and placed on the first day of production on December 1, 2011. The rest of the mixtures were produced and placed one per day. Section 1 and 7, both approximately 2000 ft long, were placed side by side on the southbound and northbound lanes, respectively. All other test sections were placed on both directions of the roadway.

Just before placing the overlay, a layer of underseal (or seal coat) was placed on top of the existing pavement surface (see Figure A-5). The seal coat used was a CHFRS-2P emulsion sprayed at a rate of 0.25 gal/yd^2 and covered by a Grade 4 Type B uncoated limestone aggregate at a rate of 260 yd^2/yd^3. Then, the belly dump trucks released the loose mix on the fresh seal coat and a material transfer vehicle, alternately known as a shuttle buggy, picked up the mixture and transferred it to

Figure A-5. Application of underseal before overlay laydown.

Figure A-6. Windrow operation using shuttle buggy.

Figure A-7. Paver equipped with PAVE-IR bar.

the paver hoper (see Figure A-6). Typically, each day the chip seal placement started at 9:00 a.m. and the paving was completed by 3:30 p.m.

Table A-3 shows the list of the equipment used during construction. The paver was equipped with a MOBA brand PAVE-IR bar to record the surface temperature of the mat right behind the paver (see Figure A-7). The loose mat was compacted with one dual-wheel steel roller as a breakdown roller, one pneumatic-tire roller as an intermediate roller, and one small steel-wheel roller as a finisher. On one occasion, two pneumatic rollers were used. Paving was done from north to south regardless of the direction of travel. In general, the paving width was 16 ft (4.9 m) in each direction with an average of 2 in. (50 mm) of compacted mat thickness.

Sample Collection

Plant mix was collected from the truck right after discharge from the silo (see Figure A-8). Large quantities of mixtures were collected for later use in the laboratory as well as for on-site specimen preparation. In addition, the research team collected samples of the three asphalt binder performance grades, virgin aggregates, RAP, and RAS materials. The materials sampling scheme is summarized in Table A-4.

On-Site PMLC Specimen Compaction

The plant loose mix collected from the truck at the plant was quickly brought to the on-site mobile laboratory and placed in the oven from 1 to 2 hours to achieve the required compaction temperature. This on-site laboratory was owned by RTI. All on-site specimens were compacted using a Troxler Superpave gyratory compactor to 7 ± 1 percent AV content (see Figure A-9). As part of NCHRP 9-49, approximately thirteen 6.0-in. (152-mm) diameter specimens were compacted on-site using plant loose mix from five test sections.

Field Specimens

Because of several interruptions during construction due to weather conditions and holidays, all road cores were collected after the completion of the entire project and labeled as having the same field age, although the control section (HMA with PG 70-22) was constructed during the first week of December 2011 and the final test section was paved on January 18, 2012.

The first set of road cores was collected during the last week of January 2012. Thus, the first sets of cores collected from the different test sections were subjected to environmental and traffic conditions in the field between 2 weeks and 8 weeks.

Table A-3. Paving equipment used at the Texas I field site.

Equipment Type	Manufacturer	Model
Material Transfer Vehicle	Roadtec	SB 25000
Paver	Barber-Green	BG 2000
Breakdown Roller (steel-wheeled)	Volvo	
Pneumatic Roller	Bomag	24RH
Finish Roller (steel-wheeled)	Ingersoll Rand	
Finish Roller	Dynapac	CC 142

Figure A-8. Loose mix collection from the truck at RTI hot mix plant.

Figure A-9. Troxler compactor used to prepare the on-site PMLC specimens.

Table A-4. Materials sampling scheme at the Texas I field site.

Sample Type	Material	Point of Sampling
Lab Mixed, Lab Compacted	Fine Aggregate	Stockpile
	Coarse Aggregate	Stockpile
	RAP	Stockpile
	RAS	Stockpile
	Binder (all grades)	Transport Truck at Asphalt Mix Plant
	Evotherm DAT™ Additive	Asphalt Mix Plant
Plant Mixed, Lab Compacted	Loose Mix	Truck at Asphalt Mix Plant
Plant Mixed, Field Compacted First Set after Construction—January 2012	Road Cores	Travel Lane (between wheelpath)
Plant Mixed, Field Compacted Second Set—September 2012	Road Cores	Travel Lane (between wheelpath)
Plant Mixed, Field Compacted Third Set—March 2013	Road Cores	Travel Lane (between wheelpath)

Figure A-10. Typical Texas pavement structure including the recent overlay.

The road cores were drilled at the center of the travel lane (between wheelpath), spread approximately equal distance on both directions. Figure A-10 shows a typical core obtained from Section 9.

The next round of road cores was obtained during the second week of September 2012—approximately 8 to 9 months after overlay construction. In addition, the research team also collected a third set of cores from the test sections during March 2013.

Field Performance

An overall assessment of the condition of the pavement was performed on three occasions: 2 to 3 months, 6 to 7 months, and 14 to 15 months after construction. The conditions of Sections 1, 7, and 8 are shown in Figures A-11, A-12, and A-13.

New Mexico Interstate Highway 25 (NM IH 25)

General Description

The New Mexico test sections were located on IH 25 between Williamsburg and Elephant Butte, Sierra County, in the southeast part of the state. An overall view of the construction site is shown in Figure A-14. IH 25 is a four-lane (two lanes in each direction) rural highway with moderate traffic. The stretch of highway where the test sections were placed is characterized by rolling terrain. This field site was constructed during the third week of October 2012. James Hamilton Construction Company from Silver City, New Mexico, was the contractor for this project.

The total length of the overlay project was approximately 18 mi (28.8 km). Near the location of the test sections, the total roadway width was 26 ft. The main-lane paving width was approximately 13 ft. The average overlay thickness was 2.5 in. (63 mm). The existing pavement surface had moderate to severe alligator cracking on the wheelpath. Therefore, before

(a) HMA (b) WMA Foaming (c) WMA Evotherm

Figure A-11. Performance of the Texas field sections after 2 to 3 months in service.

(a) HMA (b) WMA Foaming (c) WMA Evotherm

Figure A-12. Performance of the Texas field sections after 6 to 7 months in service.

(a) HMA

(b) WMA Foaming

(c) WMA Evotherm

Figure A-13. Performance of the Texas field sections after 14 to 15 months in service.

Figure A-14. New Mexico IH 25 field site.

placing the overlay, the top of the existing pavement was milled off 2.5 in. (63 mm).

Mixtures and Materials

The New Mexico Department of Transportation (NMDOT) used a 0.75-in. (19-mm) NMAS dense-graded mixture (i.e., NMDOT SPIII) for the surface layer. The aggregate, a siliceous rock, was obtained from a nearby pit located very close to the Rio Grande River. This type of aggregate is highly absorptive; the combined aggregate water absorption at saturated surface dry condition was reported as 2.9 percent. The moisture content of the aggregate was between 3.5 to 4.0 percent. Thirty-five percent RAP was also added to the mixture, which was screened over a 2-in. (50-mm) sieve.

Besides the control HMA with RAP mixture, this project included a control HMA without RAP, WMA Foaming with RAP, and Evotherm 3G with RAP. Only 500 tons of HMA without RAP and approximately 2000 tons of WMA with Evotherm 3G were placed. The HMA without RAP mixture used 5.4 percent PG 76-22 asphalt binder. The other three mixtures, which included 35 percent RAP, were designed with a PG 64-28 modified asphalt binder. The percentage of virgin binder and total binder content for these mixtures was 3.0 percent and 5.4 percent, respectively. NuStar Energy Company from Santa Fe, New Mexico, supplied both asphalt binder types. Evotherm 3G was blended with the asphalt binder at 0.5 percent by weight of total binder content at the asphalt mixture plant.

The plant was equipped with an Astec Green System asphalt foaming system. During mixture production, the water added to produce the WMA foaming mixture was reported as 2.0 percent by weight of virgin asphalt binder content. Except for the HMA with RAP mixture, all other mixtures used the same aggregate gradation and binder content. Binder contents of the mixtures were verified during quality control through ignition oven analysis. All four mixtures incorporated 1 percent Versabind mineral filler mixed with the virgin aggregates at 4.4 percent moisture content using a pug mill before entering into the drum through the conveyor belt. The moisture content of the aggregate was approximately 3.5 to 4.0 percent. All four Superpave mixtures were designed at 100 gyrations (N_{des}). The mix designs with and without RAP are detailed in Figures A-15 and A-16.

Plant and Mixture Production

The asphalt mixture plant, a portable Astec double barrel (see Figure A-17), was a counter-flow drum design with an external mixing drum. The plant was about 10 years old at the time of construction. The capacity of the plant was 450 tons/hour, and it had two horizontal binder tanks. The plant did not have a silo; rather, it had a 50-ton capacity Astec surge bin. The plant used three (out of five) cold bins for virgin aggregates. RAP materials were screened over a 2-in. (50-mm) screen before entering the mixing drum. The plant had a conventional baghouse fines collection system, and part of the baghouse fines was reintroduced into the drum. A drag slat conveyor carried the mixture from the drum to the surge bin.

Typically, the plant operator started production at a higher temperature and then lowered it to the target mixing temperature after four or five truck loads. In this project, the majority of the asphalt mixture that was produced consisted of WMA foaming with 35 percent RAP. Although the project was almost 18 mi (29.0 km) long, the test sections were located within 3 mi (4.8 km). Table A-5 shows the temperature data for the particular production day and time when the loose mix for each mixture type was collected.

Construction

The asphalt mixture plant was located at the north end of the project, while the test sections were located toward the south end. The average hauling distance and time from the plant to the test sections were approximately 15 mi and 20 minutes, respectively. The mixtures were hauled using belly dump trucks with a tarp on top. Once the belly dump trucks released the mixtures on the road, a windrow pick-up machine (see Figure A-18) transferred the mixtures and dropped them into the paver chute.

This job used three steel-wheeled rollers (see list of paving equipment in Table A-6). The breakdown roller was used to compact the loose mix using seven passes in vibratory mode. The intermediate roller immediately followed, compacting with approximately seven passes. Surprisingly, the finish roller also operated in vibratory mode, although at very low frequency and low amplitude. The finish roller also used seven passes. Before paving, the contractor applied CSS-1H tack coat with a mixture ratio of 2:1 at a rate of 0.05 gal/yd^2 (0.23 L/m^2) over the milled surface.

New Mexico DEPARTMENT OF TRANSPORTATION
MOBILITY FOR EVERYONE

State Materials Bureau
ASPHALT DESIGN UNIT
P.O. Box 1149, Santa Fe, NM 87504-1149

Superpave Volumetric Mix Design for HMA - Version PA 12.8

SUMMARY OF MIX DESIGN

SP-III w/35% RAP Experimental HMA Test Section for James Hamilton Construction Co.

Project No.:	1C00003 - HMA Test Section		
Control No.:	1C00003	QLA?	Yes
Location:	I-25		
County:	Sierra	District:	1
Project Mgr:	Brian Torres	D1-Phone: 575-544-6581	
Dist Lab Spvr.	Gilberto Garcia	D1-Phone: 575-544-6581	
Contractor:	James Hamilton Construction Co.		
Designed by:	CA2 Testing		
HMA Supplier:	James Hamilton Construction Co.		

Aggregates[1]	Source #1	Source #2
Aggr. Source	T or C pit	N/A
Location:	T or C	N/A
County:	Sierra	N/A
Filler Source:	N/A	N/A
RAP Source:	Project	N/A
Versabind	GCC Cemento	

Design ESALs(ml)	Compaction Parameters $N_{initial}$	N_{design}	$N_{maximum}$
19.70	8	100	160
Estimated Plug Wt. @ N_{des}		4600	grams

Combination Used		Bulk Sp. Gravities	Abs. %
CA	33.0%	2.457	2.3
Int	7.0%	2.431	2.7
Dirty Fines	24.0%	2.396	3.7
RAP	35.0%	2.580	
Versabind	1.0%	3.000	

SML Mix Design No:	12-D1-17-134 HMA-
NMDOT Lab No:	2012-01534
ITL Mix Design No:	MD125-12
Mix Des Submittal Date:	10/30/2012
Date of Final Revision:	11/5/2012
Issue Date:	11/5/2012

Test Method used to Determine CA & FA Sp. Gr. & Absorps. AASHTO T-84 / T-85

NOTE: Field Verification SG Tests shall use the same method.

Method of Calibration for Gyratory Compactors: Internal Angle

NOTE: Same Calibration Method should be used for all GCs.

Asphalt

Grade Specified:	PG 76-22	Gr. Used:	PG 64-28
Sp. Gravity of Used Asphalt (Gb):			1.02
Asphalt Source:	Nustar		
Plant Location:	Rosario, NM.		
Mixing Temperature:	310 ° F		
Molding Temperature:	295 ° F		
Laydown Temperature:	295 +/- 22 F		
Lab Compaction Temp:	295 +/- 4 F		

Type of Mat'l:	Sand & gravel	
Type of Mix:	SP-III	
Est. HMA Qty.	109,270	Tons

Combined Aggregate Specific Gravity (G_{sb}):	2.486
Combined Specific Gravity of Fine Aggregates:	2.396
Combined Aggregate Water Absorption @ SSD:	2.9%

GRADATIONS USED:		2.0 in.	1.5 in.	1 in.	3/4 in.	1/2 in.	3/8 in.	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200
Percentage Passing:		100	100	100	95	80	66	43	29	20	15	11	8.4	5.6
SPECIFICATION:		100	100	100	90 - 100	Max 90			23-49					2.0-8.0
Acceptance Tolerences:	Max			100	100		74	50						7.0
	Min				90		58	36						4.2

SUMMARY OF DESIGN INFORMATION

Specimens compacted to N_{Design} 100 Gyrations

Design Parameters	Design[2] Values	Design Criteria Lower	Design Criteria Upper	Acceptance Limits Lower	Acceptance Limits Upper
Asphalt	5.4	-		4.9	5.9
Virgin A.C.	3.0	-		-	-
Air Voids	4.0	4.0		2.6	5.4
VMA	13.5	13.5	15.0	11.9	15.1
VFA	70.4	68.0	75.0	-	-
Gse	2.561	-		-	-
Eff.% AC	4.3	-		-	-
Versabind	1.0	1.0 Min		1.0	1.4

Design Parameters	Design[2] Values	Design Criteria Lower	Design Criteria Upper	Acceptance Limits Lower	Acceptance Limits Upper
Unit Weight, lb/ft3	141.8	-		-	-
Gmb (T-166) @ Ndes	2.272	-		-	-
Gmm (T-209) @ Design A.C.	2.367	-		-	-
Density @$N_{initial}$ 8 Gyrations	86.9	89 Max		-	-
Density @$N_{maximum}$ 160 Gyrations	97.5	98 Max		-	-
Dust Proportion	1.3	0.6	1.4	1.0	1.6
Binder Absorption	1.20	-		-	-
Water@ Mixing of + Aggregates	4.4%	-		-	-

PROPERTIES @ DESIGN GYRATIONS

AC (%)	Gmm (T-209)	Gmb (T-166)	Unit Wt. (lb/ft3)	Air Voids (%)	VMA (%)	VFA (%)	Dust to Binder Ratio
5.00	2.371	2.215	138.2	6.6	15.3	57.0	1.39
5.50	2.355	2.243	140.0	4.8	14.7	67.6	1.24
6.00	2.339	2.268	141.5	3.0	14.2	78.8	1.11
6.50	2.321	2.287	142.7	1.5	14.0	89.5	1.00

Tensile Strength Ratio (T 283-07)	Average Dry Tensile Strength	Average Wet Tensile Strength	Tensile Strength Ratio	Specification
	175.5 psi	156.8 psi	89	> or = 85

	Pit Mat'l.	RAP 1	RAP 2	Specs.	Limit
Flat & Elongated Pieces @ 3:1 (ASTM D4791)	6%	12%		20%	Max
Fine Aggregate Angularity (T-304, Method A)	46.6%			45%	Min
CA 1 Fractured Face (NMDOT)	100%	100%		95%	Min
CA 2 Fractured Faces (NMDOT)	100%	94%		90%	Min
Combined Sand Equivalent (T-176)	53%			45%	Min
Soundness Loss (T-104)	2%			15%	Max
LA Wear (T-96)	18%	35%		40%	Max - RAP
Aggregate Index Pit #1:	9			25	Max
Aggregate Index Pit #2:				25	Max

Tested by:	David Salas
	Mary
Reported by:	Danny Marquez
Reviewed & Submitted by:	Lea Ann Marquez
NM P.E. License No.	16029
Contact Phone:	505-718-3030

Special Notes & Instructions	
	1. The design data was obtained from material submitted by the Contractor.
	2. Minor field adjustments may be necessary to achieve and maintain specified limits.
	3. The specification limits are in accordance with Section 423 of the NMDOT Std Specifications for Road & Bridge Construction - 2007 Edition & Supp. Specs
	4. We removed 0.6% of the -#200 fraction during the verification of this design

Reviewed by:	Parvez Anwar P.E
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E-mail:	[illegible]

Figure A-15. Mix design for HMA and WMA mixtures with 35 percent RAP.

New Mexico DEPARTMENT OF TRANSPORTATION
MOBILITY FOR EVERYONE

State Materials Bureau
ASPHALT DESIGN UNIT
P.O. Box 1149, Santa Fe, NM 87504-1149

Superpave Volumetric Mix Design for HMA - Version PA 12.8

SUMMARY OF MIX DESIGN

SP-III w/PG 76-28 Experimental Virgin HMA Test Section for James Hamilton Construction Co.

Project No.:	1C00003 Virgin HMA Test Section		
Control No.:	1C00003	QLA?	Yes
Location:	I-25		
County:	Sierra	District:	1
Project Mgr:	Brian Torres	D1-Phone: 575-544-6561	
Dist Lab Spvr.	Gilberto Garcia	D1-Phone: 575-544-6581	
Contractor:	James Hamilton Construction Co.		
Designed by:	CA2 Testing		
HMA Supplier:	James Hamilton Construction Co.		

Aggregates[1]	Source #1	Source #2
Aggr. Source	T or C pit	N/A
Location:	T or C	N/A
County:	Sierra	N/A
Filler Source:	N/A	N/A
RAP Source:	N/A	N/A
Versabind	GCC Cemento	

Design ESALs(mi)	Compaction Parameters $N_{initial}$	N_{design}	$N_{maximum}$
19.70	8	100	160
Estimated Plug Wt. @ N_{des}		4500	grams

Combination Used		Bulk Sp. Gravities	Abs. %
3/4"	38.0%	2.522	1.0
3/8"	17.0%	2.521	0.9
CF	44.0%	2.484	0.7
Versabind	1.0%	3.000	

SML Mix Design No:	2012-01-19-136 VHMA-TS
NMDOT Lab No:	2012-01536
ITL Mix Design No:	MD126-12
Mix Des Submittal Date:	11/5/2012
Date of Final Revision:	11/7/2012
Issue Date:	11/7/2012

Test Method used to Determine CA & FA Sp. Gr. & Absorps. AASHTO TP 77 (NMDOT 001)

NOTE: Field Verification SG Tests shall use the same method.

Method of Calibration for Gyratory Compactors: Internal Angle

NOTE: Same Calibration Method should be used for all GCs.

Asphalt			
Grade Specified:	PG 76-22	Gr. Used:	PG 76-28
Sp. Gravity of Used Asphalt (Gb):			1.004
Asphalt Source:	Nustar		
Plant Location:	Rosario, NM.		
Mixing Temperature:	340 °F		
Molding Temperature:	320 °F		
Laydown Temperature:	320 +/- 22 F		
Lab Compaction Temp:	320 +/- 4 F		

Type of Mat'l:	Sand & Gravel	
Type of Mix:	SP-III	
Est. HMA Qty.	109,270	Tons

Combined Aggregate Specific Gravity (G_{sb}):	2.509
Combined Specific Gravity of Fine Aggregates:	2.484
Combined Aggregate Water Absorption @ SSD:	0.9%

GRADATIONS USED:		2.0 in.	1.5 in.	1 in.	3/4 in.	1/2 in.	3/8 in.	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200
Percentage Passing:		100	100	100	95	80	68	47	33	23	17	12	9.3	4.5
SPECIFICATION:		100	100	100	90 - 100	Max 90			23-49					2.0-8.0
Acceptance Tolerences:	Max			100	100		76	54						5.9
	Min				90		60	40						3.1

SUMMARY OF DESIGN INFORMATION

Specimens compacted to N_{Design} 100 Gyrations

Design Parameters	Design[2] Values	Design Criteria Lower	Design Criteria Upper	Acceptance Limits Lower	Acceptance Limits Upper	Design Parameters	Design[2] Values	Design Criteria Lower	Design Criteria Upper	Acceptance Limits Lower	Acceptance Limits Upper
Asphalt	5.4	-		4.9	5.9	Unit Weight, lb/ft3	140.7	-		-	-
Virgin A.C.	5.4	-		-	-	Gmb (T-166) @ Ndes	2.255	-		-	-
Air Voids	4.0	4.0		2.6	5.4	Gmm (T-209) @ Design A.C.	2.350	-		-	-
VMA	15.0	13.5	15.0	13.4	16.6	Density @$N_{initial}$ 8 Gyrations	86.9	89 Max		-	-
VFA	73.3	68.0	75.0	-	-	Density @$N_{maximum}$ 160 Gyrations	97.0	98 Max		-	-
Gse	2.545	-		-	-	Dust Proportion	0.9	0.6	1.4	0.6	1.2
Eff.% AC	4.9	-		-	-	Binder Absorption	0.57	-		-	-
Versabind	1.0	1.0 Min		1.0	1.4	Water@ Mixing of + Aggregates	2.4%	-		-	-

PROPERTIES @ DESIGN GYRATIONS

AC (%)	Gmm (T-209)	Gmb (T-166)	Unit Wt. (lb/ft3)	Air Voids (%)	VMA (%)	VFA (%)	Dust to Binder Ratio
4.90	2.360	2.233	139.4	5.4	15.3	65.1	1.00
5.40	2.349	2.263	141.2	3.7	14.7	75.0	0.92
5.90	2.333	2.282	142.4	2.2	14.4	85.0	0.84
6.40	2.309	2.296	143.3	0.5	14.3	96.3	0.75

Tensile Strength Ratio (T 283-07)	Average Dry Tensile Strength	Average Wet Tensile Strength	Tensile Strength Ratio	Specification
	188.2 psi	170.0 psi	90	> or = 85

	Pit Mat'l.	RAP 1	RAP 2	Specs.	Limit
Flat & Elongated Pieces @ 3:1 (ASTM D4791)	6%			20%	Max
Fine Aggregate Angularity (T-304, Method A)	48.5%			45%	Min
CA 1 Fractured Face (NMDOT)	100%			95%	Min
CA 2 Fractured Faces (NMDOT)	100%			90%	Min
Combined Sand Equivalent (T-176)	53%			45%	Min
Soundness Loss (T-104)	2%			15%	Max
LA Wear (T-96)	18%				
Aggregate Index Pit #1:	9			25	Max
Aggregate Index Pit #2:				25	Max

Tested by:	David Salas
	Navor Trujillo
Reported by:	Danny Marquez
Reviewed & Submitted by:	Lea Ann Marquez
NM P.E. License No.	16029
Contact Phone:	505-718-3030

Special Notes & Instructions	
	1. The design data was obtained from material submitted by the Contractor.
	2. Minor field adjustments may be necessary to achieve and maintain specified limits.
	3.The specification limits are in accordance with Section 423 of the NMDOT Std Specifications for Road & Bridge Construction - 2007 Edition & Supp. Specs
	4. We removed 2.5% of the -#200 fraction during the design phase of this mix

Reviewed by:	Parveez Anwar P.E.
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Figure A-16. Mix design for HMA mixture without RAP.

Figure A-17. Asphalt mixture plant at the New Mexico field site.

Figure A-18. Windrow equipment used at the New Mexico field site.

Sample Collection

Plant mix was collected right after the trucks released the mixtures in front of the windrow (see Figure A-19). The time that elapsed between the truck leaving the plant and the loose mix sample collection was approximately 20 minutes. A large quantity of mixtures was collected for later use in the laboratory using the scheme listed in Table A-7. A smaller quantity of mixture collected from the road was immediately brought back to the FHWA mobile laboratory trailer located next to the project site for on-site specimen compaction.

The research team also collected PG 76-22 and PG 64-28 asphalt binder, virgin aggregate, Versabind (mineral filler), Evotherm 3G, and RAP from the asphalt mixture plant. With the help of the contractor's personnel, 48 road cores from four test sections were collected right after construction.

On-Site PMLC Specimen Compaction

Fifty-six 6-in. (150-mm) diameter specimens were compacted on-site using plant mix at the FHWA mobile laboratory trailer located next to the project site in Williamsburg. Thirty-two of them were 2.4 in. (61 mm) tall, and 24 of them were 3.75 in. (95 mm) tall. Plant loose mix collected by the windrow was quickly brought to the mobile laboratory trailer and placed in the oven for from 1 to 2 hours to achieve the desired compaction temperature. Specimens were compacted using an IPC Superpave gyratory compactor to 7 ± 1 percent AV content. In addition, six specimens of each of these four mixtures were compacted for Asphalt Mixture Performance Tester (AMPT) testing. The AMPT specimens were cored and sawed from larger specimens to a final dimension of 4 in. (100 mm) in diameter by 6 in. (150 mm) in height.

Connecticut Interstate Highway 84 (CT IH 84)

General Description

This overlay project located on westbound IH 84 (Figure A-20) on the eastside of Hartford Connecticut, was constructed in the third week of August 2012. Tilcon, Inc., of

Table A-5. Production, paving, and ambient temperatures for the New Mexico field site.

Mixture Type	Date of Production	Plant Mix Temp (°F)	Paving Temp (°F)	Ambient Temp (°F)
WMA Foaming with RAP	10.16.12	285	265–270	60 to 80
HMA with RAP	10.17.12	315	285–290	60 to 80
HMA without RAP	10.18.12	345	330–335	60 to 80
WMA Evotherm 3G with RAP	10.19.12	275	255–260	60 to 80

Table A-6. Paving equipment used at the New Mexico field site.

Equipment Type	Manufacturer	Model
Windrow	Weiler	E 650A
Paver	Caterpillar Inc.	CAT 10-20B
Breakdown Roller (steel-wheeled)	Bomag	HYPAC C784A
Intermediate Roller (steel-wheeled)	Bomag	HYPAC C784A
Finish Roller (steel-wheeled)	Bomag	HYPAC C784A

Figure A-19. Loose mix sampling at the New Mexico field site.

Figure A-20. IH 84 at the Connecticut Field Site.

Oldcastle Materials Group, was the contractor for this overlay paving job. This stretch of IH 84 is a heavily trafficked road located in urban area. The paving job was done during the night time. The average thickness was 2.0 in. (50 mm). The asphalt plant was located about 24 miles from the construction site. The hauling time was approximately 30 to 45 minutes.

Researchers from National Center for Asphalt Technologists (NCAT) were present during the construction for monitoring and sample collection. They also brought their mobile laboratory into the asphalt plant for sample collection and specimen preparation.

Mixtures and Materials

This project used a 0.5-in. (12.5-mm) NMAS dense-graded Superpave mixture designed with 100 gyrations for the surface layer. A basalt virgin aggregate was used with 20 percent RAP in this mixture. The RAP came from multiple sources. RAP materials were crushed to 0.5 in. (12.7 mm) and then screened over a 9⁄16-in. (14.3-mm) sieve before entering the plant. Two types of mixtures used in this job were: HMA and WMA foaming. Both mixtures were identical in design except that the WMA foaming used 2 percent water by weight of binder. The aggregate stockpile percentages are provided in Table A-8.

The total binder content of this mixture was 4.9 percent. The virgin binder and the binder from RAP were 3.7 percent and 1.2 percent, respectively. Binder contents of the mixtures were verified during quality control through ignition oven analysis. The virgin binder, PG 76-22 (SBS modified), was supplied by Pekham Materials from Athens, New York. This mixture did not have any anti-stripping agent. The detail mix design is shown in Figure A-21.

Table A-7. Material sampling scheme at the New Mexico field site.

Sample Type	Material	Point of Sampling
Lab Mixed, Lab Compacted	Fine Aggregate	Stockpile
	Coarse Aggregate	Stockpile
	RAP	Stockpile
	Versabind	Silo
	PG 76-22 Asphalt	Transport Truck
	PG 64-28 Asphalt	Transport Truck
	Evotherm 3G	Plant
Plant Mixed, Lab Compacted	Loose Mix	Windrow
Plant Mixed, Field Compacted First Coring—October 2012	Road Cores	Main Lane (between wheelpath)

Table A-8. Aggregate percentages for of HMA production and WMA production at the Connecticut field site.

Aggregate Stockpile	Total Aggregate (%)
1/2s	29.6
Natural Sand	6.4
Stone Sand A	11.2
Stone Sand B	23.2
Crushed RAP	32.8

Plant	Tilcon	Project:	
Location	New Britain D	Mix type:	12.5mm Level: 3
Plant Type/Capacity	Drum / 600 TPH	Gyrations:	8/100/160
Submitted by	JRI		
Date Submitted	30-Apr-12	Revision Date:	

Description	Size/Type of Aggregate	Source of Supply	Location	Blend Percent
1 1/2"				
1"				
1/2"	1/2"	New Britain		32
3/8"	3/8"	New Britain		0
1/4"				
Natural Sand	Natural Sand	New Britain		8
Screenings				
Stone Sand	Stone Sand	New Britain		40
Rap	Rap	New Britain		20
Shingles				
Mineral Filler				
	PG Grade	76-22	Pecham	100
	Anti-Stripping			

Nom. Size	Contractor Data															
	Coarse Aggregates					Fine Aggregates						Calc.	Specifications			Contractor
12.5mm	Agg 1	Agg 2	Agg 3	Agg 4	Agg 5	Agg 6	Agg 7	Agg 8	Agg 9	Agg 10	Agg 11	JMF	Cont. pt	Rest. Zone	PCS	JMF
Description	1 1/2"	1"	1/2"	3/8"	1/4"	N.S.	Scr.	S. S.	RAP	Shingles	MF	Comp.				Submitted
Blend Percent			32.0			8.0		40.0	20.0			100.0		*Info Only*		
0.075						2.6		4.4	6.2			3.2	2-10			*4.0*
0.150						7.7		6.3	15.0			6.1				*6.0*
0.300						28.2		9.3	20.0			10.0		15.5		*12.0*
0.600						64.4		19.0	27.0			18.2		19.1-23.1		*20.0*
1.18				1.5		84.7		38.5	38.0			29.8		25.6-31.6		*29.0*
2.36			0.7	2.1		93.1		71.0	52.0			46.5	28-58	39.1	39	*48.0*
4.75			0.8	15.2		97.7		99.4	71.0			62.0				*67.0*
9.5			28.9	92.1		100.0		100.0	93.6			76.0	Max 90			*86.0*
12.5			92.4	100.0		100.0		100.0	100.0			97.6	90-100			*97.0*
19.0			100.0	100.0		100.0		100.0	100.0			100.0	Min. 100			*100.0*
25.0			100.0	100.0		100.0		100.0	100.0			100.0				
37.5			100.0	100.0		100.0		100.0	100.0			100.0				
50.0			100.0	100.0		100.0		100.0	100.0			100.0				
Pb						Production Pb (w/ RAP)			4.9%		JMF Pb	5.0	Min 4.8%			5.0
Gsa			2.986	3.000		2.740		2.995	2.851			2.943				*2.943*
Gsb			2.932	2.924		2.595		2.835	2.727			2.825				*2.825*

Test Results		
Gmm	2.64	
Gmb - Nmax	2.577	
Gmb - Ndes	2.543	
Gmb - Nin	2.304	
Height-Nmax	113.3	
Height-Ndes	114.8	
Height-Nmin	126.7	
Gse	2.875	
Va - Ndes	3.7	
VMA	14.5	14-16
VMA_a	14.4	
% Gmm@Nmax	97.6	
% Gmm@Nmin	87.3	90.5
VFA - Ndes	74.4	65/75
Pba	0.6	
Pbe	4.4	
Dust/Pbe	0.7	

Comments:

Figure A-21. Mix design for CT IH 84.

Figure A-22. Tilcon asphalt plant.

Figure A-23. Paver equipped with PAVE-IR bar.

Plant and Mixture Production

The asphalt plant was an Astec double barrel, a counter-flow drum design with an external mixing drum (Figure A-22). The production capacity of the plant was 400 tons/hour, and it had two horizontal binder tanks. The plant had nine 300-ton silos. The plant had a conventional baghouse fines collection system, and part of the baghouse fines was reintroduced into the drum. Drag slat conveyor carried the mixture from the drum to a silo. The plant was equipped with an Astec Green System asphalt foaming system.

Table A-9 shows the data for the particular production day and time when loose mix was collected. There was very little difference (10°F) between the production temperature of HMA and WMA for this job. This was a conscious decision of the agency.

Construction

Paving for both technologies (mixtures) were done in tandem with two different pavers. The average distance between the asphalt plant and the construction site was approximately 24 mi. The average haul time was approximately 30 to 45 minutes. The mixtures were hauled using 24 belly dump trucks with tarps (12 to each paving crew). Once the belly dump trucks released the mixtures on the road, a material transfer device picked up the mixtures and dropped them into the paver chute. This job used two steel-wheeled rollers and one pneumatic roller (Table A-10). The paver was equipped with Pave-IR bar (Figure A-23) to measure the temperature of the mat right behind the paver.

Sample Collection

Aggregate and mixture moisture samples were taken twice a night. Finally, the aggregate, RAP, and virgin binder used in the mixtures were sampled so that LMLC specimens could be fabricated.

Plant mix was collected from the truck at the plant. The research team also collected some road cores from these test sections.

The materials sampling scheme is shown in Table A-11.

Table A-9. Production, paving, and ambient temperatures at the Connecticut field site.

Mixture	Date of Production	Plant Mix Temp	Paving Temp
HMA	08.19.12	322°F (avg.)	296°F (avg.)
WMA Foaming	08.20.12	312°F (avg.)	294°F (avg.)

Table A-10. Equipment used for placement and compaction at the Connecticut field site.

Equipment Type	Manufacturer	Model
Material Transfer Device		
Paver	Caterpillar Inc.	CAT AP1000D
Breakdown Roller (Steel-wheeled)	Caterpillar Inc.	
Pneumatic Roller	Caterpillar Inc.	
Finish Roller (Steel-wheeled)	Caterpillar Inc.	

Table A-11. Sample collection scheme at the Connecticut field site.

Sample Type	Material	Point of Sampling
Lab Mixed, Lab Compacted	Fine Aggregate	Stockpile
	Coarse Aggregate	Stockpile
	RAP	Stockpile
	PG 76-22 Binder	Plant
Plant Mixed, Lab Compacted	Loose Mix	Truck at the Plant
Plant Mixed, Field Compacted	Road Cores	Main lane

Field LMLC Specimen Compaction

NCAT's mobile laboratory was on site for both nights of paving. Twenty-five 5-gal buckets of mixture were sampled each night about 2 hours into production once the plant was running smoothly. Volumetric and performance samples were compacted on the laboratory each night without letting the mixture cool down in order to negate the effect of reheating the mixture. The following samples were made:

- Three volumetric samples at 100 gyrations using the same mass as the quality control laboratory
- E* and flow number
- S-VECD
- Hamburg
- IDT
- Overlay tester
- TSR

Wyoming State Route 196 (WY SR 196)

General Description

The Wyoming project (Project STP 1006020) was sampled in cooperation with Western Research Institute and the Wyoming Department of Transportation (WYDOT). This 1.5-in. (38-mm) overlay project was constructed on State Route 196 (Figure A-24) near the town of Buffalo during the third week of August 2013. McGarvin-Moberly Construction Co. was the general contractor for this project. This overlay project was approximately 5 mi long.

Figure A-24. Field site on WY Route 196.

Besides HMA, two WMA technologies were used in this project: foaming and Evotherm. Two different production temperatures were used with each technology. WYDOT personnel collected all the samples and data and prepared specimens on site.

Mixtures and Materials

The mixtures consisted of a 0.5-in. (12.5-mm) NMAS dense-graded aggregate with 5 percent PG 64-28 binder from Cenex (polymer modified) and 1 percent lime. No RAP was used in these mixtures. Mixtures produced included a control HMA (315°F [157°C]), Evotherm warm mix at two different temperatures (275°F [135°C] and 255°F [124°C]), and Gencor foam warm mix at two different temperatures (295°F [146°C] and 275°F [135°C]).

The mixtures contained 36 percent coarse aggregate (½ in. [12.7 mm]), 16 percent coarse aggregate (⅜ in. [9.5 mm]), 40 percent crushed fines, and 8 percent washed sand to achieve the gradations in Table A-12.

All five sections essentially had same aggregate structure and binder content. The mixtures contained 5.0 percent PG 64-28 added by weight of total mixture and 1.0 percent hydrated lime added by weight of dry aggregate.

Plant and Mixture Production

The plant was a portable, counter-flow drum manufactured by Gencor with a 70-ton silo (Figure A-25). This Gencor Ultra Drum with a production capacity of 400 tons/hour was built in 1997. The dimensions of the drum was 9 ft. (2.7 m) in diameter and 46 ft. (14 m) long. The plant had two horizontal binder storage tanks. The plant was fitted with Gencor Ultrafoam GX System foaming device (Figure A-26).

Weather was generally mild to warm throughout the duration of the paving. Partial to full sun was visible during paving and temperatures were between 65°F to 85°F (18°C to 29°C). The plant production temperatures and ambient temperatures are presented in Table A-13. There was little or no wind except on the morning of August 22 when there was some strong wind.

Table A-12. Aggregate gradation for the Wyoming field site.

Sieve Size	WYDOT Gradation	Dowl Hkm Combined Gradation	Stockpile Average	JMF	JMF Tolerance	Wide Band Specification
¾ in.	100	100	100	100	100	100
½ in.	93	94	94	94	90 to 100	90 to 100
⅜ in.	74	74	76	74	70 to 80	55 to 90
No. 4	44	44	44	44	39 to 49	35 to 70
No. 8	31	30	31	30	26 to 34	20 to 55
No. 30	17	15	16	15	12 to 18	5 to 35
No. 200	5.6	5.2	5.8	5.0	3.0 to 7.0	2 to 7

Figure A-25. Asphalt plant at the Wyoming field site.

Figure A-26. WMA foaming system at the Wyoming field site.

Construction

The average distance between the asphalt plant and the construction site was only 3 mi (4.8 km). The haul time was approximately 5 to 7 minutes. The mixtures were hauled using belly dump trucks with tarp. Once the belly dump trucks released the mixtures on the road in a windrow, they were picked up and dropped into the paver chute. This job used two steel-wheeled vibratory rollers for breakdown (Figure A-27). Each of the breakdown rollers typically made five passes followed by five passes of pneumatic-tired roller (Figure A-28). The finish roller operated at static mode made 2 to 3 passes. The equipment used in placing the mixtures is given in Table A-14. Due to the short haul distance and favorable weather condition, the mixture temperatures decreased only 5°F to 10°F, as presented in Table A-13. The paving temperatures were very close the mixture production temperature.

Sample Collection

Loose mix was collected from the plant for on-site and off-site laboratory-compacted specimens. Besides plant-produced loose mix, bulk quantity of aggregate and asphalt were collected from the plant and later shipped to the research team for further testing. The sampling scheme is presented in Table A-15. Samples of loose mix were taken from the silo using a front-end loader as shown in Figures A-29 and A-30. WYDOT personnel also obtained 26 road cores from these five sections.

Table A-13. Plant, paving, and ambient temperatures for the Wyoming field site.

Mixture	Date of Production	Plant Mix Temp	Paving Temp	Ambient Temp
Hot mix	08.20.2012	315°F	305°F	
WMA Foam	08.20.2012	295°F	290°F	
WMA Foam	08.21.2012	275°F	270°F	65°F to 85°F
WMA Evotherm 3G	08.22.2012	275°F	270°F	
WMA Evotherm 3G	08.23.2012	255°F	250°F	

Figure A-27. Breakdown with vibratory rollers.

Figure A-28. Compaction using pneumatic roller.

Field PMLC Specimen Compaction

Thirty 6-in. (150-mm) diameter and 2.4-in. (61-mm) tall specimens were compacted on site using plant mix at the WYDOT laboratory located in Buffalo, WY. Plant loose mix collected from the silo using a front loader was quickly brought to the laboratory and placed in the oven between 1 and 2 hours to achieve respective compaction temperature for that particular mixture. Specimens were compacted using a Superpave gyratory compactor to 7 ± 1 percent AV content.

South Dakota Highway 262 (SD SH 262)

General Description

The South Dakota warm mix test sections, located on South Dakota State Highway 262 between Bridgewater and Alexandria (Figure A-31), were constructed during the first and second week of October 2012. This construction project, P 0262(06)356, was mainly designed as part of a South Dakota Department of Transportation (SDDOT) WMA research project conducted by the University of Nevada, Reno. The project location was in the southeast part of the state spread between two counties: Hanson and McCook. The contractor for this job was Commercial Asphalt Company located at Spencer, SD. Commercial Asphalt Company is part of Spencer Quarries, Inc. Besides TTI, two other universities participated in this project: University of Nevada, Reno, and Louisiana State University Transportation Research Center (NCHRP 9-48).

SD 262 is a two-lane two-way rural highway with light to moderate traffic. Occasionally it experiences some high-volume farming-related truck traffic. The total length of the project was approximately 17 mi (27.22 km) with average paving width of 28 ft. (8.54 m). The average thickness was 2.0 in. (50 mm). Most of the existing pavement has old concrete pavement underneath. It had moderate to severe transverse

Table A-14. Equipment used for laydown and compaction at the Wyoming field site.

Equipment Type	Manufacturer	Model
Windrow		
Paver	Caterpillar Inc.	CAT AP1055D
Two Breakdown Rollers (Steel-wheeled)	Caterpillar Inc.	CAT CB64
Pneumatic Roller	Bomag	
Finish Roller (Steel-wheeled)	Dynapac	

Table A-15. Material sampling scheme at the Wyoming field site.

Sample Type	Material	Point of Sampling
Lab Mixed, Lab Compacted	Fine Aggregate	Stockpile
	Coarse Aggregate	Stockpile
	Lime	Silo
	PG 64-28 Asphalt	Transport Truck
Plant Mixed, Lab Compacted	Loose Mix	Silo Through a Front Loader
Plant Mixed, Field Compacted	Road Cores	Main Lane (between wheelpath)

Figure A-29. Mixture samples retrieved by bucket loader.

and longitudinal cracking. Before the placement of the overlay, the existing pavement was milled to a depth of 1.0 in. (25 mm). Later the milled surface was either patched or bladed as needed to make it level. Some small areas had two lifts of overlay.

Mixtures and Materials

This project used a 0.5-in. (12.5-mm) NMAS dense-graded (SDDOT Q2R) for the surface layer. This mixture used virgin aggregate (quartzite) from nearby Spencer Quarries. The 20 percent RAP used in this mixture also contained mostly quartzite rocks. The RAP materials were screened over a 1.5-in. screen.

Besides the control HMA mixture, this project had three WMA sections made with Plant Foaming, Advera®, and Evotherm. Approximately 5000 tons or more of mixtures were paved using each of the WMA technologies. Jebro Inc. provided the SBS-modified PG 58-34 binder, manufactured using various Canadian sources, from its Sioux City, Iowa, terminal. Advera was added with the mixture at the asphalt plant, but Evotherm (3G) was blended with the binder at the terminal. The plant was equipped with an Astec Green System asphalt foaming system. All four mixtures basically used same aggregate structure. All four mixtures used 1 percent hydrated lime mixed with the virgin aggregates using a pug mill before entering into the drum through the conveyor belt. The moisture content of the aggregate was approximately 1.5 to 2 percent. A recent drought in South Dakota made the aggregates fairly dry. All four Superpave mixtures were designed at 50 gyrations (N_{des}) and had 3.8 percent and 5.3 percent virgin binder and total binder content, respectively. Binder contents of the mixtures were verified during quality control through ignition oven analysis. Detailed mixture designs are shown in Figure A-32.

Figure A-30. Plant mix samples placed in buckets.

Figure A-31. SD 262 field site.

Plant and Mixture Production

The asphalt plant, a portable Astec double barrel (Figures A-33 and A-34), was a counter-flow drum design with an external mixing drum, and was 4 years old. The capacity of the plant was 400 tons/hour and it had two horizontal binder tanks. The plant did not have a silo; rather, it had a 50-ton capacity Astec surge bin. The plant used four (out of five) cold bins for virgin aggregates. RAP materials were screened over a 3-in. screen before entering into the mixing drum. The plant had a conventional baghouse fines collection system, and part of the baghouse fines was reintroduced into the drum. A drag slat conveyor carried the mixture from the drum to the surge bin. During the construction the slat conveyor experienced some malfunctions, especially during the production at lower temperatures (WMA). These malfunctions were most likely due to the high power requirements for the drag slat electric motors during WMA production.

REPORT ON AGGREGATE-BITUMINOUS MATERIAL PROPORTIONS
SOUTH DAKOTA DOT BITUMINOUS MIX DESIGN LAB
700 EAST BROADWAY AVE. PIERRE, SD

DOT 64

12-53

Project : P 0262(06)356 County : Hanson & McCook

PCN : 02QD Reported: 10-Sep-12

Job Location : SD262 fr I90 to SD42 - 17.050 Miles

Sample Submitted By : Spencer Quarry Aggr. Sub.: 14-Aug-12 All Data Sub.: 14-Aug-12

Aggregate Source : Spencer Quarry

Type of Work : Asphalt Concrete Resurfacing **Class Q2R**

Nominal Agg. Size : 12.5 mm

Prime Contractor : Spencer Quarries Inc

GRADATION OF MINERAL AGGREGATE USED FOR TRIAL MIXTURES
(Accumulative Percentages Passing)

% Virgin	17	17	38	28		Contractor Virgin	Virgin Aggregate
% Total =>	14	14	30	22	20	Agg.Comp.	Gradation
Sieve	3/4 Minus	9/16 Chip	Mix Agg	MnSnd	RAP	% Passing	Requirements
3/4 IN.	100.0	100.0	100.0	100.0	100.0	100	100
5/8 IN.	94.0	100.0	100.0	100.0	100.0		
1/2 IN.	67.0	87.0	100.0	100.0	100.0	92	85-99
3/8 IN.	33.0	62.0	100.0	100.0	96.0	82	75-89
# 4	3.0	17.0	92.0	100.0	72.6		
# 8	2.0	4.0	71.0	86.0	46.2	52	47-67
# 16	1.8	3.0	58.4	60.8	33.4		
# 30	1.5	2.0	49.8	43.4	27.2		
# 50	1.4	1.5	35.2	24.9	20.1		
# 200	1.0	1.0	7.6	1.1	7.2	4.1	2.1-6.1

Virgin Binder by weight of total mix = 3.8 per cent Jebro PG 58-34
Total Binder by wt. of total mix (estimate) = 5.3 per cent
Hydrated Lime by weight of total mix = 1.0 per cent

Gyratory Mix Design Verification as follows: Nini 6 Ndes 50 Nmax 75

	Gmb@ Ndes	Gmm @ Ndes	Va	%VMA	D/EB	Flat & Elong.	FAA	Snd Equiv.
SD DOT MIX LAB	2.319	2.441	5.0	16.0	0.9	1.5	47.3	73
CONTRACTOR LAB	2.348	2.446	4.0	14.8	1.1	1.8	47.0	72

Field Target Air Void Level : 4.0 % **Aggregate Moisture Content at SSD :** 0.5 %

Temperature of mixture when emptied from the drum: 295 ± 20 ° F
Temperature of mixture on delivery to the road: 285 + 30 or - 20 ° F
Asphalt application temperature at the mixer: 300 ± 20 ° F
Gyratory compaction temperature: 275 ± 5 ° F

Aggr.Composite Gsb
2.615 (Average Results)
Aggr -#4 Gsb
2.616 (Average Results)

Figure A-32. HMA mix design for South Dakota SD 262.

Figure A-33. Commercial Asphalt Inc. plant in South Dakota.

Figure A-34. Counter-flow drum at Commercial Asphalt Inc. plant in South Dakota.

Typically, the plant operator started the production at a higher temperature and lowered to the target mixing temperature within four to five truck loads. In this project, the majority of the asphalt mixture was supposed to be HMA. At the beginning of the project, the contractor started producing only hot mix; later they produced foam WMA. Nevertheless they produced and placed mixtures for 2 days each with Advera WMA and Evotherm WMA mixture. Table A-16 presents the data for the particular production day and time when loose mix was collected.

Construction

The average distance between the asphalt plant and the construction site was approximately 15 mi. The average hauling time was approximately 20 minutes. The mixtures were hauled using belly dump trucks with tarps. Once the belly dump trucks released the mixtures on the road, windrow picked up the mixtures and dropped them into the paver chute. This job used two steel-wheeled rollers and one pneumatic roller The breakdown roller compacted the paved mixture using four passes in vibratory mode. A pneumatic-tired roller was used as an intermediate roller for approximately four passes. The finish roller operated in both vibratory and static modes. The finish roller typically used one to two vibratory passes followed by one to two static passes. Starting from the laydown by the paver to the compaction by finish roller, paving was completed within 20 to 25 minutes. Before paving, the contractor applied CSS1H tack coat with a mix ratio of 2:1 @ 0.05 gal/yd^2 (0.23 L/m^2) rates. Table A-17 shows the equipment used for laydown and compaction.

The paver was instrumented with Pave-IR bar provided by MOBA. The Pave-IR bar collected the temperature of the mixture just behind the paver for the entire project length.

Sample Collection

Plant mix was collected from the road right after the trucks released the mixtures in front of the windrow for an average of approximately 25 minutes between production and sampling.

Table A-16. Production, paving, and ambient temperatures at the South Dakota field site.

Mixture	Date of Production	Plant Mix Temp	Paving Temp	Ambient Temp
WMA Foaming	10.03.12	270–275°F		
WMA Advera	10.04.12	275–280°F	240–250°F	52°F
WMA Evotherm 3G	10.05.12	265–270°F		
HMA	10.08.12	300–310°F	275–280°F	45°F

Table A-17. Paving equipment used at the South Dakota field site.

Equipment Type	Manufacturer	Model
Windrow	Weiler	E 650A
Paver	Caterpillar Inc.	CAT AP1055E
Breakdown Roller (Steel-wheeled)	Caterpillar Inc.	CAT CB64
Pneumatic Roller	Caterpillar Inc.	CAT PS360B
Finish Roller (Steel-wheeled)	Caterpillar Inc.	CAT CB64

Table A-18. Materials sampling scheme at the South Dakota field site.

Sample Type	Material	Point of Sampling
Lab Mixed, Lab Compacted	Fine Aggregate	Stockpile
	Coarse Aggregate	Stockpile
	RAP	Stockpile
	Lime	Manufacturer's Plant
	PG 58-34 Asphalt	Transport Truck
	PG 58-34 Asphalt with Evotherm	Transport Truck
Plant Mixed, Lab Compacted	Loose Mix	Windrow
Plant Mixed, Field Compacted	Road Cores	Main Lane (between wheelpath)

Large quantities of mixtures were collected for later use in the laboratory at TTI. The materials sampling scheme is presented in Table A-18. A smaller quantity of mixture collected from the road was immediately brought back to the SDDOT mobile laboratory located within the asphalt plant for compaction into 150-mm by 61-mm and 150-mm by 95.25-mm samples. The research team collected straight PG 58-34 and PG 58-34 binder blended with Evotherm, virgin aggregate, and RAP from the asphalt plant. Hydrated lime and Advera samples were provided by the respective manufacturer. With the help of SDDOT personnel, the research team also collected 40 road cores from four test sections.

Field LMLC Specimen Compaction

Fifty-six 6-in. (150-mm) diameter specimens were compacted on site using plant mix at the SDDOT mobile laboratory located within the asphalt plant premises. Thirty-two of them were 2.4 in. (61 mm) and 24 of them were 3.75 in. (95.25 mm) tall. Plant loose mix collected at the windrow was quickly brought to the mobile laboratory and placed in the oven between 1 to 2 hours to achieve respective compaction temperature for that particular mixture. Specimens were compacted using a Pine portable Superpave gyratory compactor (Model AGFB) to 7 ± 1 percent AV content.

Central Iowa Expo Center (IA Fairgrounds)

General Description

Unlike other test sections, Central Iowa Expo Center test sections, located in Boone County, were not laid out on a highway. Instead the test sections were laid out on a county fairground. The fairground is located on SH 17—a few miles east of Boone, Iowa. The fairground has a number of north–south and east–west streets forming a rectangular grid. As shown in Figure A-35 north–south (vertical) streets had eight different asphalt test sections and the east–west (horizontal) streets had concrete test sections. These streets are typically subjected to light- to medium-weight seasonal traffic during the fair and other special occasions. Although the owner of this facility is Boone County, the Iowa Department of Transportation paid for and executed this construction project. The surface layer (asphalt) was constructed toward the end of June 2013. This construction project, RTB-RB-34(013)-90-00, was mainly set up as a part of NCHRP 9-52 to study the effect of aggregate absorption and production temperature. Researchers from Iowa State University also conducted some of their tests at this location to study several aspects of asphalt and concrete pavements. The contractor and consultant of this job were Manatts, Inc. and Foth Infrastructure & Environment, LLC, respectively.

Each of the 13 streets with asphalt surface is approximately 700 ft (213 m) long and 18 ft. (5.5 m) wide (Figure A-36). Due to an east–west concrete pavement at the midway, each of the asphalt pavement streets was divided into two segments. The average thickness of surface layer was 2.0 in. (50 mm). Typically a 4-in. asphalt base mixture was placed on top of the existing 6-in. (150-mm) flexible base before paving the 2-in. (50-mm) surface layer. Some of the sections had additional treated sub-base or geo-grid beneath the flexible base layer. These test sections consisted of eight different mixtures (two levels of asphalt absorption × two mixtures [HMA and WMA] × two production temperatures).

Mixtures and Materials

This project used a 0.5-in. (12.5-mm) NMAS dense-graded mixture for the surface and base layers. Two separate mixtures were designed using two different aggregate blends: one with relatively low absorption (<1.0 percent) and one with high-absorption aggregate (≥1.5 percent). Again, each of these two aggregate blends was used to design HMA and foaming WMA. The plan was for each of the four mixtures to be produced at two different temperatures (control and control + 30°F). Due to some operational problems at the plant, production temperatures were adjusted from the planned temperatures. Figures A-37 and A-38 present the details of mixture designs.

Low-absorptive aggregate blends used limestone aggregates from Martin Marietta, Ames Mine, located just north of Ames, Iowa. Additionally, these mixtures used some field sand and 20 percent RAP. These aggregate blends had approximately 0.90 percent water absorption.

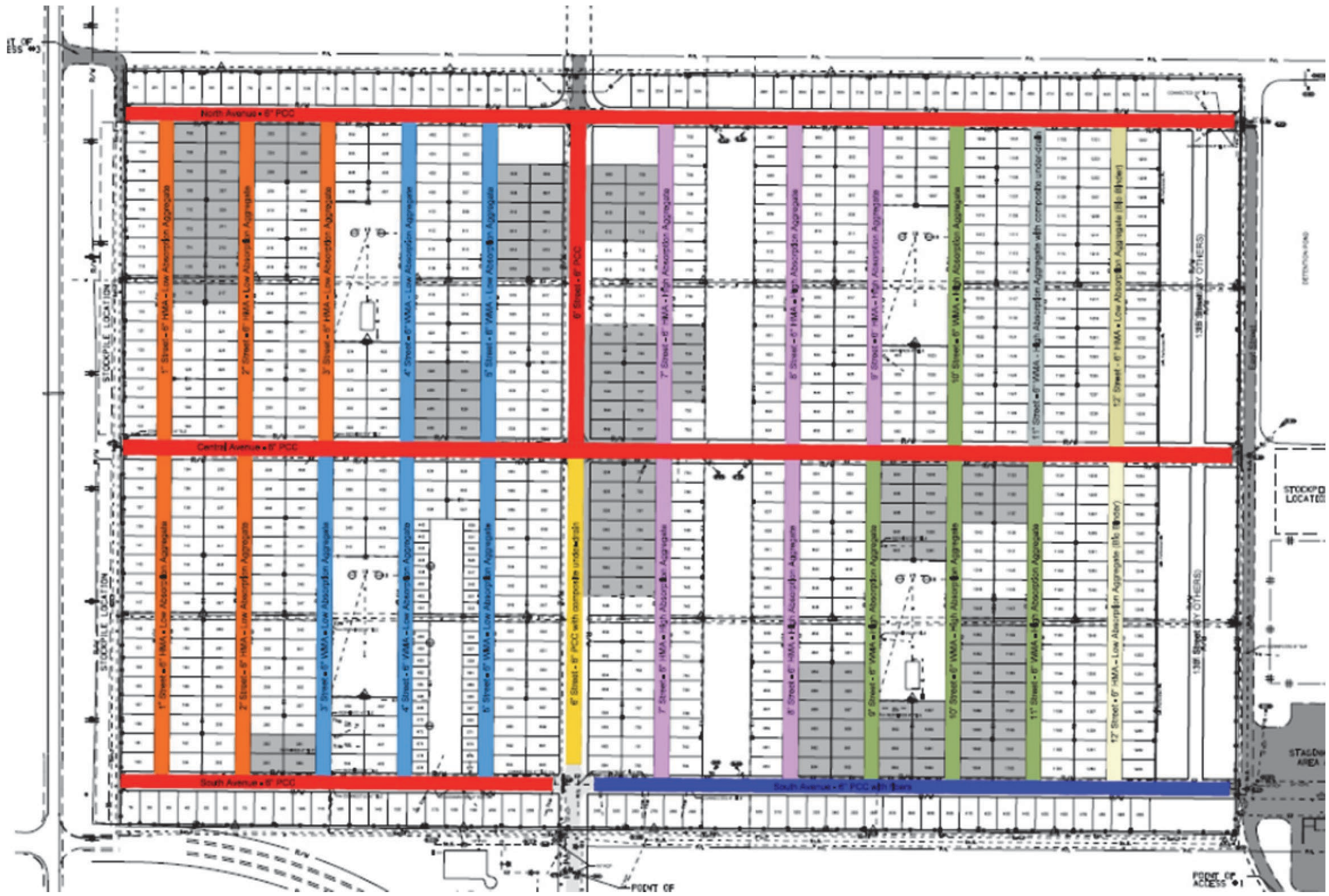

Figure A-35. Layout of test section at Central Iowa Expo Center.

The high-absorptive aggregate blends had limestone aggregate from Martin Marietta, Sully Mine, located just south of Newton, Iowa. Although both blends used field sand from the same source, the high-absorptive blends used 20 percent RAP from a different source. These aggregate blends had approximately 3.2 percent water absorption.

Figure A-36. Typical test section at Central Iowa Expo Center.

Bituminous Materials Company provided the PG 58-28 binder from its Des Moines, Iowa, terminal. The asphalt was produced at Flint Hills Resources refinery at Rosemount, Minnesota. This refinery used crude oil from Canadian source and did not use any modifier. For any given aggregate blend (low or high absorption), all four mixtures basically used the same aggregate structure. None of the mixtures used any anti-stripping agent. The moisture content of the low-absorptive aggregates was approximately 4.5 percent. The moisture content of the high-absorptive aggregates was approximately 6.5 percent. A heavy shower was reported a few days prior to surface layer paving. All mixtures were designed following the Superpave method at 76 gyrations (N_{des}). Mixtures with low-absorptive aggregate had 4.1 percent and 5.1 percent virgin binder and total binder content, respectively. On the other hand, mixtures with high-absorptive aggregate had 6.0 percent and 7.0 percent virgin binder and total binder content, respectively. However quality control data shows that slightly higher (0.1 to 0.25 percent) total asphalt content was found in the low-absorptive mixture. Binder contents of the mixtures were verified during quality control through ignition oven analysis. During the production of WMA, the quantity of water used was 1.5 percent of virgin binder content.

Form 956 ver. 10.03

Iowa Department of Transportation

Highway Division - Office of Materials

HMA Gyratory Mix Design

Mix Data

Nmax (Mix Transfer)

County :	Boone		Project : RTB-RB-34(013)--90-00		Letting Date :	1/16/2013
Mix Size (in.) :	1/2	Type A	Contractor : MANATTS - AMES		Mix No. :	1BD10-047
Mix Type:	HMA 1M	No Frictn Req	Design Life ESAL's :	1,000,000	Contract #:	00-RB34-013
Intended Use :	Surface		Location :		Date:	05/13/10

Aggregate	% in Mix	Source ID	Source Location	Beds	Gsb	%Abs	FAA	Friction
3/4 CRUSHED LC	20.0%	A85006	Ames Mine/Martin Marietta	#N/A	2.675	0.47	47.0	
1/2 CL CHIP LC	12.0%	A85006	Ames Mine/Martin Marietta	#N/A	2.680	0.44	47.0	
3/8 CL CHIP LC	8.0%	A85006	Ames Mine/Martin Marietta	#N/A	2.680	0.44	47.0	
MANF SAND	20.0%	A85006	Ames Mine/Martin Marietta	#N/A	2.673	0.60	46.0	
SAND	20.0%	A85510	Ames South/Hallett Materials Co	#N/A	2.588	1.30	40.0	
Certified RAP	20.0%		20% 1RAP13-001 (4.35 % AC)		2.576	1.84	41.5	

Job Mix Formula - Combined Gradation (Sieve Size in.)

	1"	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200
Upper Tolerance	100	100	100	95	72	54		26			6.3
	100	**100**	**96**	**88**	**65**	**49**	**35**	**22**	**9.5**	**5.2**	**4.3**
Lower Tolerance	100	100	89	81	58	44		18			2.3

Home

Asphalt Binder Source and Grade: Bituminous Matr'l & Supply (Tama, IA) PG 58-28

Gyratory Data

% Asphalt Binder	4.50	**4.95**	5.50	5.75	
Corrected Gmb @ N-Des.	2.374	**2.384**	2.397	2.406	
Max. Sp.Gr. (Gmm)	2.501	**2.484**	2.463	2.454	
% Gmm @ N- Initial	88.2	**89.2**	90.4	91.0	
%Gmm @ N-Max	96.1	**97.2**	98.5	99.1	
% Air Voids	5.1	**4.0**	2.7	2.0	
% VMA	14.1	**14.1**	14.1	14.0	
% VFA	63.9	**71.6**	81.0	86.0	
Film Thickness	7.90	**8.85**	10.00	10.52	
Filler Bit. Ratio	1.11	**1.00**	0.87	0.83	
Gsb	2.638	**2.638**	2.638	2.638	
Gse	2.682	**2.681**	2.680	2.680	
Pbe	3.89	**4.35**	4.92	5.17	
Pba	0.64	**0.62**	0.61	0.61	
% New Asphalt Binder	81.4	**83.1**	84.9	85.6	
Asphalt Binder Sp.Gr. @ 25°C	1.029	**1.029**	1.029	1.029	
% Water Abs	0.93	**0.93**	0.93	0.93	
S.A. m^2 / Kg.	4.92	**4.92**	4.92	4.92	
% +4 Type 4 Agg. Or Better	0.0	**0.0**	0.0	0.0	
% +4 Type 3 Agg.	0.0	**0.0**	0.0	0.0	
% -4 Type 2	0.0	**0.0**	0.0	0.0	
Fineness Modulus of Type 2	0.0	**0.0**	0.0	0.0	
Angularity-method A	42	**42**	42	42	
% Flat & Elongated	0.7	**0.7**	0.7	0.7	
Sand Equivalent	82	**82**	82	82	
Aggregate Type Used	A	**A**	A	A	
% Crushed	61	**61**	61	61	

Number of Gyrations

N-Initial

7

N-Design

76

N-Max

117

Gsb for Angularity

Method A

2.616

Pba / %Abs Ratio

0.67

Slope of Compaction Curve

15.3

Mix Check

Excellent

Pb Range Check

1.25

RAM Check

OK

Specification Check

Comply

Moisture Sensitivity Check

Not Required

Disposition : An asphalt content of 5.0% is recommended to start this project.

Data shown in 4.95% column is interpolated from test data.

The % ADD AC to start project is 4.1%

Figure A-37. Central Iowa Expo Center low-absorption HMA mix design.

Form 956 ver. 10.05

Iowa Department of Transportation

Highway Division - Office of Materials

HMA Gyratory Mix Design

Mix Data

Ndesign

County :	Boone		Project : RTB-RB-34(013)--90-00	Letting Date :	2/19/2013
Mix Size (in.) :	1/2	Type A	Contractor : MANATTS INC - AMES	Mix No. :	1BD13-048
Mix Type:	HMA 1M	No Frictn Req	Design Life ESAL's : 1,000,000	Contract #:	00-RB34-013
Intended Use :	Surface		Location : CENTRAL IOWA EXPO	Date:	06/07/13

Aggregate	% in Mix	Source ID	Source Location	Beds	Gsb	%Abs	FAA	Friction
3/4 Asphalt Stone	20.0%	A50002	Sully Mine/Martin Marietta	36-41	2.541	3.85	47.4	4
3/8 Chips	20.0%	A50002	Sully Mine/Martin Marietta	36-41	2.547	3.88	47.1	4
Manufactured Sand	20.0%	A50002	Sully Mine/Martin Marietta	36-41	2.535	3.93	47.9	4
SAND	20.0%	A85510	Ames South/Hallett Materials Co		2.594	1.35	40.0	4
Classified RAP	20.0%		20% 1RAP12-016 (5.3 % AC)		2.504	3.13	42.2	3

Job Mix Formula - Combined Gradation (Sieve Size in.)

	1"	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200
Upper Tolerance	100	100	100	96	74	50		25			6.6
	100	**100**	**94**	**89**	**67**	**45**	**32**	**21**	**9.6**	**5.8**	**4.6**
Lower Tolerance	100	100	87	82	60	40		17			2.6

Home

Asphalt Binder Source and Grade: Bituminous Matr'l & Supply (Tama, IA) PG 58-28

Gyratory Data

% Asphalt Binder	6.00	6.38	**6.98**	7.20	
Gmb @ N-Des.	2.275	2.280	**2.303**	2.290	
Max. Sp.Gr. (Gmm)	2.424	2.413	**2.399**	2.395	
% Gmm @ N- Initial	86.3	87.0	**88.5**	87.9	
%Gmm @ N-Max					
% Air Voids	6.1	5.5	**4.0**	4.4	
% VMA	15.9	16.1	**15.8**	16.5	
% VFA	61.4	65.8	**75.2**	73.4	
Film Thickness	8.87	9.58	**10.70**	10.88	
Filler Bit. Ratio	1.03	0.95	**0.85**	0.84	
Gsb	2.544	2.544	**2.544**	2.544	
Gse	2.654	2.656	**2.660**	2.670	
Pbe	4.42	4.78	**5.34**	5.43	
Pba	1.68	1.71	**1.76**	1.91	
% New Asphalt Binder	83.2	84.3	**85.7**	86.2	
Asphalt Binder Sp.Gr. @ 25°C	1.029	1.029	**1.029**	1.029	
% Water Abs	3.23	3.23	**3.23**	3.23	
S.A. m^2 / Kg.	4.99	4.99	**4.99**	4.99	
% +4 Type 4 Agg. Or Better	88.7	88.7	**88.7**	88.7	
% +4 Type 3 Agg.	0.6	0.6	**0.6**	0.6	
% -4 Type 2	0.0	0.0	**0.0**	0.0	
Fineness Modulus of Type 2	0.0	0.0	**0.0**	0.0	
Angularity-method A	41	41	**41**	41	
% Flat & Elongated	1.5	1.5	**1.5**	1.5	
Sand Equivalent	77	77	**77**	77	
Aggregate Type Used	A	A	**A**	A	
% Crushed	73	73	73	73	

Number of Gyrations
N-Initial
7
N-Design
76
N-Max
117

Gsb for Angularity
Method A
2.556

Pba / %Abs Ratio
0.55

Slope of Compaction
Curve

Mix Check
Fair

Pb Range Check
1.20

RAM Check
OK

Specification Check
Comply

Moisture Sensitivity Check
Not Required

Disposition : An asphalt content of 7.0% is recommended to start this project.

Data shown in 6.98% column is interpolated from test data.

The % ADD AC to start project is 6.0%

Figure A-38. Central Iowa Expo Center high-absorption HMA mix design.

Figure A-39. Manatts, Inc. plant at Ames, Iowa.

Plant and Mixture Production

The asphalt plant, a Cedar Rapids E 400 SL (Figure A-39), was a counter-flow drum. Although the plant was erected in 1995, the old drum was replaced with this new one in 2012. Previously, this plant was a batch plant. Due to the transformation from batch plant to drum plant, the drum has unusually high elevation. This natural gas–operated plant has a maximum capacity of 350 tons/hour mixture production. The dimensions of the drum were 48 ft (14.6 m) long and 100 in. (2.54 m) in diameter. The length of mixing zone and drying zone were 18 ft (5.5 m) and 30 ft (9.1 m), respectively. The plant had two Cedar Rapids silos, each with a capacity of 200 tons, and three horizontal binder tanks. The drag slat conveyor belt was heated/insulated and in good condition. The plant used six cold-feed bins, which were used for virgin aggregates and RAP. RAP materials were screened over a 2-in. (50-mm) screen before entering into the mixing drum. The plant had a conventional (cyclone) baghouse fines collection system, and part of the baghouse fines was reintroduced into the drum. Plant modification to accommodate foaming WMA was performed in 2013 using a unit supplied by Aqua Foam (Figure A-40).

Typically, the plant operator started the production at a higher temperature and lowered to the target mixing temperature within four to five truck loads. Binder temperature at storage tank was always maintained between 310°F to 312°F (154°C to 155°C). At the beginning of the project, the contractor started producing only hot mix; later it produced foaming WMA. Table A-19 presents the production sequence and temperature data for the particular production day and time when loose mix was collected. Due to some technical problems with baghouse fines collection system, the plant operator produced the high-temperature and high-absorption mixture at 10°F to 15°F higher than originally planned. Total tonnage for each of the surface mixtures varied between 300 and 500 tons. An illustration of the paving operation is presented in Figure A-41.

Construction

The average distance between the asphalt plant and the construction site was approximately 15 mi (24 km). The average hauling time was approximately 20 to 25 minutes. The mixtures were hauled using rear dump trucks with tarp on top. The rear dump trucks released the loose mix directly

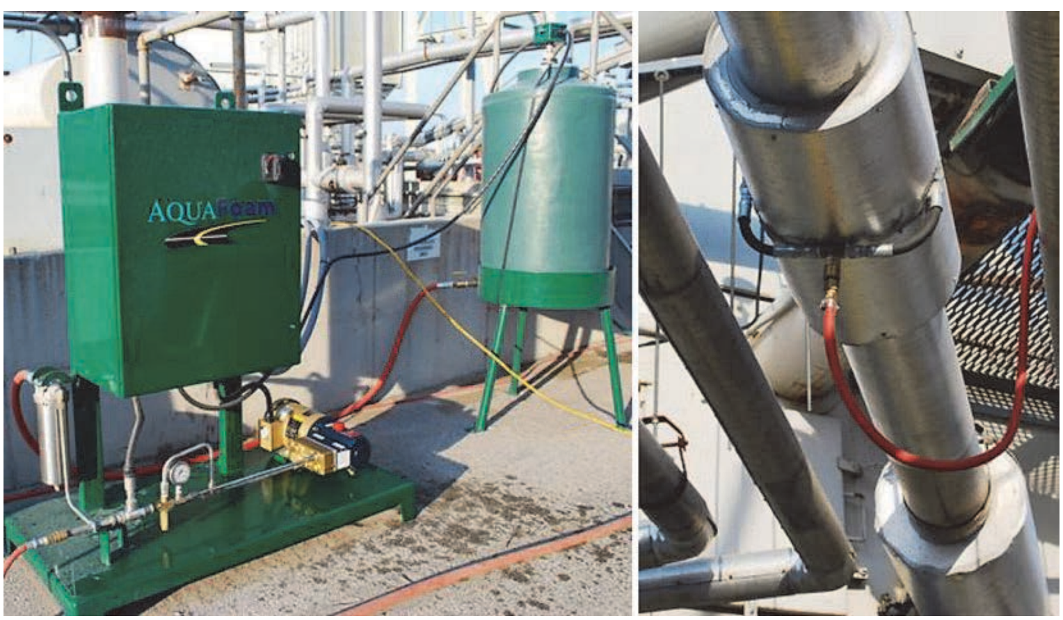

Figure A-40. Asphalt foaming system at Manatts, Inc. plant.

Table A-19. Production, paving, and ambient temperatures at the Iowa field site.

Aggregate	Mixture	Date of Production	Plant Mix Temp, °F	Paving Temp, °F	Ambient Temp, °F
Low-Absorptive Aggregates	WMA Control	06.28.13	260–265	250–260	70–73
	WMA Control + 30 °F	06.28.13	290–295	285–290	75–79
	HMA Control	06.28.13	290–295	285	80–85
	HMA Control + 30 °F	06.28.13	320–325	305	85–88
High-Absorptive Aggregates	WMA Control	07.01.13	260	250	63–67
	WMA Control + 30 °F	07.01.13	290	285	70–76
	HMA Control	07.01.13	290–295	290	78–80
	HMA Control + 30 °F	07.01.13	305–310	295	80–81

Figure A-41. Paving of surface layer at Central Iowa Expo Center.

into the paver hopper. This job used one standard-sized steel-wheeled roller and a small steel-wheeled finish roller. The breakdown roller compacted the loose mix using four to six passes in vibratory and static mode. The finish roller typically used three to four passes in static mode. Starting from the laydown by the paver to the compaction by finish roller, paving was completed within 15 to 20 minutes. Table A-20 shows the equipment used for laydown and compaction. Minimum mixture temperature reduction was achieved due to moderate ambient temperature and short hauling distance.

Prior to the placing of 2-in. (50-mm) thick surface layer, all the sections were paved with a 4-in. (100-mm) thick asphalt base layer using only foaming WMA low-absorption mixture produced at the control temperature (260°F [127°C]). The research noted the beginning and ending of each test section. Before paving, the contractor applied a CSS1H tack coat with mix ratio of 2:1 @ 0.05 gal/yd^2 (0.23 L/m^2) rates.

Sample Collection

Plant mix was collected from the truck at the plant. There was scaffolding at the rear end of the plant. Some of the trucks were routed next to the scaffolding for sampling after they were loaded with loose mix from the silo. Loose mix from the plant was collected for on-site specimen compaction and later use in the laboratory at TTI. The materials sampling scheme is presented in Table A-21. The research team collected straight PG 58-28, virgin aggregate, and two different RAP sources from the asphalt plant. The contractor obtained the road cores as part of the Iowa Department of Transportation's QA plan. After density measurement by the contractor, the research team obtained the cores for further testing at TTI's laboratory. All loose mix, binder, cores, aggregate, RAP, and in-site specimens were brought back to Texas using a freight company.

Table A-20. Paving equipment used at the Iowa field site.

Equipment Type	Manufacturer	Model
Paver	ROADTEC	RP 195
Breakdown-Intermediate Roller (Steel-wheeled)	HAMM	HD + 120 VV-HV
Finish Roller (Steel-wheeled)	HAMM	HD 12 VV

Table A-21. Materials sampling scheme at the Iowa field site.

Sample Type	Material	Point of Sampling
Lab Mixed, Lab Compacted	Fine Aggregate	Stockpile
	Coarse Aggregate	Stockpile
	RAP 1 and RAP 2	Stockpile
	PG 58-28 Asphalt	Terminal
Plant Mixed, Lab Compacted	Loose Mix	Truck at Plant
Plant Mixed, Field Compacted	Road Cores	Travel Lane (random locations)

PMPC Specimen Compaction

Sixty-four 6-in. (150-mm) diameter and 2.4-in. (61-mm) tall specimens were compacted using plant mix at nearby Iowa State University's civil engineering laboratory. The time required to haul the loose mix from the collection point (truck at the plant) to the Iowa State University laboratory was approximately 15 minutes. Plant loose mix collected at the truck at plant was quickly brought to the laboratory and placed in the oven to achieve the desired compaction temperature for that particular mixture. The time required from mixture production to specimen compaction varied between 1 and 2 hours for any given mixture. Specimens were compacted using a Troxler Superpave gyratory compactor to 7 ± 1 percent AV content.

Florida Interstate 95 Rest Area (FL Parking)

General Description

This section summarizes the mixture production and construction of four test sections for a field evaluation project near Jacksonville, Florida, in July 2013 to investigate the effect of short-term aging on HMA and WMA using low- and high-absorption aggregate materials. The test sections were constructed in the parking lot of the Florida Welcome Center on the southbound direction of I-95. The WMA technology used on this project was an asphalt foaming system incorporated in a unitized counter-flow drum developed by Astec Industries, also known as the Astec Double Barrel® Green (DBG) system. Four mixtures (two HMA and two WMA) were produced using two different aggregate sources, including granite (low absorption) and limestone (high absorption). The granite HMA and WMA mixtures were paved on July 17, and the limestone HMA and WMA mixtures were paved on July 18. The contractor for this project was Duval Asphalt whose plant was located in Jacksonville, Florida.

Mix Designs

The mixtures produced for this project were based on two 0.5-in. (12.5-mm) NMAS Superpave mix designs, with a design compaction effort of 75 gyrations. The first mix design used Martin Marietta granite aggregates from Georgia, and the other mix design used Cemex limestone aggregates shipped in from Miami, Florida, specifically for this project. Each design used 25 percent RAP. The RAP was from multiple sources and crushed to 0.5 in. (12.7 mm). The base binder used in each mixture was a PG 58-22 binder supplied by Marathon Oil in Jacksonville, Florida. A liquid anti-strip, LOF 6500, supplied by ArrMaz was used at 0.5 percent by weight of the binder as an anti-stripping agent for the two mix designs.

Each mix design was produced first at HMA temperatures then at WMA temperatures using the Astec DBG foaming system at 2 percent water by weight of the binder. The material percentages in the mix design and production for each mixture are shown in Tables A-22 and A-23. The same percentages were used throughout for both granite mixtures. A slight aggregate change was made about halfway through production of the limestone HMA, and the change was kept for the rest of the HMA production and the entirety of the WMA limestone production. Table A-24 summarizes the design aggregate gradation, design volumetric properties, specifications, and allowable tolerances.

Table A-22. Aggregate percentages for binder content for granite mixtures.

Aggregate Type	Mix Design, %	Production	
		HMA, %	WMA, %
#7 Granite	20	23	23
#89 Granite	25	18	18
W-10 Screenings	17	28	28
M-10 Screenings	8	0	0
Local Sand	5	6	6
−0.5" Crushed RAP	25	25	25
Total AC	5.1	5.0	5.0

Table A-23. Aggregate percentages and binder content for limestone mixtures.

Aggregate Type	Mix Design, %	Production	
		1st Part of HMA Production, %	2nd Part of HMA Production and All of WMA, %
#7 Limestone	20	20	21
#89 Limestone	18	20	20
Screenings	37	35	34
−1/2" Crushed RAP	25	25	25
Total AC	6.8	6.7	6.7

Table A-24. Mix design gradation, asphalt content, and volumetrics for Florida mixtures.

Sieve Size mm	JMF Granite	Control Points Granite	JMF Limestone	Control Points Limestone
19.0 (3/4")	100	100	100	100
12.5 (1/2")	97	89–100	99	89–100
9.5 (3/8")	88	Max. 89	88	Max. 89
4.75 (#4)	60	—	66	—
2.36 (#8)	43	28–58	52	28–58
1.18 (#16)	34	—	36	—
0.6 (#30)	28	—	25	—
0.3 (#50)	22	—	17	—
0.15 (#100)	13	—	7	—
0.075 (#200)	4.5	2–10	4.5	2–10
Asphalt Content, %	5.1	—	4.0	—
AV, %	4	—	14.5	—
VMA, %	14.0	—	72.0	—
VFA, %	71.0	—	0.9	—
D/A Ratio	1.0	—	4.0	—

Production

The Astec DBG plant (Figure A-42) used to produce the mixtures for this field evaluation was located in Jacksonville, Florida, approximately 30 mi from the site. Figure A-43 shows the Astec DBG multiple-nozzle foaming manifold used at the plant.

The production of the granite HMA began at about 8:30 a.m. on July 17 and lasted until approximately 11:00 a.m. The production of the granite WMA began at about 12:30 p.m. the same day and lasted until approximately 2:30 p.m. The production of the limestone HMA began the next day on July 18 at about 7:30 a.m. and lasted until approximately 10:30 a.m. The production of the limestone WMA began about 12:00 p.m. and lasted until approximately 2:00 p.m. that same day. Table A-25 summarizes the production temperatures.

Figure A-42. Duval asphalt DBG drum plant in Jacksonville, Florida.

Mixture Properties

The mixtures were sampled at the plant to fabricate specimens in the NCAT mobile laboratory on site and in the main NCAT laboratory for determining mixture volumetric and performance properties.

Moisture Content and Volumetric Properties

The moisture content of plant-produced loose mix was determined based on two samples for each mixture in accordance with AASHTO T 329, "Moisture Content of HMA by Oven Method." Each sample was approximately 1000 g. The samples were heated to a constant mass (less than 0.05 percent change), as specified in AASHTO T 329.

The average moisture contents were 0.06 percent and 0.10 percent for the HMA and WMA using granite and

Figure A-43. DBG multiple-nozzle foaming manifold used in Jacksonville, Florida.

Table A-25. Production temperatures for Florida mixtures.

Temperature	HMA Granite	WMA Granite	HMA Limestone	WMA Limestone
Average, °F	306.1	272.4	307.8	267.1
Standard Deviation, °F	4.9	5.6	9.0	12.2

0.0 percent and 0.04 percent for the HMA and WMA mixtures using limestone, respectively. Specimens were compacted to the N_{design} (75 gyrations) in the Superpave gyratory compactor. These volumetric samples were plant mixed and lab compacted on site in the NCAT mobile laboratory so that the mixtures would not have to be reheated, which has been shown to affect asphalt absorption and other volumetric properties. The NCAT mobile laboratory on site is shown in Figure A-44. The samples were placed in an oven for a short time after sampling only to get back up to the compaction temperature. Water absorption levels were low (<2 percent); therefore, bulk specific gravity (G_{mb}) was determined in accordance with AASHTO T 166. The mixtures were also brought back to the main NCAT laboratory where the asphalt content and gradation of each mixture were tested according to AASHTO T 308 and AASHTO T 30, respectively. The results from this testing are shown in Tables A-26 and A-27 for the mixtures with granite and limestone.

Figure A-44. NCAT mobile laboratory on site in Jacksonville, Florida.

Construction

The HMA and WMA mixtures were placed in the parking lot of the Florida Welcome Center on the southbound direction of I-95. The plant used for this field evaluation was an Astec DBG plant located in Jacksonville, Florida. The plant was approximately 30 mi from the site, and the haul time was 40 minutes to 1 hour. Figure A-45 shows the test section layout and Figure A-46 shows the paving site in relation to the plant.

The paver used for this project was a RoadTec RP190, as shown in Figure A-47. The temperature of the mixture was measured every 5 to 20 minutes both in the auger and behind

Table A-26. Design gradation, asphalt content, and volumetrics for plant-produced granite mixtures.

Property	JMF Granite	HMA Granite	WMA Granite	Control Points
Sieve Size		% Passing		
19.0 mm (3/4")	100	100.0	100.0	100
12.5 mm (1/2")	97	98.9	99.1	89–100
9.5 mm (3/8")	88	86.9	88.4	Max. 89
4.75 mm (#4)	60	55.6	58.0	—
2.36 mm (#8)	43	40.5	41.5	28-58
1.18 mm (#16)	34	31.7	32.2	—
0.60 mm (#30)	28	26.5	26.9	—
0.30 mm (#50)	22	22.0	22.1	—
0.15 mm (#100)	13	12.1	11.7	—
0.075 mm (#200)	4.5	5.6	5.1	2–10
Asphalt Content (%)	5.10	4.50	4.67	—
G_{mm}	2.508	2.537	2.548	—
G_{mb}	2.408	2.442	2.439	—
AV (%)	4.0	3.7	4.3	—
VMA (%)	14.0	12.9	13.1	—
VFA (%)	71.0	71.0	67.5	—
Dust/Binder	1.0	1.45	1.36	—
P_{ba} (%)	0.57	0.67	0.97	—
P_{be} (%)	4.30	3.85	3.74	—

Table A-27. Design gradation, asphalt content, and volumetrics for plant-produced limestone mixtures.

Property	JMF Limestone	HMA Limestone	WMA Limestone	Control Points
Sieve Size		% Passing		
19.0 mm (3/4")	100	100.0	100.0	100
12.5 mm (1/2")	99	99.9	99.5	89–100
9.5 mm (3/8")	88	90.7	89.2	Max. 89
4.75 mm (#4)	66	67.1	63.8	—
2.36 mm (#8)	52	52.6	50.2	28–58
1.18 mm (#16)	36	38.5	37.4	—
0.60 mm (#30)	25	29.2	28.5	—
0.30 mm (#50)	17	21.2	20.6	—
0.15 mm (#100)	7	11.0	10.7	—
0.075 mm (#200)	4.5	5.9	5.8	2–10
Asphalt Content (%)	6.80	6.56	6.41	—
G_{mm}	2.342	2.350	2.363	—
G_{mb}	2.249	2.266	2.271	—
AV (%)	4.0	3.6	3.9	—
VMA (%)	14.5	14.1	13.8	—
VFA (%)	72.0	74.7	71.9	—
Dust/Binder	0.9	1.23	1.29	—
P_{ba} (%)	1.65	1.88	2.04	—
P_{be} (%)	4.80	4.80	4.50	—

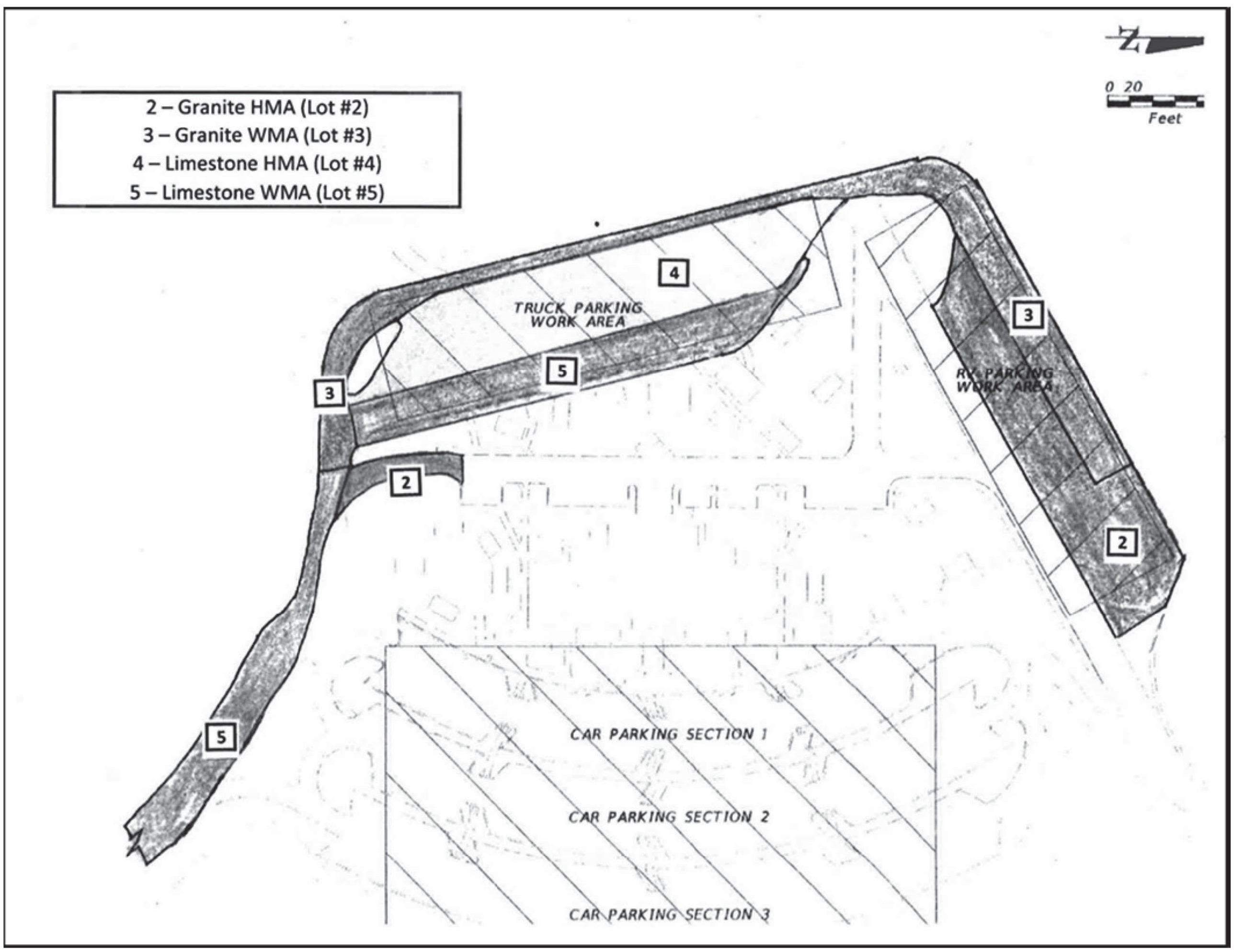

Figure A-45. Layout of test sections in Jacksonville, Florida.

Figure A-46. Location of plant and paving site in Jacksonville, Florida.

Figure A-47. RoadTec paver used in Jacksonville, Florida.

Figure A-48. Breakdown roller compacting granite HMA.

the paver with a handheld temperature gun. Tables A-28 and A-29 show the temperatures of the mixture in the auger and behind the screed, respectively.

Both granite mixtures were compacted using two breakdown rollers in tandem operating in static mode. The rolling pattern used was seven passes on each side of the mat in the static mode. A finishing roller was not used for the granite mixtures. For both limestone mixtures, the same breakdown rollers were used, but only five passes in the static mode were needed to achieve density. A rubber tire roller was used to roll each joint once on the limestone mixtures. This was done an hour or more after placement, once the mixture had cooled significantly, and according to the contractor, it was done mainly for aesthetic purposes. Figure A-48 shows one of the breakdown rollers compacting the granite HMA mixture. Figures A-49 and A-50 show examples of the compacted mat for the granite and limestone mixtures, respectively.

Weather data were collected hourly at the paving location using a handheld weather station. There was no rain during the construction of either mixture. Table A-30 shows the ambient temperatures, wind speed, and humidity for both mixtures produced.

Field Cores for Further Testing

After construction, ten 6-in. (150-mm) cores were obtained from each test section. These cores were taken back to the laboratory, and the density of the surface layer was determined after trimming from the underlying layers. The densities were determined in accordance with AASHTO T 166. If the water absorption was higher than 2 percent, the samples

Table A-28. Temperatures of mixture in the auger in Jacksonville, Florida.

Mixture Type	HMA Granite	WMA Granite	HMA Limestone	WMA Limestone
Temperature, °F. Avg.	278.6	251.8	278.6	241.4
Std. Dev.	8.1	9.0	8.1	8.0
Minimum	253.5	233.5	253.5	226.0
Maximum	297.0	266.5	297.0	255.5

Table A-29. Temperatures of mixture behind screed in Jacksonville, Florida.

Mixture Type	HMA Granite	WMA Granite	HMA Limestone	WMA Limestone
Temperature, °F. Avg.	276.6	251.6	276.6	242.9
Std. Dev.	9.9	6.4	9.9	9.0
Minimum	260.5	241.5	260.5	227.0
Maximum	296.5	259.5	296.5	258.5

Figure A-49. Example of compacted granite mixture mat.

Figure A-50. Example of compacted limestone mixture mat.

were then tested according to AASHTO T 331. Average test results are shown in Table A-31. Average core densities were virtually the same for both mixtures, with the WMA averaging just slightly denser results. This was expected since it has often been observed that WMA technologies can provide increased compactability compared to HMA, even at lower temperatures.

City of Indianapolis, Residential Streets (IN Residential)

General Description

This report documents the production, construction, and materials properties for a field evaluation site in Indianapolis, Indiana. The test site consisted of four test sections constructed with four mixtures based on the same mix design but produced at hot and warm mix temperatures by two plants, a modified batch plant and a modified counter-flow drum plant. The four test sections were constructed on several residential city streets on the west side of Indianapolis beginning at the intersection of North Tibbs Avenue and West 21st Street from August 19 through 22, 2013. The contractor was Rieth-Riley Construction, Inc., and its two plants were located at Rieth-Riley's Kentucky Avenue facility in Indianapolis.

In the first 2 days of production, the batch plant and then the drum plant were used to produce two HMA mixtures. On the third day of production, the modified batch plant was used to produce a WMA mixture using WMA additive Advera at a rate of 0.25 percent by weight of total mixture (5 lb/ton). On the fourth day of production, the Astec single counter-flow barrel drum plant was used to produce another

Table A-30. Weather conditions during construction in Jacksonville, Florida.

Mixture	HMA Granite	WMA Granite	HMA Limestone	WMA Limestone
Ambient Avg. Temp. (°F)	92.6	96.4	92.4	90.9
Range	84.6–98.7	93.1–100.0	89.9–95.1	90.5–91.2
Wind Speed (mph)	1.7	2.13	1.3	1.3
Range	1.0–3.0	1.5–3.4	1.0–2.1	0.6–1.7
Humidity, %	55.0	48.2	52.6	52.4
Range	46.0–92.6	41.3–52.9	49.4–56.4	52.0–52.6

Table A-31. In-place density from cores in Jacksonville, Florida

Test	Statistic	HMA Granite	WMA Granite	HMA Limestone	WMA Limestone
In-place Density (%)	Average	93.4	93.1	92.1	91.7
	Standard Deviation	1.0	0.8	1.1	0.9

Table A-32. Aggregate and binder percentages used in mix design and production for Indianapolis field site.

Aggregate Type	Mix Design, %	Production, %
#11 Limestone	33.0	33.5
Manufactured Sand	30.5	30.9
Natural Sand	10.0	10.1
Crushed RAP	25.0	25.5
Baghouse Fines	1.5	Return 100%
Total AC	5.8	5.8

Table A-33. Design gradation, asphalt content, and volumetrics for Indianapolis mix design.

Sieve Size	JMF	Control Points
19.0 mm (3/4")	100	100
12.5 mm (1/2")	100	100
9.5 mm (3/8")	96.8	100
4.75 mm (#4)	68.3	<90
2.36 mm (#8)	46.3	32–67
1.18 mm (#16)	29.7	—
0.6 mm (#30)	19.2	—
0.3 mm (#50)	10.5	—
0.15 mm (#100)	6.4	—
0.075 mm (#200)	5.1	2–10
AC, %	5.8	—
AV, %	4	—
VMA, %	15.8	—
VFA, %	74.7	—
D/A Ratio	1.0	—
P_{be}	5.2	—

WMA using the foaming technology AquaFoam® at 2 percent by weight of the binder. All the mixtures were produced based on the same mix design.

Mix Designs

The asphalt mixtures were produced based on a 0.37 in. (9.5 mm) NMAS Superpave mix design, with a design compaction effort of 75 gyrations. The mix design contained 25 percent RAP crushed to 0.5 in. (12.7 mm). The base binder used for all mixtures was a PG 64-22 supplied by Interstate Asphalt out of Chicago and Peoria, Illinois. No anti-strip agent was used for any of the mixtures produced. The aggregate consisted of limestone and natural sand from the Martin Marietta quarry next to Rieth-Riley's facility. The same material percentages were used throughout production of all four mixtures. Table A-32 shows the design and production percentages. The design aggregate gradation, optimum asphalt content, design volumetrics, specifications, and allowable tolerances are shown in Table A-33.

Production

Two different plants were used to produce the four mixtures evaluated in this project, a modified batch plant and a counter-flow drum plant. Both of them were located at Rieth-Riley's Kentucky Avenue facility in Indianapolis, Indiana. The distance to the site was approximately 7 mi. The batch and drum plants can be seen in Figures A-51 and A-52, respectively.

Production of the HMA mix using the batch plant began on August 19, 2013, at about 6:40 a.m. and lasted until approximately 6:00 p.m. The total production was approximately 1,250 tons. Production of the HMA using the drum plant started on August 20 at approximately 7:00 a.m. and lasted until 4:00 p.m. The total mixture production this day was approximately 760 tons.

Figure A-51. Rieth-Riley's modified batch plant in Indianapolis, Indiana.

Figure A-52. Rieth-Riley's counter-flow drum plant in Indianapolis, Indiana.

On the third day of production, August 21, a WMA mix was produced using the modified batch plant with the WMA additive Advera. Mixture production started at approximately 7:20 a.m. and lasted until 11:00 a.m. The total mixture production that day was approximately 770 tons. On the last day of production, August 22, the mixture was produced using the Astec single counter-flow barrel drum plant with the WMA foaming technology AquaFoam. The production started at approximately 7:20 a.m. and ended at about 4:30 p.m. A total of 1,000 tons was produced that day. Table A-34 summarizes the production temperature data for the four mixtures.

Mixture Properties

Each mixture was sampled during production and then used to fabricate a variety of specimens for determining moisture, volumetric, and performance properties in both the NCAT mobile laboratory and main laboratory.

AASHTO T 329, "Moisture Content of HMA by Oven Method," was followed to evaluate the moisture content of loose plant-produced mix (two samples per mixture). Each sample was approximately 1000 g. The samples were heated to a constant mass (less than 0.05 percent change), as defined by AASHTO T 329.

The average moisture contents were 0.11 percent and 0.10 percent for the HMA and WMA that were produced using the batch plant and 0.08 percent for the HMA and WMA mixtures produced by the drum plant.

Figure A-53. NCAT mobile laboratory on site in Indianapolis, Indiana.

Volumetric specimens were PMLC on site in the NCAT mobile laboratory (Figure A-53) so that the mixtures would not have to be reheated, which has been shown to affect asphalt absorption and other volumetric properties. The specimens were compacted to 75 gyrations in the Superpave gyratory compactor. The samples were placed in an oven for a short time after sampling only to get back up to the compaction temperature. Water absorption levels were low (<2 percent), therefore bulk specific gravity (G_{mb}) was determined in accordance with AASHTO T 166. The mixtures were also brought back to the main NCAT laboratory where the asphalt content and gradation of each mixture were tested according to AASHTO T 308 and AASHTO T 30, respectively. The results from this testing are shown in Table A-35.

Construction

The HMA and WMA mixtures were placed in several residential city streets on the west side of Indianapolis beginning at the intersection of North Tibbs Avenue and West 21st Street. The paving site was approximately 7 mi from the plant. With traffic, the haul distance was approximately 15 to 25 minutes. Figure A-54 shows the test section layout and Figure A-55 shows the paving site in relation to the plant.

The paver used for this project was a RoadTec RP175, as shown in Figure A-56. The target pavement thickness was 1.5 to 2.0 in. (38 to 50 mm). The temperature of the mixture was

Table A-34. Production temperatures for Indianapolis field site.

Temperature	Batch HMA	Batch WMA	Drum HMA	Drum WMA
Average, °F	305.4	273.2	300.4	271.4
Standard Deviation, °F	8.6	15.9	9.3	5.5

Table A-35. Gradation, asphalt content, and volumetrics for plant-produced mixtures in Indianapolis, Indiana.

Property	JMF	HMA Batch	WMA Batch	HMA Drum	WMA Drum	Control Points
Sieve Size	% Passing					
19.0 mm (3/4")	100.0	100.0	100.0	100.0	100.0	100
12.5 mm (1/2")	100.0	100.0	100.0	100.0	100.0	100
9.5 mm (3/8")	96.8	97.0	96.8	97.7	97.0	100
4.75 mm (#4)	68.3	66.1	66.3	73.3	69.3	<90
2.36 mm (#8)	46.3	44.7	45.9	49.2	43.8	32–67
1.18 mm (#16)	29.7	29.9	31.5	31.1	27.6	—
0.60 mm (#30)	19.2	20.3	21.8	19.8	17.5	—
0.30 mm (#50)	10.5	12.1	13.4	11.7	10.2	—
0.15 mm (#100)	6.4	8.2	9.2	8.3	7.1	—
0.075 mm (#200)	5.1	5.9	6.8	6.1	5.1	2–10
Asphalt Content (%)	5.8	6.00	6.05	6.39	6.07	—
G_{mm}	2.452	2.451	2.448	2.446	2.455	—
G_{mb}	2.354	2.390	2.406	2.368	2.348	—
AV (%)	4.0	2.5	1.7	3.2	4.4	—
VMA (%)	15.8	13.3	12.8	14.5	14.9	—
VFA (%)	74.7	81.2	86.5	78.0	70.8	—
Dust/Binder	1.0	1.26	1.43	1.24	1.10	—
P_{ba} (%)	0.69	1.41	1.39	1.58	1.53	—
P_{be} (%)	5.15	4.67	4.74	4.91	4.63	—

Figure A-54. Layout of test sections in Indianapolis, Indiana.

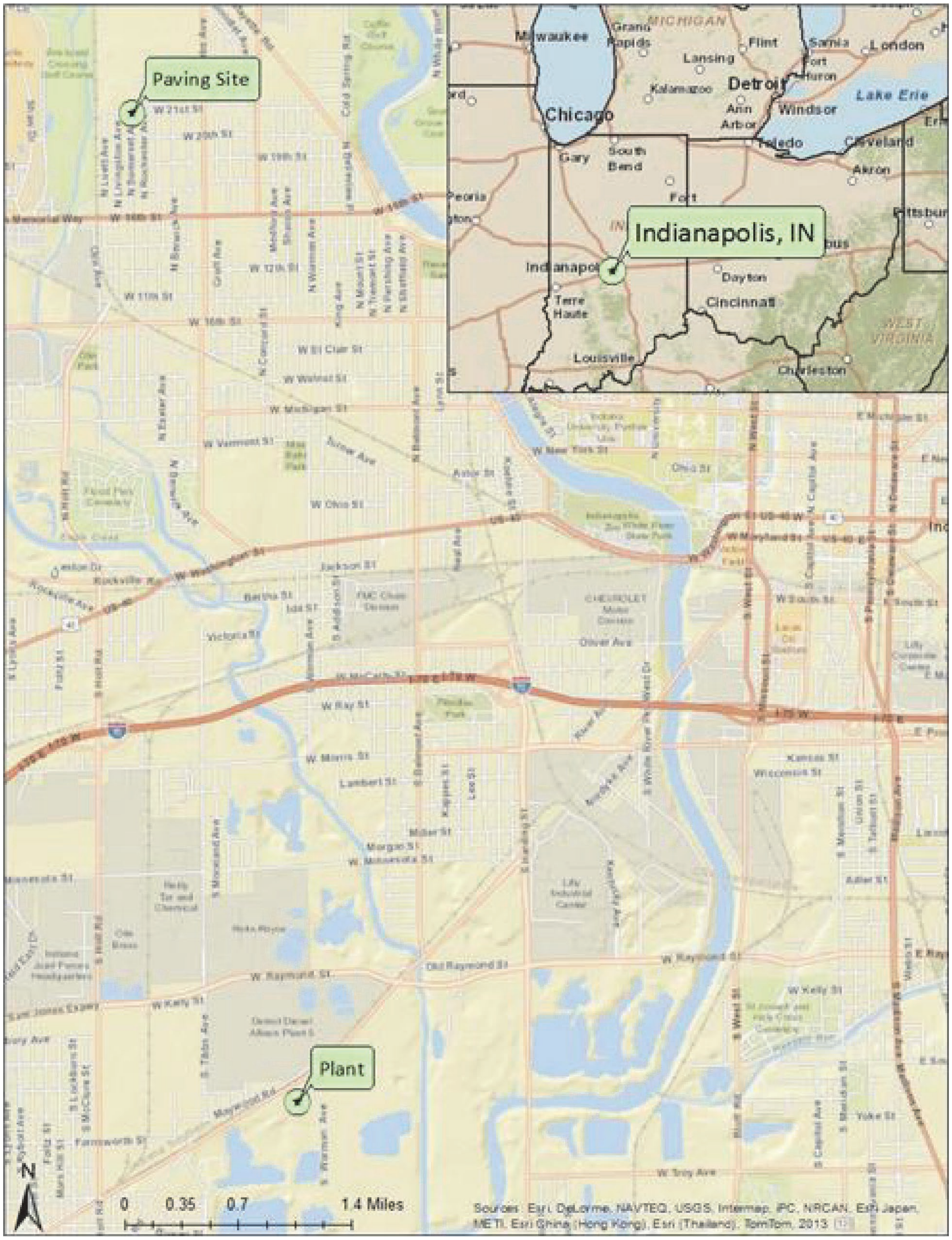

Figure A-55. Locations of plant and paving site in Indianapolis, Indiana.

Figure A-56. RoadTec paver used in Indianapolis, Indiana.

Figure A-57. Breakdown and finishing rollers compacting the HMA drum mix.

measured every 10 to 30 minutes both in the auger and behind the paver with a handheld temperature gun. Tables A-36 and A-37 show the temperatures of the mixture in the auger and behind the screed, respectively.

All the mixtures were compacted using both a breakdown and finishing roller. Figure A-57 shows the two rollers compacting the HMA mixture produced by the drum plant. Figures A-58 and A-59 show examples of the compacted mat for the HMA and WMA mixtures produced using the batch plant.

Weather data were collected hourly at the paving location using a handheld weather station. Table A-38 shows the ambient temperatures, wind speed, and humidity for both mixtures produced.

Field Cores for Further Testing

After construction, ten 6-in. (150-mm) cores were obtained from each mixture section. These cores were taken back to the laboratory and the density of the surface layer was determined after trimming from the underlying layers. The densities were determined in accordance with AASHTO T 166. If

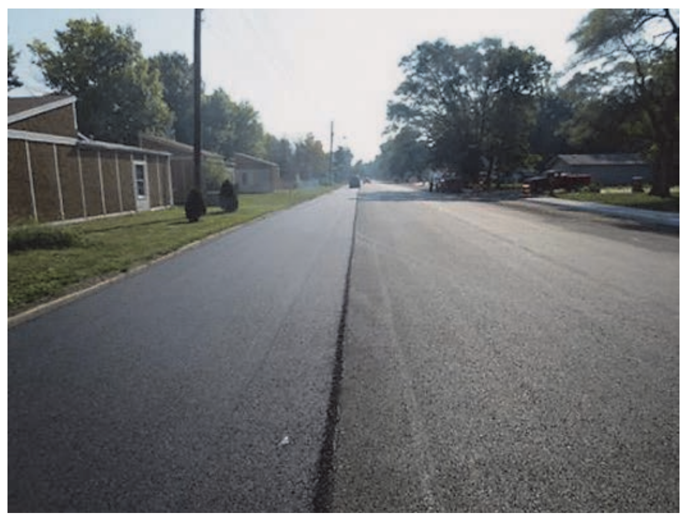

Figure A-58. Example of compacted HMA (batch plant).

Table A-36. Temperatures of mixture in the auger in Indianapolis, Indiana.

Temperature	Batch HMA	Batch WMA	Drum HMA	Drum WMA
Average, °F	287.3	246.0	269.5	251.5
Standard Deviation, °F	12.8	9.2	8.5	5.7

Table A-37. Temperatures behind the screed in Indianapolis, Indiana.

Temperature	Batch HMA	Batch WMA	Drum HMA	Drum WMA
Average, °F	283.9	243.1	265.5	250.1
Standard Deviation, °F	10.9	9.9	8.5	5.9

Figure A-59. Example of compacted WMA (batch plant).

Figure A-60. Midland–Odessa test sections.

the water absorption was higher than 2 percent, the samples were then tested according to AASHTO T 331. Average test results are shown in Table A-39. Average core densities were virtually the same for both mixtures, with the WMA averaging just slightly denser results. This was expected since it has often been observed that WMA technologies can provide increased compactability compared to HMA, even at lower temperatures.

Midland–Odessa City Street (TX II Local)

General Description

Midland–Odessa, Texas, test sections were designed to examine the effects of binder sources and plant types on the aging of HMA. This section was located on a recently built city street for a new subdivision in Odessa, Texas. Reece Albert, Inc., a general construction contractor, produced the mixtures and paved the street. At researchers' request, the contractor agreed to use asphalt from two different binder sources and two different HMA plants. The street was paved in the middle of April 2014. This curb-and-gutter type city street has two lanes in each direction with a turn lane at the center. Including the shoulder, the total width of paving was 61 ft (18.6 m). Four subsections (two binders × two plants), each with approximately 1200 ft (366 m) in length, were laid side by side. Figure A-60 shows the layout of the test sections.

Mixtures and Materials

Only one type of aggregate gradation was used for all four mixtures. This project used TxDOT's Type D (equivalent to 0.37-in. [9.5-mm] NMAS aggregate size) mixture designed with limestone aggregate from nearby Parks Bell quarry. The contractor laid mixtures with this same design for several other projects in the past. Figure A-61 presents the summary of mix design. The mixture was designed using Texas gyratory

Table A-38. Weather conditions during construction in Indianapolis, Indiana.

Mixture	Batch HMA	Batch WMA	Drum HMA	Drum WMA
Ambient Avg. Temp. (°F)	87.0	87.3	88.3	87
Range	72–93	77.5–94.8	78.5–95.7	74.2–93.7
Wind Speed (mph)	0.4	0.6	1.03	0.5
Range	0–1.8	0–1.3	0–2.7	0–1.2
Humidity, %	42.3	52.9	44.1	53.9
Range	30–68.5	37.7–71.1	36.5–63.8	44.2–75.8

Table A-39. In-place density from cores in Indianapolis, Indiana.

Test	Statistic	Batch HMA	Batch WMA	Drum HMA	Drum WMA
In-place Density (%)	Average	90.1	91.3	90.9	90.3
	Standard Deviation	1.77	1.50	2.16	0.66

HMACP MIXTURE DESIGN : COMBINED GRADATION

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SAMPLE ID:	City Streets	SAMPLE DATE:	8-22-12
LOT NUMBER:		LETTING DATE:	
STATUS:	Tubbs Pit Mix designs	CONTROLLING CSJ:	
COUNTY:	Midland	SPEC YEAR:	1993
SAMPLED BY:	Jose A. Mata #711	SPEC ITEM:	
SAMPLE LOCATION:	Lab	SPECIAL PROVISION:	
MATERIAL:		MIX TYPE:	Type_D
PRODUCER:	CSA Materials Inc.		
AREA ENGINEER:		PROJECT MANAGER:	
COURSE\LIFT:		STATION:	
DIST. FROM CL:			

BIN FRACTIONS	Bin No.1		Bin No.2		Bin No.3		Bin No.4		Bin No.5		Bin No.6		Bin No.7		Combined Gradation
Aggregate Source:	Parks Bell		Parks Bell		Parks Bell Pit										
Aggregate Number:	TY-D		TY-FF		Screenings										
Sample ID:															
Rap: (Yes/No), % Asphalt:															Total Bin
Individual Bin (%):	33.0	Percent	25.0	Percent	42.0	Percent		Percent		Percent		Percent		Percent	100.0%

Sieve Size:	Sieve Size: (mm)	Bin 1 Cum.% Passing	Bin 1 Wtd Cum. %	Bin 2 Cum.% Passing	Bin 2 Wtd Cum. %	Bin 3 Cum.% Passing	Bin 3 Wtd Cum. %	Bin 4 Cum.% Passing	Bin 4 Wtd Cum. %	Bin 5 Cum.% Passing	Bin 5 Wtd Cum. %	Bin 6 Cum.% Passing	Bin 6 Wtd Cum. %	Bin 7 Cum.% Passing	Bin 7 Wtd Cum. %	Cum. % Passing	Within Spec's	Lower & Upper Type_D Specification Limits (Lower)	(Upper)	Individual % Retained	Cumulative % Retained	Sieve Size
1/2"	12.500	100.0	33.0	100.0	25.0	100.0	42.0		0.0		0.0		0.0		0.0	100.0	Yes	98.0	100.0	0.0	0.0	1/2"
3/8"	9.500	96.8	31.9	100.0	25.0	100.0	42.0		0.0		0.0		0.0		0.0	98.9	Yes	85.0	100.0	1.1	1.1	3/8"
No. 4	4.750	1.5	0.5	78.4	19.6	99.7	41.9		0.0		0.0		0.0		0.0	62.0	Yes	50.0	80.0	36.9	38.0	No. 4
No. 10	2.000	1.2	0.4	2.5	0.6	85.0	35.7		0.0		0.0		0.0		0.0	36.7	Yes	32.0	42.0	25.3	63.3	No. 10
No. 40	0.425	1.1	0.4	2.1	0.5	39.2	16.5		0.0		0.0		0.0		0.0	17.4	Yes	11.0	26.0	19.3	82.6	No. 40
No. 80	0.180	1.1	0.4	2.0	0.5	25.3	10.6		0.0		0.0		0.0		0.0	11.5	Yes	4.0	15.0	5.9	88.5	No. 80
No. 200	0.075	0.9	0.3	1.8	0.5	12.0	5.0		0.0		0.0		0.0		0.0	5.8	Yes	1.0	6.0	5.7	94.2	No. 200
	0.000		0.0		0.0		0.0		0.0		0.0		0.0		0.0	0.0	No			5.8	100.0	
	0.000		0.0		0.0		0.0		0.0		0.0		0.0		0.0	0.0	No			0.0	100.0	

\# Not within specifications # Not cumulative

Asphalt Source & Grade:	PG 64-22	Binder Percent, (%):	6.2	1.030

Remarks

Figure A-61. Mix design used in Midland–Odessa field site.

Figure A-62. Midland–Odessa batch plant.

Figure A-63. Midland–Odessa drum plant.

compactor. The design asphalt content was 6.2 percent. The mixture did not use hydrated lime or any other admixture. The project used PG 64-22 unmodified binder from two different sources. The sources were Binder A refinery located at Big Spring in West Texas and Binder V refinery located at Sunray, Texas. Binder A refinery is approximately 60 miles from the project site and uses crude oil from a west Texas source. Binder V refinery at Sunray is located in the northwest Texas panhandle area.

Plants Description

As part of the experimental design, the contractor used two HMA plants and two different aggregate sources, keeping other variables same. Albert Reece, Inc. owns both plants. Figures A-62 and A-63 show the batch plant and drum plant, respectively.

Batch Plant

The batch plant was located approximately 25 mi from the job site. The hauling time was between 30 and 35 minutes. The batch plant was older—manufactured in 1957. The production capacity of this plant is 100 tons/hour with a 2.5-ton batch size. It had a 200-ton capacity silo. The plant also had two horizontal binder tans. It has a draft type emission-control system.

Drum Plant

Figure 63 shows the portable drum plant used in this project. This plant was located only 7 mi or 12 minutes from the project site. It was a counter-flow drum with a production capacity of 400 tons/hour. The model of the plant is CMI STD 400 triple drum. This 20-year-old plant had a 9-ft-diameter and 44-ft-long drum. The drum's mixing zone was 12 ft long. This plant had a conventional bag house emission-control system. A partial amount of fines were returned back to the drum during production. It ran about 175 tons/hour production rate. Instead of a silo, the plant had a 50-ton capacity surge bin.

Mixture Production

For a given binder source, each plant produced approximately 200 to 225 tons. Production started at slightly higher temperatures at the beginning. The silo storage time of batch plant-produced mixture was about 30 to 40 minutes. The silo (surge bin) storage time of the drum plant was approximately 10 to 15 minutes. Each of the 2 days of mixture production began with the batch plant followed by the drum plant. Belly dump trucks used for hauling mixtures had tarp to cover the mixtures. Table A-40 presents the mixtures production summary.

Construction

This asphalt mixture layer was placed directly on top of a freshly constructed flexible base layer. The 6- to 8-in. (15- to 20-cm) thick flexible base layer was prime-coated a few days before the HMA was placed. Once the belly dump trucks released the mixtures on the road, a windrow elevator picked up the mixtures and dropped them into the paver chute as shown in Figure A-64. The paving width and thickness of the sections

Table A-40. Production, paving, and ambient temperatures at the Midland–Odessa field site.

Mixture	Binder Source	Plant Type	Date of Production	Plant Mix Temp, °F	Paving Temp, °F	Ambient Temp, °F
BPA	Binder A	Batch	04/16/14	325–330	270–280	52–60
DPA	Binder A	Drum	04/16/14	325–330	270–280	60–65
BPV	Binder V	Batch	04/17/14	325–330	280–290	70–75
DPV	Binder V	Drum	04/17/14	325–330	280–285	75–78

Figure A-64. Paving with windrow operation at the Midland–Odessa field site.

were 15 ft (4.6 m) and 2 in. (50 mm), respectively. Table A-41 provides the list of paving equipment used during HMA layer construction.

The vibratory steel-wheeled roller immediately followed the paver. The pneumatic roller followed the vibratory roller once the mat temperature was approximately 180°F (82°C). The contractor used a small 4-ft (1.2-m) wide steel-wheel finish roller at the end. Typically, the vibratory roller made two vibratory passes followed by one static pass. The pneumatic roller made five passes.

Sample Collection

Plant mix was collected from the trucks at plants by climbing on scaffolding. Figures A-65 and A-66 show the sample collection at batch and drum plant, respectively. Samples were collected usually from the fifth or sixth truck when the plant temperatures become stable. In each case, loose samples were split as shown in Figure A-65. The materials sampling scheme is presented in Table A-42.

The mixture sample collected from the plant was immediately brought back to the FHWA mobile laboratory located within the drum plant facility for compaction into two specimens of two sizes (6.0 × 2.4 in. [150 × 61 mm], 6.0 × 7.0 in. [150 × 175 mm]). The FHWA mobile laboratory crew helped the research team by collecting both binders from the transport truck while they were delivering binders at the drum plant facility. With the help of the paving contractor, the research team also collected thirty-two 6-in. (150-mm) diameter road cores from four test sections. Road cores were obtained from the

Table A-41. Paving equipment used at the Midland–Odessa field site.

Equipment Type	Manufacturer	Model
Windrow Elevator	Lincoln	660 AXL
Paver	Vogel (Wirtgen)	5203-2i
Vibratory Steel-Wheeled Roller	HAMM AG	HD + 140 VO
Pneumatic Roller	Caterpillar	CAT 5378
Steel-Wheel Finish Roller (4 ft)		

Figure A-65. Loose mix sampling from truck in Midland–Odessa batch plant.

Figure A-66. Loose mix sampling from truck in Midland–Odessa drum plant.

center of the 15-ft. (4.6-m) wide paving mat at 80-ft. (24.4-m) intervals excluding the first 400 ft. (122 m) of paving.

Field Specimen Compaction

Forty-four 6-in. (150-mm) diameter specimens were compacted on site using plant mix at the FHWA mobile laboratory located within the drum plant premises. Thirty-two of them were 2.4 in. (61 mm) and 12 of them were 7.0 in. (175 mm) tall. The taller specimens were cored and saw cut to conform to E* testing. Plant loose mix collected at the plants was quickly brought to the mobile laboratory and placed in the oven 1 to 2 hours to achieve respective compaction temperature for that particular mixture. The FHWA mobile laboratory crew compacted these specimens with an IPC Servopac Superpave gyratory compactor to 7.0 ± 1.0 percent AV content.

Table A-42. Materials sampling scheme for the Midland–Odessa field site.

Sample Type	Material	Point of Sampling
Lab Mixed, Lab Compacted	Fine Aggregate	Stockpile
	Coarse Aggregate	Stockpile
	PG 64-22 Asphalt	Transport Truck
	PG 76-22 Asphalt	Transport Truck
Plant Mixed, Lab Compacted	Loose Mix	Windrow
Plant Mixed, Field Compacted	Road Cores	Center of the Paving Width

APPENDIX B

Effect of Plant Type on Binder Aging

The properties of asphalt binder change as it is heated and mixed with aggregates in an asphalt mixing plant. The heating and mixing action drives off some of the lighter ends of the asphalt binder, facilitates absorption of the asphalt binder into the aggregate, and results in oxidation of the binder (Anderson and Bonaquist 2012; Glover et al. 2009; Morian et al. 2011). Increased mixing times and temperatures are generally believed to result in increased aging. It is important to be able to simulate these changes in binder properties in the laboratory so that mixtures can be conditioned prior to testing to represent field properties. This is necessary to be able to better predict performance of the asphalt mixture in the field.

The thin-film oven test was developed to simulate this aging that occurs when HMA is produced. This test is conducted under a single set of conditions (temperature, heating time, film thickness, etc.) so at best it is a rough estimate of the changes in properties since mixtures are produced in the field over a range of temperatures, mixing times, and other conditions. With new materials such as warm mix asphalt (WMA), the mixing temperatures are much lower than that for hot mix asphalt (HMA), likely resulting in more variability in the recovered binder properties (Rand and Lee 2012).

There has been much interest in the use of WMA since it was introduced to the United States in 2004, and many state departments of transportation (DOTs) have begun to use a significant amount of WMA. The National Asphalt Pavement Association (NAPA), on behalf of the FHWA, has estimated that during 2010, the amount of WMA produced in the United States was approximately 47 million tons, which is approximately 10 percent of all dense-graded asphalt mixtures (Nadeau 2012). The amount has increased since then and many believe that within the next few years, the amount of WMA that is used will exceed the amount of HMA used. Since WMA is produced at temperatures lower than the HMA temperatures, it is clear that the oxidation, loss of lighter ends, and absorption of the binder could be significantly affected.

There is a need to evaluate present methods for estimating the properties of asphalt binders and mixtures after plant production and to determine an improved method for simulating the aging that occurs during mixture production. There are a number of asphalt plant types and many different operating conditions for each plant type. Asphalt mixtures are also produced over a range of temperatures even within a particular plant type, which will affect the changes in binder properties. The factors during plant production that most widely affect aging and absorption of the asphalt binder need to be identified so that they can be considered when investigating effects of mixing plants on binder property changes. It is very likely that plant design is a contributing factor to the aging of asphalt mixtures.

Objective and Scope

The objective of the report presented in this appendix is to identify the operating characteristics of the various types of asphalt plants that affect the oxidation, loss of lighter ends, and absorption properties of an asphalt binder.

This study was performed through a literature search on aging and absorption of asphalt binders. While there has been a significant amount of work on aging of asphalt binders, there has been very little work investigating the effects of different types of asphalt plants on aging characteristics of asphalt binders. Much of the information on the effect of plant operations on asphalt aging was developed based on the experience of the research team. The research team has also collected information from plant manufacturers about the plant types and characteristics.

This study did not include any new testing but primarily involved collecting the information available in the literature along with the experience of the research team. The results of this study were used to develop a test plan for evaluating the aging and absorption of asphalt binders under various conditions during the production of WMA and HMA.

Aging of Asphalt Binder

Asphalt aging is generally caused by loss of some of the volatiles in the asphalt binder, oxidation of the asphalt binder, and selective absorption of the asphalt binder into the aggregate pores (Anderson and Bonaquist 2012; Glover et al. 2009; Morian et al. 2011). Aging is at its most severe condition when the asphalt binder comes in contact with oxygen, at high temperatures and in thin films. Some mixtures tend to have a thinner asphalt film thickness (FT) and these mixtures may have a higher rate of aging. The source of asphalt binder can have a significant effect on the aging characteristics (Rand and Lee 2012). Hence, care must be used when comparing the effect of plant production on aging when using asphalt binders from different sources.

Aging is exacerbated when the binder is absorbed into the aggregate particles. This absorption is believed to be selective resulting in some of the lighter ends of the binder being absorbed into the aggregate leaving a stiffer binder on the aggregate surface. This results in a stiffer mixture. For a given aggregate, more absorption occurs at higher mixture temperatures (lower asphalt binder viscosity) and when the mixture is held at a higher temperature for a longer time (Houston et al. 2005). Experience has shown that the amount of absorption of asphalt binder into the aggregate pores is generally affected more by the aggregate absorption (AASHTO T 84 and T 85) and binder viscosity than by the plant operating characteristics.

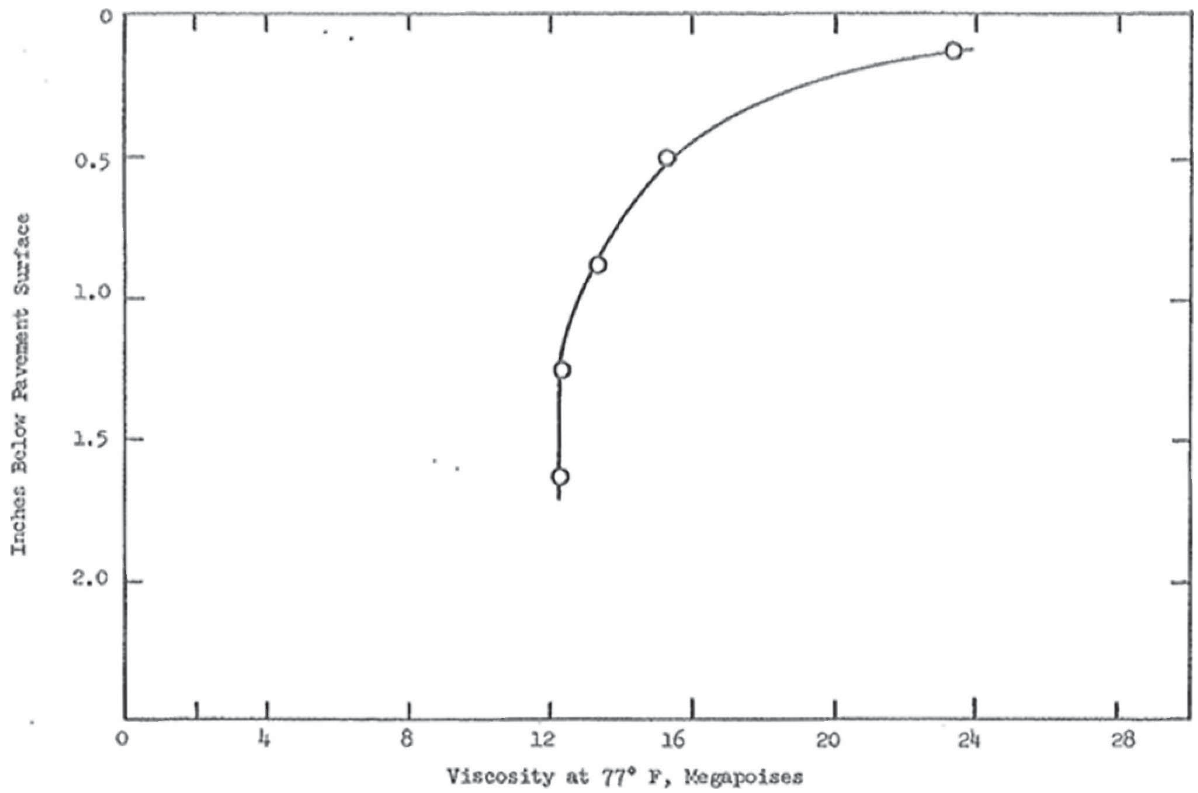
Once the asphalt mixture has been produced, placed, and compacted, the amount and rate of aging is affected by the climate and the in-place mixture properties such as film thickness (FT) and voids in the mixture. Dickinson (1980) showed, as expected, the rate of hardening is higher in hot climates than in cold climates and that the higher the air voids in the mixture the higher the viscosity of the recovered binder. Corbett and Merz (1975) showed that the location of the asphalt layer in the pavement structure also influences the level of aging in the mixture. Hardening occurs more quickly near the surface of a pavement because of greater exposure to air and sunlight. Coons and Wright (1968) evaluated the hardening of asphalt binder with depth in actual pavement cores. A total of 14 projects were considered in this study with core ages ranging from 4 months to 12 years. They found that the rate of change in viscosity was highly variable in the top 1.5 in. of the pavement and that below that level in the pavement, there is little difference in the rate of change in viscosity with depth. The rate of change in viscosity is much higher for the asphalt mixture closest to the surface. This finding is shown in Figure B-1, where the average viscosity of all samples of different ages was plotted as a function of depth. At a depth of approximately 1.5 in. and greater, the viscosity can be seen to be approximately the same.

Houston et al. (2005) developed an aging model that was incorporated into the *Mechanistic–Empirical Pavement Design Guide* (MEPDG). This model shows that the most significant changes in binder viscosity in asphalt pavement occur in the top few inches of the pavement. Prapaitrakul (2009) indicates that one difficulty with the assumptions in this model is that the solvent recovery process could leave enough solvent in the recovered binder to soften its properties considerably. This remaining solvent could be greater for the more heavily aged binders, which tend to be stiffer. Because of this, stiffer binders could have a higher impact caused by residual solvent, leading to inaccurate relative viscosity values when binder properties are monitored over time.

Glover et al. (2005) conducted a study in which a large number of Texas pavements were cored and the binder extracted, recovered, and tested to determine binder stiffness as a function of age. This study showed that binders do age in pavements below the surface and that the hardening of binder continues over time.

There is little that can be done about the climate but the air void (AV) content in the in-place mixture and other mixture properties are controlled during the production and compaction process. Most specifications require some minimum compaction level to ensure that the mixture has adequate density to begin with and that the asphalt mixture maintains acceptable properties for an extended period of time. While high in-place AV content may not have a significant effect on short-term performance, these high void levels do result in more rapid oxidation of the binder and may result in long-term durability issues such as cracking and raveling.

The focus of this report is on short-term aging as affected by asphalt plant type; hence, the primary concern is the oxidative aging process during mixture production. Material properties that may have an effect on aging during production are also discussed. However, the effect of asphalt plant type on short-term aging cannot be determined without some understanding of the effect of binder and mixture properties on



Source: Coons and Wright (1968).

Figure B-1. Average viscosity of field samples as a function of depth.

this aging process. Knowing the effect of binder and mixture properties on short-term aging will allow these components to be controlled so that the effect of plant operations can be more accurately determined.

Factors that Affect Binder Aging During Plant Production

The aging (hardening) of asphalt binder takes place in two phases: short-term aging during plant production and laydown and long-term in-service aging that occurs during the life of the pavement (Houston et al. 2005). The short-term aging is primarily due to conditions such as mixing temperature, mixing time, and other factors that occur during the production of the mixture. The long-term aging is more affected by items such as AV in the compacted in-place mixture and climatic conditions.

Short-term aging occurs due to volatilization and oxidation, as well as other factors, such as absorption of asphalt binder into the aggregate particles that are responsible for the changes in binder properties. Oxidation is the change in chemical composition of the binder due to the reaction with oxygen. Volatilization is the loss of lighter oil components of the binder due to evaporation. Oxidation and volatilization are expedited when the temperature of the asphalt binder is increased. They are also accelerated by agitation during mixing in a plant or in remixing at the job site, although mixing is critical to ensure the uniformity of binder distribution and aggregate distribution in the mixture as well as the uniformity of temperature. Houston et al. (2005) stated that 10 to 30 percent of the total ultimate hardening of asphalt binder occurs in a short period of time during mixing and laydown operations.

Mixing Temperature

Houston et al. (2005) suggested that mixing temperature and the source of asphalt binder are the most important factors affecting short-term aging. Heating the mixture to a higher temperature during production results in a higher loss of volatile materials, more absorption of asphalt binder into the aggregate particles, and increased oxidation of the binder. Experience has shown that the trend, up until a few years ago, had been a gradual increase in the temperature of the mixture during plant production. One reason for these higher temperatures has been the increased use of polymer-modified asphalt. These modified asphalts are stiffer and require higher temperatures to obtain a workable mixture. There has also been an increased emphasis on using angular aggregates and coarser gradations which tend to produce stiffer mixtures, again requiring a higher mixing temperature.

These higher temperatures may provide for a more workable mixture and a safety factor in mixture temperature in case there is a delay in the construction operation. It is reasonable to believe that this increase in temperature has increased the rate of mixture aging resulting in some loss in durability. These higher mixture temperatures have also caused an increase in mixture cost due to the requirement for more energy to produce these hotter mixtures. Another effect of these higher temperatures is higher emissions and this is one of the reasons for the increasing use of WMA.

A study conducted by Kennedy and Huber (1985) evaluated engineering properties of binders and mixtures produced over a range of mixing temperatures and stockpile moisture contents in both drum mix and batch plants. The mixing temperatures were varied from 325°F to 175°F, which is much lower than normal operating temperatures for HMA. The levels of moisture content in the stockpiles were denoted as dry, wet, and saturated. The penetration of the extracted and recovered binder was measured to estimate the hardening that occurred under the experimental conditions. The amount of asphalt hardening was not significantly affected within the ranges of mixing temperatures selected; however, the higher mixing temperature resulted in a penetration reduction of 5 to 10 units. There was some variability in the data, which resulted in a drop of 5 to 10 penetration units not being statistically significant. It was difficult to obtain uniform coating of the aggregates with mixing temperatures below 200°F. No significant difference was identified between mixtures produced in the batch plant and the drum plant for all conditions of mixing temperature and stockpile moisture, but an increase in moisture content caused a reduction in asphalt hardening during mixing. In conclusion, the study was not able to identify a clear relationship between mixing temperature, moisture content, and the properties of asphalt mixtures produced in either batch or drum plants for the aggregates studied. Some differences were noted, but these differences were not determined to be statistically significant.

Asphalt Binder Grade

Asphalt binder grade is also a critical factor in the aging process. Softer binders are more likely to experience significantly higher changes in binder properties than stiffer binders for a given temperature. Many times stiffer binders are heated to a higher temperature during production, which may increase the changes in properties for the stiffer binder.

Another property that is dependent on viscosity of the asphalt binder is binder absorption. Binders at lower temperatures will have higher viscosities resulting in lower absorption of the binder into the pores of the aggregate. Accordingly, at high temperatures, there will be more of the binder absorbed into the aggregate pores. Lee et al. (1990) describes some of the problems that may result if asphalt absorption values in asphalt mixture tested in the laboratory do not match the absorption values during production:

1. Incorrect measurement of AV, voids in mineral aggregates (VMAs), or voids filled with asphalt, which may lead to durability or stability problems with the mixture.
2. Low effective binder content that may cause raveling, cracking, or stripping.
3. Premature age hardening and low-temperature cracking as a result of changes in asphalt properties.
4. Construction problems such as segregation and tender mixtures.

Mixing Time

Mixing time will have an influence on the hardening of the asphalt binder. During mixing, there is more opportunity for the coated particles to react with oxygen, increasing oxidation. Longer mixing time will cause greater hardening due to the increased loss of volatile material and more oxidation. Consequently, mixing time is generally minimized (just long enough to obtain even coating of the asphalt binder and the aggregate particles) to keep oxidation at a minimum and to reduce the amount of breakdown of the aggregate during the mixing process. The *Hot-Mix Asphalt Paving Handbook* (US Army Corps of Engineers 2000) states the following:

- If the binder is properly distributed, additional mixing time does not improve the coating and only hardens the binder by exposing it to air.
- In batch plants, for a short wet-mixture time of 28 to 35 seconds, the penetration will decrease 30 to 45 percent for an average binder; also, viscosity will increase approximately the same percentage. If the mixing time is increased up to 45 seconds, the penetration of the binder may decrease up to 60 percent and the viscosity may increase up to four times its value.
- For drum mix plants (DMPs), the hardening of the binder with respect to batch plants is variable and depends on different factors such as binder composition and thickness of the asphalt cement film around the aggregate particles. A decrease in mixture discharge temperature and an increase in the volume of aggregate and production rate will generally produce less hardening of the binder in a parallel-flow DMP.
- Considerably less hardening will occur in the binder in a counter-flow drum than in a parallel-flow DMP. This reduced oxidation is likely because the counter-flow plant keeps the asphalt binder away from the flame and hot gases while, in a parallel-flow drum, the asphalt binder does have some contact with the flame and the hot gases.

Plant Type

Lund and Wilson (1984) compared the aging in asphalt binder between batch mix plants (BMPs) and DMPs. They concluded that there was a significant difference, at a 90 percent confidence level, between the two types of plants. The drum mixers result in less hardening of asphalt binder, which some considered detrimental in terms of tenderness of the mixture. An approach to quantify the aging of the asphalt binder was needed. For this evaluation, the percent change in asphalt viscosity (C) at the time of paving was used, as described in Eq. (B-1):

$$\mathrm{C} = \frac{R - A}{B - A} \times 100\% \qquad \text{Eq. (B-1)}$$

Where:

A = absolute viscosity (Oregon Department of Transportation [ODOT] TM 417) of original asphalt binder used in production of the mixture.

B = absolute viscosity (ODOT TM 417) of rolling thin-film oven residue for asphalt used in production of the mixture.

R = absolute viscosity (ODOT TM 417) of asphalt recovered from the mixture.

This ratio was used to judge potential for failures due to aging based on field observations. When C values were above 50 percent, no problems were experienced; when C values were between 30 and 50 percent, some problems were experienced; and when C values were 30 percent or less, pavement problems were always experienced. Another observation during this study was that low temperature in the mixing or aggregate drying process, particularly in DMPs, will produce less aging and may produce poor combustion of burner fuel oil, which may result in fuel contamination of the mixture.

The low mixing temperature that was used in DMPs in the early 1980s did result in some issues and the temperatures were soon raised to be similar to that used in batch plants. At the time this report was prepared, the operating temperature of DMPs was very similar to that used for batch plants.

Another study was conducted by Chollar et al. (1989) to identify possible differences in asphalt binders in drum plants versus batch plants and to compare changes induced by various laboratory conditioning methods versus those taking place in drum plants. A total of 27 virgin asphalts were subjected to different conditioning that included thin-film oven, rolling thin-film oven (RTFO), small steam distillation (SSD), forced-air distillation (FAD) and rolling forced-air distillation (RFAD). Two of these methods, FAD and RFAD, involved blowing air over asphalt binder and collecting the volatile matter removed. The SSD technique involved steam bubbled through the asphalt that removes volatile asphalt components from the residue. Different physical and chemical properties were evaluated and compared with extracted and recovered asphalt binder from drum plant operations. The authors found that the recovered asphalt binders from drum plants were slightly harder with lower penetration values at 25°C and higher viscosity values at 60°C than those from batch plant mixing and that they were more oxidized with higher carbonyl and oxidized nitrogen contents. This is opposite the information that many other researchers, including Lund and Wilson mentioned previously, have found. Chollar et al. also concluded that the binders recovered from the asphalt mixtures produced at a DMP, at the time the report was published (1989), were stiffer than binders recovered in previous years for mixtures produced in a parallel-flow DMP.

As mentioned earlier, originally drum mix asphalt plants heated the asphalt mixture to a much lower temperature than batch plants. For example, many drum mix projects, especially in the early days of DMPs, produced asphalt mixture at a temperature of approximately 240°F (116°C) while most batch plant mixtures were produced at over 300°F (149°C). This difference in temperature was true in the early 1980s, but by the end of the 1980s, the temperatures used to produce mixture in the two types of plants were approximately equal and these temperatures continued to be approximately equal at the time this report was written.

There are not many studies that looked at the effect of aging between the two types of plants, but most studies were performed when there was a significant temperature difference between the two plants. It is believed that the Lund and Wilson (1984) study was before the temperatures in DMPs had changed and it is believed that the Chollar et al. (1989) study was performed after the drum mix temperatures had increased to approximately equal that in a batch plant. Thus, it is not surprising that the two studies found opposite results.

Aggregate Gradation

Aggregate gradation can have a significant effect on aging of the asphalt mixture. The gradation of the aggregate is directly related to the surface area for a given amount of material. If the gradation is on the finer side of the requirements for a given nominal maximum aggregate size (NMAS), the surface area is larger resulting in more exposure of the asphalt film to oxygen and increased oxidation if the asphalt content is the same.

The percentage of aggregate passing the finer sieves, such as the No. 200 sieve, will have a much greater effect on the aggregate surface area than coarser material. The fines can make the binder stiffer, modify the moisture resistance of the mixture, and affect the aging characteristics. Fine particles that are less than approximately 20μm in diameter may become part of the asphalt binder, causing an increase in hardening of the binder and a stiffer mixture. An increase in the dust percentage will normally fill some of the voids in the aggregate matrix and hence decrease the VMAs. Increasing the amount of material passing the No. 200 sieve results in an increase in the total surface area of the aggregate and typically a reduction in the optimum asphalt content, hence, resulting in a thinner FT that may lead to mixture durability issues (Chadbourn et al. 1999).

Reclaimed Asphalt Pavement

The use of RAP in the asphalt mixture during production will make it more difficult to determine aging of the virgin asphalt binder. Most asphalt contractors now use RAP in their mixtures as a standard practice. The asphalt binder in the RAP has typically hardened to the point that it is significantly harder than the virgin binder in the asphalt mixture. For relatively small amounts (up to 15 to 20 percent) of RAP, there is typically no change in the virgin asphalt binder grade that is used. However, even at these small RAP percentages, there is some stiffening of the properties of the binder recovered from a mixture as a result of the aged binder in the RAP. The properties of the RAP binder and the virgin binder are considerably different making it difficult to know if the change in binder properties between two mixtures produced in two different plants is caused by the addition of RAP or aging of the asphalt binder during plant production. Also, the mixing temperature is sometimes increased to improve workability when RAP is used resulting in more aging of the binder during the production process. In most types of plants, the aggregate has to be superheated when mixed with the RAP to ensure that the combined temperature of the asphalt mixture is acceptable.

Aggregate Absorption

The absorption of asphalt binder into the aggregate will generally result in some stiffening of the effective binder due to selective absorption of lighter fractions of the binder (Lee et al. 1990). Effective binder is defined as that which is not absorbed into the aggregate. When the lighter ends of the binder are absorbed by the aggregate pores, the effective binder remaining on the surface of the aggregate is stiffer. Some aggregates have high water absorption values (3 to 4 percent) and other aggregates have low absorption values (less than 1 percent). If the water absorption (AASHTO T 84 and T 85) of the aggregate is small, then the amount of asphalt that will be absorbed during production will be small; however, for highly absorptive aggregates, the amount of asphalt binder absorbed can be significant. This selective absorption of asphalt binder can result in significant changes in the properties of the effective asphalt binder.

Use of Lime as Anti-strip

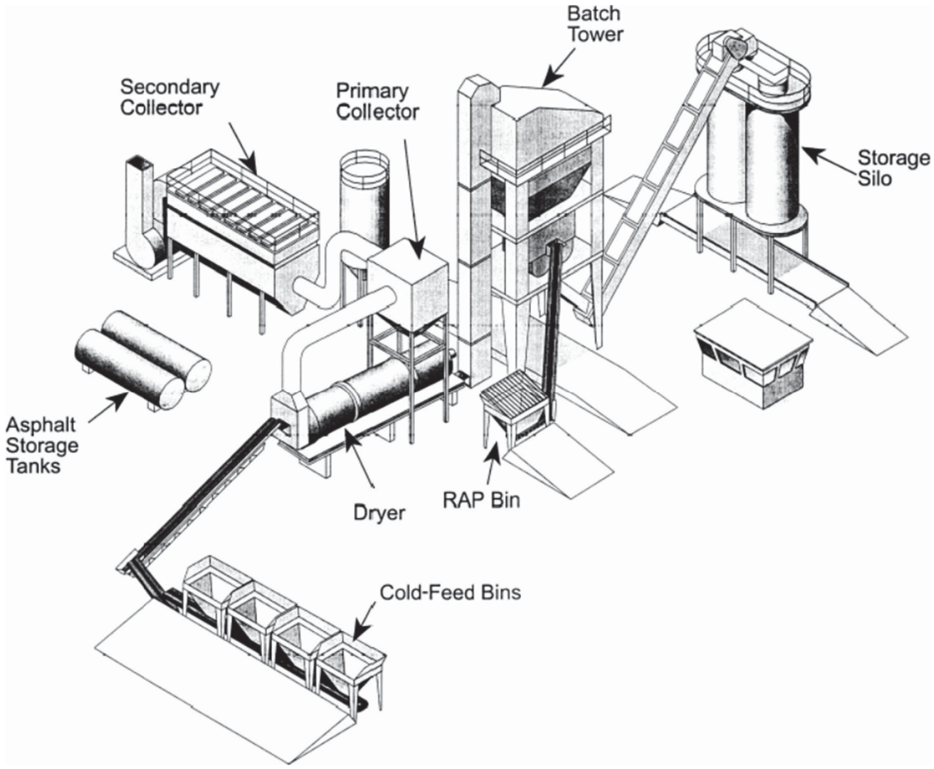
Lime, which may be added as mineral filler, is often used to improve the resistance of asphalt mixtures to moisture damage. As a result of lime's ability to improve moisture susceptibility, many state DOTs require the use of lime in their asphalt mixtures. While lime improves resistance to moisture, research by Plancher et al. (1976) has also shown that lime reduces aging of the asphalt binder during production of the mixture. Hence, if lime is being used to improve moisture susceptibility, it will likely affect the properties of the recovered binder. If lime is used on one project and not on another project, it may be difficult to evaluate the effect of plant operations on the recovered properties of the binder.

Incomplete Ignition of Fuel

There have been concerns expressed over the years about the possibility of incomplete combustion of the burner fuel during production (US Army Corps of Engineers 2000) in a DMP. There have been cases where this unburned fuel ends up being collected in the baghouse. There have even been cases where fire has occurred as a result of this problem. If combustion is incomplete, some of the unburned fuel will react with the asphalt in the mixture to soften the asphalt binder and likely result in a mixture with a tender surface. This is likely a more significant problem when using a parallel-flow DMP, but it potentially can be a problem with all types of plants. This has not been a significant problem but it has occurred and does need to be considered.

Batch Plants

Today, the percentage of batch plants being used to produce HMA in the United States is on the decline, but there is still a significant amount of this plant type being used. The basic components and operations of a batch plant (Figure B-2) include aggregate storage and cold feeders, aggregate dryers, screening and storage of hot aggregate, storage and heating of the binder, scales for weighing the proper amount of each aggregate hot bin and the asphalt binder, a pug mill for mixing of aggregate and binder, and likely a storage silo. The aggregate is removed from stockpiles and placed in the cold-feed bins, which add the different aggregates in the proper proportions to the collector belt. From here, the aggregates are carried to the dryer where the aggregate is heated and dried. For most HMA projects, the aggregate is heated to approximately 300°F or higher. The aggregate is then transported to the top of the mixing tower by a bucket



Source: US Army Corps of Engineers (2000).

Figure B-2. Schematic of typical batch plant.

elevator, referred to as the hot elevator, and discharged from the elevator onto a set of vibrating screens where it is separated by size and deposited into hot bins.

The correct aggregate proportion to be used from each bin is determined by weight by its introduction into a weigh hopper. If RAP is used, it is typically fed directly into the weigh hopper and is heated primarily by the virgin aggregate, which has to be heated to a significantly higher temperature (superheated) if RAP is used. After weighing in the proper amount of material from each hot bin, the materials are dropped into a pug mill where they are mixed with the asphalt binder that has been weighed separately. Prior to adding the asphalt binder to the mixture, the binder is stored in a binder storage tank where it is kept at an elevated temperature until being pumped into the binder weigh bucket. The aggregate is mixed dry ahead of the binder for approximately 5 seconds. The asphalt binder that has been weighed is then discharged into the pug mill, and the aggregate and asphalt binder are mixed for a specified amount of time. This wet mixing time should be no longer than needed to get complete coating of the aggregate. A typical wet mixing time is 25 to 35 seconds. If the asphalt mixture is agitated too long, it will result in excessive oxidation of the asphalt binder and excessive breakdown of the aggregate. After the mixing is completed, the mixture is discharged from the pug mill into a waiting truck or conveyed to a hot storage silo. On most projects, the asphalt mixture is conveyed to a hot storage silo where it is stored until used, typically a few hours or less.

Factors that Could Cause Significant Hardening of the Binder (Generally Assumed to be Caused by Oxidation)

Mixing Temperature and Mixing Time

The aggregates are delivered from the cold-feed bins to the dryer to remove moisture from the aggregates and to heat the material to the required discharge temperature. The burner is adjusted to provide sufficient heat to dry and heat the material. If the rate of feed is increased, the burner will have to consume more fuel to bring the aggregate to the same temperature. Since 92 to 96 percent, by weight, of the asphalt mixture is aggregate, the aggregates control the temperature of the mixture that is ultimately obtained after mixing in the pug mill. Excessive heating of the aggregates can cause hardening of the asphalt binder during the mixing process. On the other hand, under-heated aggregates are difficult to coat evenly and the mixture will be difficult to handle and place. In a batch plant, there is direct control of the dry and wet mixing time of the mixture. In a DMP, this mixing time is controlled indirectly by drum diameter, length of mixing unit, slope, and speed of rotation of the drum. Changes in mixing time and temperature will definitely affect the aging of the binder.

Production of Recycled Mixture

There are three variables that dictate the temperature the new aggregate needs to be heated to so that sufficient heat transfer with the ambient RAP temperature results in the desired mixture temperature: (1) the moisture content of the RAP, (2) the discharge temperature of the mixture, and (3) the amount of RAP used (US Army Corps of Engineers 2000). The new aggregate needs to be superheated when using RAP to a higher-than-normal temperature, so the resulting mixture temperature is satisfactory. This superheated temperature of the aggregate can be significantly higher than that required to produce a conventional mixture with little or no RAP.

When RAP is incorporated, batch plants may need some modification since excessive smoking and material buildup problems may occur. Also for higher amounts of RAP, the aggregate must be heated to a very high temperature and this may create problems in some plants. Modifications may be needed in the aggregate dryer, burner, hot bins, or dryer exhaust system.

The methods for batch plant operations to add RAP are briefly described below (Kandhal and Mallick 1997):

- **Method 1**—The superheated aggregate and RAP are introduced into the boot of the hot elevator and the mixed material is screened and stored in hot bins. The scavenger system in the tower removes the water evaporated from the RAP. With this method there are no problems with emissions, but only low percentages of RAP may be used because excessive asphalt binder may blind the screens.
- **Method 2**—In this method the batch tower requires one hot bin to hold the mixture components (screens are removed). The hot aggregate and RAP are introduced at the foot of the hot elevator and stored in the hot bin without screening. This method allows the use of up to 40 percent RAP.
- **Method 3**—This method, also called the Maplewood method, uses cold prescreened RAP that is delivered directly to the weigh hopper of the batch tower with the superheated virgin aggregate from the hot bins. The RAP is sandwiched between superheated aggregate thus reducing the time needed for the RAP to become adequately heated.
- **Method 4**—This method has a new control feed system. RAP is weighed separately and dropped intermittently into the pug mill with a feeder that introduces RAP over a 20- to 30-second period. This method allows more control of the steam that is generated.
- **Method 5**—In this system RAP material is preheated in a separate dryer before mixing with the aggregate. The RAP is conveyed to a separate heated storage bin with weigh hopper. RAP material is weighed separately and conveyed to the pug mill to produce the recycled mixture.

Batch Size

The batch size that is used is a function of the pug mill size and other parameters. The capacity of the plant to produce asphalt mix ranges from 1 ton up to approximately 10 tons in a single batch. Regardless the batch size, the total required mixing time is approximately the same. For a given pug mill size, a smaller batch is likely to result in more oxidation of the binder than a larger batch. However, the batch size is not expected to be a significant problem since most asphalt is produced with a batch size within the limits recommended by the manufacturer.

Material and Mixture Properties

It is also important to ensure that the most common material and mixture issues that will affect the oxidation of the asphalt binder are controlled. These issues include use of lime, aggregate absorption, gradation of aggregate, and use of RAP. The best approach is to set up the test plan to monitor the use of lime and RAP and carefully document these to understand their potential effects on the outcome.

Parallel-Flow Drum

A parallel-flow DMP is a continuous mixing process that uses proportioning cold feeds to control the aggregate grading. The main components are the cold-feed system, asphalt cement supply, drum mixer, storage silos, and emission-control equipment. The major difference between this process and the batch process is that, in a parallel-flow drum, the dryer is used to both heat and dry the aggregate and mix these aggregates with the asphalt binder. Also, the parallel-flow DMP is a continuous mixing operation and the batch plant produces mixture in batches.

The aggregate is proportioned through the cold-feed system and introduced to the drum at the high end of the drum next to the burner. The drum rotates and the aggregate moves toward the low end of the drum traveling in parallel with the air flow (hence, parallel-flow). Flights lift up the aggregate particles and then drop them through the hot air flow resulting in drying. The temperature of the aggregate increases as the aggregate moves down the drum until the aggregate reaches a point where the temperature is relatively constant. The heat generated by the burner and the air flow evaporate the moisture in the aggregate. Because of this, the time the aggregate temperature remains constant depends in part on its moisture and its porosity (US Army Corps of Engineers 2000). In general, moisture from porous material takes longer to be removed from the internal pores. When the moisture is removed, the temperature of the aggregates rises, asphalt cement is injected, and the flights tumble the mixture putting the material in contact with the exhaust gases. Finally, the mixture achieves the required discharge temperature at the end of the drum. This process indicates that the mixing time and mixture temperature are highly dependent on the moisture in the aggregates.

Some of the factors that influence the time required for an aggregate particle to travel through the drum include the length, diameter, and slope of the drum, the number and type of flights, the speed of rotation of the drum, and the size of the aggregate particles. A typical duration ranges from 4 to 8 minutes for an aggregate to reach the discharge end of the drum (US Army Corps of Engineers 2000). This duration time in the drum controls the mixing time for the mixture for a given plant. Another factor that will affect the mixing time is the location where the asphalt binder is added to the drum. In a parallel-flow drum plant, the asphalt binder is added about halfway down the drum while, in one type of counter-flow plant, the mixing occurs near the end of the drum. For some counter-flow plants, there is a coater or unitized drum that allows mixing to take place outside the drum. All of these approaches for adding the asphalt binder and mixing will affect the mixing time.

The binder is introduced to the mixture components through a pipe in the rear of the mixing chamber. The point of discharge of the binder varies but is generally between the midpoint and two-thirds of the way down the drum length. Once the coating of the aggregate occurs, the mixture is discharged and conveyed to either a surge bin or HMA storage silo, where it is loaded into transport trucks. Most plants use a storage silo for storing the asphalt mixture, where it is typically stored for a few hours but may be stored up to 72 hours.

Factors that Could Cause Significant Oxidation

Mixing Time and Temperature

The most likely causes of oxidative aging are the temperature of the mixture and mixing time. The mixing time is a function of slope of dryer, diameter of dryer, rotation of dryer, etc. All of these parameters will affect the mixing time. Mixing time and temperature have to be considered in any test plan that is established to determine the effect of production on short-term aging of the asphalt binder.

Production of Recycled Mixture

When a recycled mixture is produced using a parallel-flow DMP, the most common method is to add the RAP to the aggregate from its feeder bin and conveyor system into an entry point located near the center of the drum. By adding the RAP with this split feed system, there is better control of emissions since the RAP is protected from the high-temperature exhaust gases by the veil of aggregate upstream of the RAP entry point (US Army Corps of Engineers 2000). If high-RAP content is used, less virgin aggregate is placed into the drum, reducing the veil of aggregate upstream of the RAP and decreasing the amount of heat transferred from the exhausted gases to the aggregate. This may cause the RAP to come in direct contact with the flame, possibly resulting in the generation of smoke and the further oxidation of RAP binder. When 20 percent or less of RAP is used, minimal emissions occur, the veil of aggregate is satisfactory to protect the RAP, and the discharge temperature of the mixture is adequate. Higher RAP percentages may be possible only if production is carefully monitored. Figure B-3 shows a typical configuration of the mixing operation in a parallel-flow DMP.

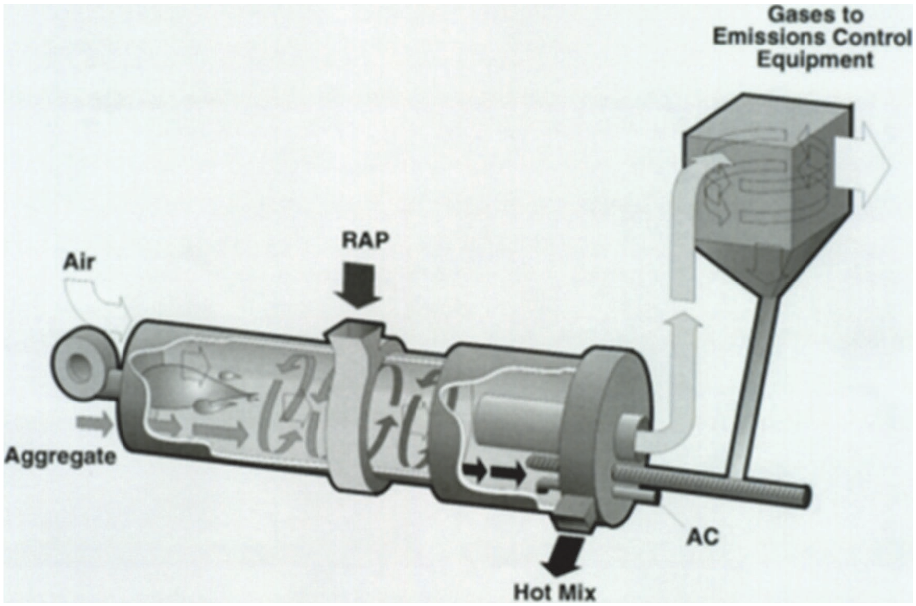
Production Rate

The production rate will depend upon several factors such as aggregate temperature, mixture discharge temperature, drum diameter, fuel type, exhausted gas velocity and volume, estimated air leakage into the system, average aggregate moisture, and gradation. One of the most important factors on plant production is the average moisture content of the coarse and fine aggregates. When the average percentage of moisture in the aggregate increases, the production rate of a DMP for a particular diameter decreases. On the other hand, for a constant average moisture content, the production rate increases when the drum diameter becomes larger. Plant operators of DMPs usually develop charts for drum mix production rates as a function of drum diameter and length and average moisture content. If RAP is used, the production rate can have a significant effect on the aging of the asphalt binder.

Design of Flights

The main functions of the flights in the different sections of the drum are to lift up the aggregate and drop it through the hot air flow so that it (1) shields the asphalt binder and RAP, if used, from the flames; (2) exposes the aggregate to the heat to dry the aggregate; (3) increases the aggregate temperature; and (4) facilitates coating of the aggregate with the binder.

The first flights at the upper end of the drum direct the aggregate into the drum beyond the flame. The next flights



Source: US Army Corps of Engineers (2000).

Figure B-3. Parallel-flow drum mix plant.

lift some of the aggregate from the bottom of the drum and tumble the material through the exhaust gases. The aggregate then moves down the drum; near the center of the drum, a veil of aggregate develops across the cross-sectional area. This veil allows the heat transfer between the exhausted gases and the aggregate. The asphalt cement is added at the end of the drum and mixing flights combine the aggregate with the asphalt cement and allow the heat transfer between the material and exhausted gases to be completed and also allows the material to reach the required discharge temperature.

The condition of these flights can significantly affect the oxidation of the asphalt binder. There are also adjustments that can be made inside the drum to reduce the speed of material through the drum. These adjustments can affect the mixing time and result in changes to the aging characteristics of the binder. It is important to ensure that the flights are in good operating condition. The mixing time must be estimated so that factors other than plant operating characteristics can be quantified.

Incomplete Burning of Fuel

There are a number of types of fuel that can be used in the burner for drying and heating the aggregates and asphalt materials (US Army Corps of Engineers 2000). Depending on the operating condition of the burner, how it is set to operate, and the type of fuel being used, there are cases where complete combustion of the fuel does not occur. In these cases, the unburned fuel can contaminate the asphalt binder resulting in some softening of the binder. This is expected to be more of a problem in a parallel-flow DMP than in the other plant types, because the asphalt binder will come in direct contact with any fuel that is not completely burned. This might cause some problems in other plant types since it is possible that this unburned fuel will attach to the aggregate as it moves through the dryer and then be coated with asphalt binder when added. However, in other plant types, the air flow will remove much of any unburned fuel from the flow of materials before the asphalt is added so contamination should be minimized.

Material and Mixture Properties

The material and mixture properties are expected to have approximately the same effect in the parallel-flow DMP as in the BMP as discussed previously.

Counter-Flow Drum

The counter-flow drum plant became popular in the 1980s. It utilizes a counter-flow aggregate dryer similar to the dryer used in batch plants. Steps have to be taken to ensure that the asphalt binder is protected from the burner flames. In this type of plant, the materials move in the drum in an opposite direction or counter-flow to the exhausted gases. The aggregates enter the high side of the drum and flow by gravity in the sloped drum toward the burner, which is located near the low end of the drum. In counter-flow plants, the aggregate passes the burner and is protected from the actual burner flames. There are three different types of counter-flow drum mix plants that are generally used. The first one has a mixing unit inside the drum but located on the end of the drum behind the flame. The second type refers to a drum mixer most commonly referred to as a double-barrel plant that has the mixing unit folded back around the aggregate dryer portion of the drum. The third type of counter-flow plant is a plant that dries the aggregate with the counter-flow drum after which the aggregate moves into an external mixer (sometimes referred to as a dryer-coater) where the asphalt binder is added and mixed with the aggregate. In contrast to a parallel-flow drum mix plant, no asphalt binder is added inside the drying portion of the counter-flow drum mixer.

Drum Mixer

In this type of plant, the burner is embedded into the drum and the mixing occurs in the bottom portion of the drum (Figure B-4). After the aggregate is heated and dried, it passes the flame and then is mixed with the asphalt binder in a separate chamber. Once the aggregate has entered the mixing unit of the drum, the RAP, if used, is added to the aggregate. Since the RAP is added to the drum behind the burner, it does not come in contact with the burner flame or hot gases. This helps to avoid problems with emissions. Heat transfer from the new aggregate occurs when the two materials come together at the end of the mixing unit. After the RAP is added into the mixing portion of the drum, any other additives that are required such as baghouse dust and mineral filler, are also added at or near that point. The mixing of the RAP, filler and baghouse fines, and new aggregate begins when the materials are introduced into the mixing chamber. After all of the materials have been mixed, the asphalt binder is added and coating of the aggregate takes place. Depending on the angle of the drum and the number, rotation speed, and type of flights in the mixing unit, the mixing time typically ranges between 45 and 60 seconds.

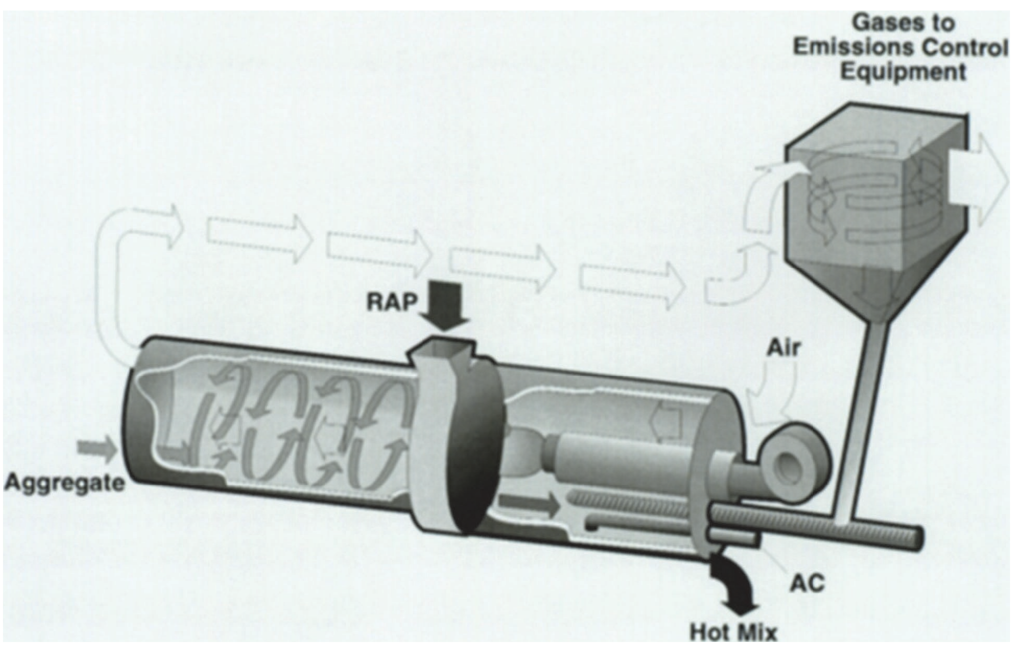
Factors that Could Cause Significant Oxidation

A number of factors in a counter-flow DMP are likely to affect oxidative aging. The most likely causes of oxidative aging are the temperature of the mixture and mixing time as with the other plant types that have been discussed. The mixing time is a function of slope of the dryer, diameter of dryer, speed of rotation of dryer, etc. All of these parameters will affect the mixing time. Mixing time and temperature have to be considered in any test plan that is established to determine the effect of production on short-term aging of the asphalt binder.

As with the other plant types, it is important to ensure that the most common material and mixture issues that will affect the oxidation of the asphalt binder be closely monitored. These issues include use of lime, aggregate absorption, gradation of aggregate, and use of RAP.

Unitized Drum Mixer (Double Barrel)

This mixing unit has the mixing chamber folded back around the aggregate drying drum. The outer shell is a non-rotating unit. The heated and dried aggregate is discharged



Source: US Army Corps of Engineers (2000).

Figure B-4. Dryer mixing drum.

from the inner drum into the nonrotating outer shell or drum (US Army Corps of Engineers 2000). Once the aggregate is fed into the mixing chamber, asphalt binder and RAP, if used, are added into the external drum and away from the flame. The material blends with the superheated aggregate and the heat transfer between aggregate and RAP takes place.

Similar to the process in a conventional counter-flow drum mixer, any moisture in the RAP and emissions that is generated during the heating process is sent back into the dryer unit by the exhaust fan. The moisture is carried into the emission-control equipment and the hydrocarbon emissions from the RAP, if any, are incinerated by the burner (US Army Corps of Engineers 2000). Any additives, such as mineral filler and baghouse fines, are also added into the outer shell. Finally, binder is added into the mixture and the mixing takes place.

Mixing takes place by a series of paddles in the outside of the aggregate dryer. The size and production capacity of the unit will determine the mixing time that is typically less than 60 seconds. The paddles are actually attached to the rotating inner drum and these rotating paddles perform the mixing and facilitate the movement of the asphalt mixture toward the high end of the drum and out the exit chute. Figure B-5 shows a typical unitized mixer configuration.

Factors that Could Cause Significant Oxidation

A number of items in a unitized DMP are likely to affect oxidative aging. The most likely causes of oxidative aging are the temperature of the mixture and mixing time. The mixing time is a function of the slope of drum, production rate, and size of the mixing unit. All of these parameters will affect the mixing time. Mixing time and temperature have to be considered in any test plan that is established to determine the effect of production on short-term aging of the asphalt binder.

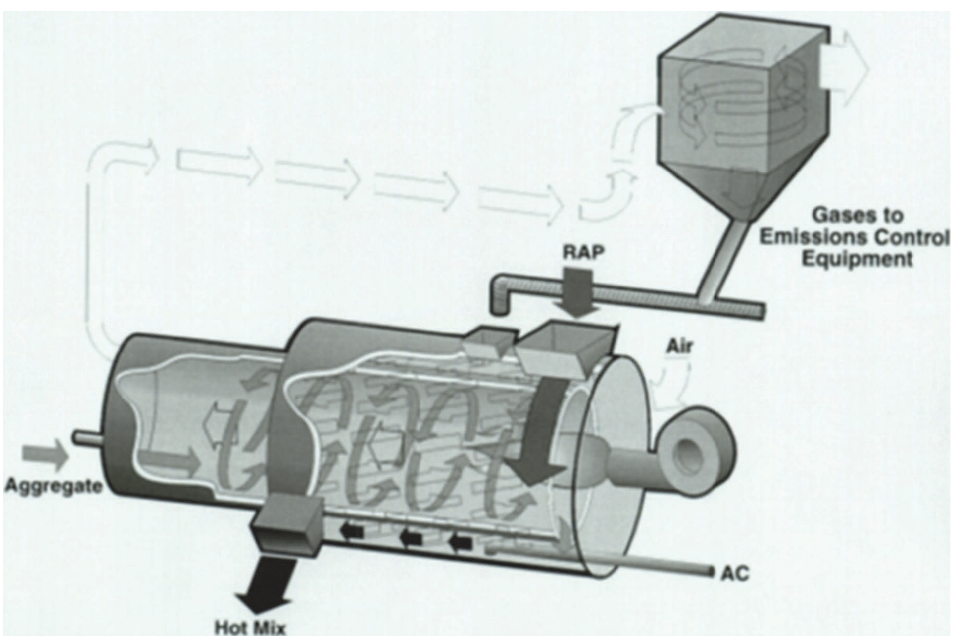
It is also important to ensure that the most common material and mixture issues that will affect the oxidation of the asphalt binder are monitored and documented. These issues include use of lime, aggregate absorption, gradation of aggregate, and use of RAP.

Mixing Outside of Dryer

The last type of asphalt plant to be discussed involves a mixer outside the drum mixer. This approach involves heating the aggregate inside the dryer and then mixing the aggregate with the asphalt binder in the external mixer. This setup separates the flame from the asphalt binder and minimizes potential damage to the binder. A schematic of the drum mixer with the mixing unit separated from the dryer is shown in Figure B-6. This technique is similar to the unitized drum mixer since the asphalt cement is mixed with the mixture outside the drum. The external mixer keeps the asphalt binder away from the flame and solves problems that may occur due to the flame coming in contact with the binder.

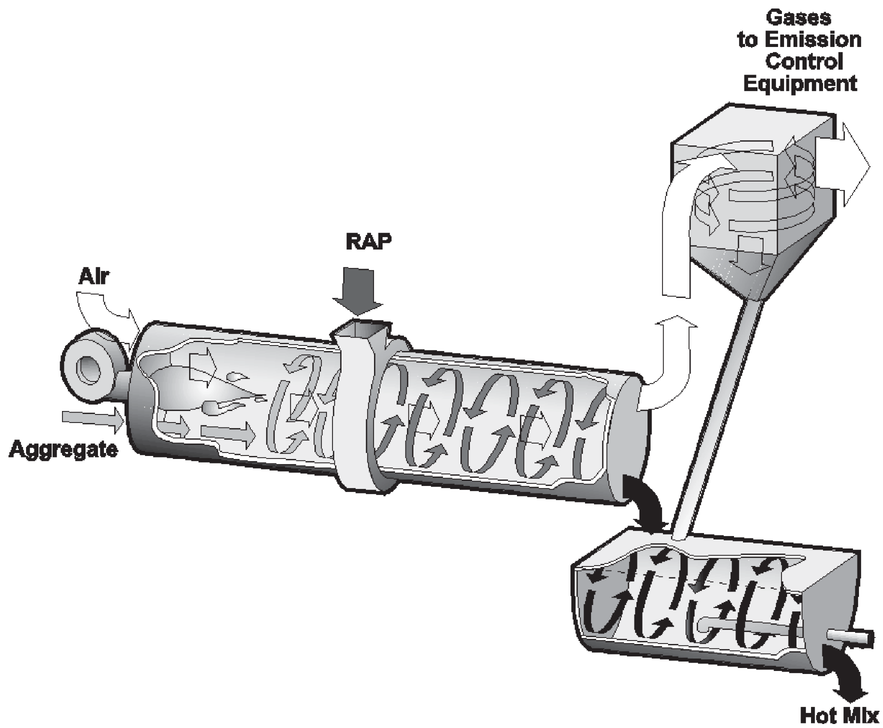
Factors that Could Cause Significant Oxidation

A number of factors in a DMP with external mixing unit are likely to affect oxidative aging. The most likely causes of oxidative aging are the temperature of the mixture and



Source: US Army Corps of Engineers (2000).

Figure B-5. Unitized drum mixer.



Source: US Army Corps of Engineers (2000).

Figure B-6. Drum mixer with external mixer.

mixing time. The mixing time is controlled by the rate of production, slope of mixer, and the capacity of the mixer. Dividing the mixer capacity by the production rate will give an estimate of the mixing time. Mixing time and temperature have to be considered in any test plan that is established to determine the effect of production on short-term aging of the asphalt binder.

It is also important to ensure that the most common material and mixture issues that will affect the oxidation of the asphalt binder are monitored and documented.

The primary difference between a parallel-flow plant and a counter-flow plant is that the asphalt mixture is exposed to hot, steam-laden gases in the mixing area of a parallel-flow plant, while the asphalt cement is introduced outside steam-laden exhaust gases in a counter-flow plant.

Plant Characteristics

A number of plant characteristics have been identified that may affect asphalt binder oxidation. Some of those characteristics are listed in Table B-1. The manufacturers shown in Table B-1 are the major producers of asphalt plants in the United States. The plants described in Table B-1 provide good examples of the various characteristics that affect oxidation. As mentioned earlier the primary factors that affect oxidation and absorption are mixing temperature and mixing time.

Many of the characteristics of asphalt plants are shown in Table B-1, but many components are custom built to meet owners' needs so some of these characteristics shown may be adjusted for specific plants that are manufactured. It is clear from Table B-1 that the counter-flow plants have gained in popularity and significantly exceed the number of parallel-flow plants. Also, there are many more models of drum mix plants than batch plants.

Storage Silos

The primary purpose of a storage silo is to provide temporary storage space for the asphalt mixture during production to help minimize the number of trucks needed and to improve the overall efficiency during construction. Without this storage space, trucks would have to operate continuously to collect the mixture being produced and to haul the mixture to the job site. This would require that the contractor have more trucks available for use than actually needed in case some are delayed while traveling or there are breakdowns.

On some projects the mixture is required to be used the same day that it is produced, but for other projects the mixture can be stored for up to 2 to 3 days or longer, and the contractor still has the ability to place and adequately compact the mixture. Of course, one concern is how much oxidation of the mixture occurs in the silo during short-term storage, but it is even more important for long-term storage. Most silos are insulated and sealed when extended storage is utilized, and many believe that this reduces the potential for significant oxidation. The binder is normally not recovered and

Table B-1. Description of asphalt plants produced by selected plant manufacturers.

Equipment Supplier	Plant Model	Plant Type	Description	Tons/Hour	Other Information
ADM	Milemaker	Counter-flow drum mix	Dual-drum counter-flow. The first portion of the drum heats the aggregate and the second portion, located behind the flame, adds filler, asphalt binder, RAP, etc. and mixes. Hence, flame is kept away from binder.	160–425	
	Roadbuilder	Parallel-flow drum mix	Conventional parallel-flow drum plant.	110–350	
	SPL Series	Parallel-flow drum mix	Conventional parallel-flow drum plant designed and built for lower production quantity.	60–160	
	Ex Series	Counter-flow drum mix	Mixing chamber behind isolation ring at burner nose.	100–425	
ALMIX	Compact Series	Counter-flow drum mix	First portion of the drum heats and dries the aggregate. Mixing with asphalt binder, RAP, and filler occurs in mixing unit behind burner.	60–140	
	Duo Drum "CF"	Counter-flow drum mix	Dual-drum counter-flow. The first drum heats the aggregate and the second drum adds filler, asphalt binder, RAP, etc. and mixes. Hence, flame is kept away from binder.	120–600	32-ft drying drum and 16-ft mixing drum
	Batch mix	Counter-flow batch	Typical batch plant where flame is kept away from asphalt binder.	50–450	
	Uniflow	Counter-flow drum mix	Single-drum plant. Mixing occurs in mixing area behind burner.	160–400	
	Dratch	Counter-flow drum mix/batch	Counter-flow drum with a batching tower and mixing drum so it can be operated as a batch plant or a dual-drum asphalt plant.	1- to 8-ton batch	
Gencor	Ultra Plant	Counter-flow drum mix	Drum plant with counter-flow drum. Mixing chamber behind flame. Can include advanced RAP entry, which allows for some heating and drying of RAP prior to entering the mixing area. Can be used in conjunction with batch tower (Convertible Ultra Plant) to offer both drum mix and batch plant capabilities.	100–800	
Astec, Inc.	Six pack portable plant	Counter-flow drum mix	Counter-flow double barrel that heats and dries the aggregate in the inner drum and then adds binder, RAP, and filler to the aggregate as it feeds in between the inner and outer drums.	200–400	
	M Pack Relocatable	Counter-flow drum mix	Counter-flow double barrel.	200–600	
	Stationary Drum	Counter-flow drum mix	Counter-flow double barrel.		
	Stationary Batch	Counter-flow batch	Batch plant.	45–500	
	Recycle Batch Plant	Counter-flow batch plant	Batch plant. RAP is mixed with superheated aggregate at the bottom of the hot elevator.		
	Batch drum plant	Counter-flow drum mix/batch	Double-barrel drum is set up along with batch tower. Provides versatility in production by allowing use of batch or DMP.		
	Coater II Plants	Counter-flow with coater for batch or continuous	Batch plant with coater drum. Can mix in batches or produce mix continuously.	Up to 725	
	Heatherington and Berner Batch Plants	Counter-flow batch	Batch plant.		
Terex	E225P/E275P	Counter-flow drum mix	Mixing unit behind flame.		Drum 7 ft diameter by 39 ft 4.5 in.
	E3-300	Counter-flow drum mix	Includes an early-entry RAP preheating zone. Mixing unit is behind the burner.	300	Drum 7 ft 4 in. diameter by 42 ft long
	E3-400	Counter-flow drum mix	Includes an early-entry RAP preheating zone. Mixing unit is behind the burner.	400	Drum 8 ft 4 in. diameter by 47 ft 6 in.
	E3-500	Counter-flow drum mix	Includes an early-entry RAP preheating zone. Mixing unit is behind the burner.	500	Drum 9 ft 3 in. diameter by 47 ft 6 in.

tested during the construction operation, so it is not known if the properties change significantly or not. Surprisingly, there is not much literature that looks at the amount of oxidation, if any, that occurs in the storage silo. Middleton et al. (1967) discusses research from the time when storage silos were first beginning to be used. Middleton et al. found a significant change in binder properties during the mixing operation but only small changes during storage. Storage times up to 100 hours were evaluated. To ensure that any effect of mixture storage is minimized, the storage time should be held constant when the short-term aging characteristics of an asphalt plant are evaluated.

Asphalt Binder Storage

There has not been much emphasis on evaluating the quality of the asphalt binder in the binder storage tank. It is generally assumed that the properties in the tank are approximately the same as the properties of the asphalt binder that is delivered. Tests on the asphalt binder are typically conducted by the material supplier and the results provided to the quality assurance staff of the owner agency. Samples are often taken at the storage tank at the asphalt plant, but in most cases, no testing is done.

Arambula et al. (2005) evaluated different factors that can contribute to the changes in asphalt binder even before it is added to the asphalt mixing operation. The testing plan was designed to simulate the effect of storage time and storage temperature on the dynamic shear rheometer parameter $G^*/\sin\delta$ after RTFO. The plan considered three levels of storage time—1 week, 1 month, and 2 months—and two storage temperature levels—168°C and 191°C. Samples were stored in small containers prepared to simulate the properties at the binder storage tank at the plant. The study found that the material responses to the two levels of storage temperature were statistically significant for 1 week and 1 month, but for 2 months, there was no significant effect. It is generally believed that very little aging occurs in the asphalt binder storage at the plant (stored in large volumes), but based on this study prolonged storage and high temperatures can result in some aging of the binder. Again, this assumes that the small samples tested accurately simulate the conditions of the asphalt binder in much larger tanks. It is not believed that significant aging occurs in the plant binder storage tanks, but the possibility should be considered in any study of short-term binder aging during asphalt mixture production.

Checklist for Mixture to be Collected

There are many variables affecting binder oxidation that cannot be controlled. These mixture properties and plant characteristics, shown in Tables B-2 and B-3, must be measured and documented during production.

Storage of asphalt binder between 300°F and 320°F for not longer than 30 days is recommended. These temperature and time limits are within normal operation procedures and

Table B-2. Mixture properties at mixing plant.

Sample Number	Mixture Type	Agg. Water Abs.	Grad.	Asphalt Content	Agg. Asphalt Abs.	Calculated Film Thickness	Mixing Temp.	RAP %	PG
1									
2									
3									
4									

Table B-3. Plant characteristics.

Plant Type	Set Mixing Time	Mixing Unit Length	Slope Mixing Unit	Revolutions per Minute Mixing Unit	Calculated Mixing Time	Storage Time	Production Rate
Batch	√					√	√
Drum Parallel Flow		√	√	√	√	√	√
Double Barrel		√	√	√	√	√	√
Dual Drum		√	√	√	√	√	√
Mixing Unit Behind Burner		√	√	√	√	√	√
Coater		√	√	√	√	√	√

will help ensure that excess oxidation has not occurred to the asphalt binder at the time of mixing. Storage of asphalt for 6 hours or less is recommended.

The use of RAP can significantly affect the properties of the recovered binder and must be controlled to determine plant effects on the short-term aging of the binder. A realistic approach to account for RAP, which most contractors routinely use, is to document the amount of RAP and, if possible, attempt to compare a mixture with no RAP to that same mixture with RAP.

The use of lime or other anti-stripping agents may significantly affect the aging characteristics. It will be important to monitor their use and document the projects in which they happen.

Mixing time for DMPs should be determined by stopping the addition of asphalt binder and measuring how long it takes for the aggregate going through the plant to show a significant reduction in coating. Alternatively, it can be calculated from the length of the mixing zone and the rate of production.

Summary

From the literature, it is clear that there have not been many studies conducted that document the effect of plant operations on the aging properties of asphalt binders. However, there have been many studies that have evaluated binder aging in general and the research team attempted to adapt the results of these studies to what would be expected to be observed at plants. Further, there has been a lot of testing during construction that allow researchers to have some understanding of the effects of the plant operations on aging.

A total of five plant types were discussed in this report: batch plant, parallel-flow DMP, and three types of counter-flow DMPs. The three types of counter-flow plants included mixing behind burner in drum, unitized plant, and external mixer. Based on the way that each of these plants operate, certain common or unique factors can affect the aging of asphalt binders during the construction process, and these are summarized in checklists provided for the plant and for the asphalt mixture being produced.

It is also possible that the storage of the asphalt binder can result in some aging if it is stored at high temperatures for long periods of time. This is not believed to be a significant problem, but it should be considered during the study if there is reason to believe that it could be a problem. The storage of the asphalt mixture may be a greater concern as the asphalt binder is in thin films when stored with the aggregate and this might expedite aging. Certainly, if this is an issue, it will be affected by storage time and temperature. As a result, the asphalt mixture in the storage silo should be used within a few hours to minimize any potential for aging.

Batch Plant

For a batch plant, the primary items that need to be controlled are mixture temperature and mixing time. These items can be directly controlled and easily measured during the construction process. While the limited literature on storage silos indicates that there is not significant change while being stored, this certainly depends on a number of factors. Hence, placing of asphalt mixture on the same day it is produced is recommended. Using asphalt mixture that was produced on previous days is likely to result in more aging and might present a problem in the analysis of short-term aging during production and placement.

Parallel-Flow Drum Mix Plant

A number of factors can affect the aging that occurs in a parallel-flow DMP beyond those expected to have an effect with the counter-flow DMP. The primary reason for this is that, in a parallel-flow DMP, all of the components are mixed in the drum and exposed to the flame. In the counter-flow plants, methods are used to separate the asphalt binder from the flame and air flow and this should reduce the potential for oxidation.

Mixture temperature and mixing time are important to control. Mixing time is more difficult to measure since it is a continuous-flow operation; however, it can be calculated based on production rates, slope of mixing unit, drum diameter, flight design, and the length of the mixing zone. Also there are times that methods are used inside a drum to slow the flow of material down to get more mixing time. The air flow is believed to affect the aging characteristics in a parallel-flow plant since the air will be flowing through the asphalt binder and will likely increase oxidation. Incomplete ignition of the burner fuel can cause contamination of the asphalt binder resulting in a change in binder properties. While this can be an effect in other plant types, the probability of this being a problem in other plant types is significantly reduced. The flight design can significantly affect aging in a parallel-flow plant through its effect on the veil of aggregate protecting the binder.

It is important to note that parallel-flow plants are becoming a much smaller portion of the total population of asphalt mixture plants and their inclusion in this study proved difficult for that reason.

Counter-Flow Plant

The two primary factors that affect the aging of the asphalt binder in a counter-flow plant are mixture temperature and mixing time. The additional factors that were noted for a parallel-flow plant are not expected to be a significant problem in a counter-flow plant.

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APPENDIX C

Preliminary Laboratory Experiment

A preliminary laboratory experiment was conducted at the beginning of this project to evaluate the effects of binder absorption, aggregate size, binder source, binder grade, and conditioning protocol on the stiffness and strength of asphalt mixtures subjected to various laboratory short-term oven aging (conditioning) protocols. Table C-1 summarizes the factors used in this experiment, and all factors had two levels. Three replicate test specimens for each mixture type were prepared using the Superpave gyratory compactor (SGC) to a target air void (AV) level of 7 ± 0.5 percent. The specimens were subjected to the resilient modulus (M_R) and indirect tensile (IDT) tests. M_R and IDT strength were the test parameters selected as indicators of the mixture stiffness and strength. A statistical analysis was performed on test results to identify the influence of the different factors on the test parameters.

Asphalt absorption by the aggregate can have a significant effect on mixture stiffness and strength. High-absorptive aggregates have the potential of reducing the film thickness of asphalt between aggregate particles and make it more susceptible to certain kinds of distress. As noted in Table C-1, two types of aggregates were investigated in the laboratory study, a high-absorption and a low-absorption aggregate. The high-absorption aggregate was from Georgetown, Texas, with a reported level of water absorption of around 3.5 percent; the low-absorption aggregate was from Brownwood, Texas, with a water absorption value of approximately 0.7 percent.

The nominal maximum aggregate size (NMAS) is also expected to play a role in mixture stiffness and strength as the size of the aggregate usually relates to the size of its pores. The pore size can affect both the quantity of asphalt absorbed and cause preferential absorption for certain asphalt fractions. In the laboratory study, two different NMAS were selected: 9.5 mm and 19 mm. In addition, binder percentages selected from mix designs were different for mixtures with different aggregates and NMAS. Thus, the effect of NMAS on mixture stiffness evaluated in this study might be attributed to different NMAS and binder contents.

Two common binders (Binder A and Binder V) from two different crude oil sources and produced in different refineries in Texas were used in the laboratory study. Binder V was produced from a South American petroleum and Binder A was from a West Texas crude. A previous study showed these binders had different hardening behaviors (Glover 2010). However, since all loose mixtures were short-term conditioned in the oven at 275°F (135°C) for 2 or 4 hours prior to compaction, the unaged stiffnesses of these two binders and the difference in hardening behavior with time could not be captured in this laboratory experiment. Binders with different performance grades (PGs) were included to assess the effect of the polymer modification on the binder oxidation and hardening process. The continuous PG binder grading was not available with the PG 70-22 and PG 64-22 binders used in the study. However, it should be noted that the difference in properties between these two binders may be less than that indicated by their grades.

The laboratory short-term oven aging protocols for the loose mix followed the recommendations of NCHRP Project 9-49. In that study, a comparison of M_R stiffness among laboratory-mixed, laboratory-compacted mixtures; plant-mixed, field-compacted mixtures; and field cores recommended 2 hours at 275°F (135°C) for HMA specimens. Additionally, 4 hours at 275°F (135°C) were found to be an acceptable protocol for some mixture types and, thus, was included in this laboratory experiment, especially since that is the standard loose mix conditioning practice followed by some state DOTs.

Ninety-six SGC specimens for 32 different factor/level combinations were fabricated and subjected to M_R and IDT tests, as shown in Table C-2. All specimens were tested approximately one week after fabrication. Regression analysis was performed to evaluate the effects of different factors on mixture stiffness and strength. The response variables used in the study were M_R and IDT strength. All factors listed in Table C-1 in addition to the two-way interactions among those factors were included.

Table C-1. Factors used in the laboratory experiment.

Factors	Level Values
Aggregate Type (Water Absorption)	Abs-1: 0.7% Abs-2: 3.5%
NMAS	NMAS-1: 9.5 mm NMAS-2: 19 mm
Binder Source	Binder A Binder V
Binder Grade	PG-1: PG 64-22 PG-2: PG 70-22
Conditioning Protocol	STOA-1: 2 h @ 275°F (135°C) STOA-2: 4 h @ 275°F (135°C)

Table C-2. Factor/level combinations used in the laboratory experiment.

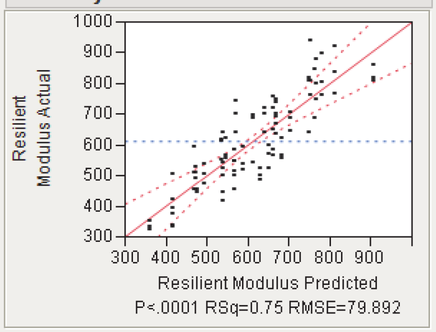
Mixture ID	Binder Absorption	NMAS, mm	Binder Source	Binder Grade	Conditioning Protocol
Low-9.5-A-70-2h	Low-Brownwood	9.5	Binder A	PG 70-22	2 h @ 275°F (135°C)
Low-9.5-A-64-2h	Low-Brownwood	9.5	Binder A	PG 64-22	2 h @ 275°F (135°C)
Low-9.5-A-70-4h	Low-Brownwood	9.5	Binder A	PG 70-22	4 h @ 275°F (135°C)
Low-9.5-A-64-4h	Low-Brownwood	9.5	Binder A	PG 64-22	4 h @ 275°F (135°C)
Low-19-A-70-2h	Low-Brownwood	19.0	Binder A	PG 70-22	2 h @ 275°F (135°C)
Low-19-A-64-2h	Low-Brownwood	19.0	Binder A	PG 64-22	2 h @ 275°F (135°C)
Low-19-A-70-4h	Low-Brownwood	19.0	Binder A	PG 70-22	4 h @ 275°F (135°C)
Low-19-A-64-4h	Low-Brownwood	19.0	Binder A	PG 64-22	4 h @ 275°F (135°C)
Low-9.5-V-70-2h	Low-Brownwood	9.5	Binder V	PG 70-22	2 h @ 275°F (135°C)
Low-9.5-V-64-2h	Low-Brownwood	9.5	Binder V	PG 64-22	2 h @ 275°F (135°C)
Low-9.5-V-70-4h	Low-Brownwood	9.5	Binder V	PG 70-22	4 h @ 275°F (135°C)
Low-9.5-V-64-4h	Low-Brownwood	9.5	Binder V	PG 64-22	4 h @ 275°F (135°C)
Low-19-V-70-2h	Low-Brownwood	19.0	Binder V	PG 70-22	2 h @ 275°F (135°C)
Low-19-V-64-2h	Low-Brownwood	19.0	Binder V	PG 64-22	2 h @ 275°F (135°C)
Low-19-V-70-4h	Low-Brownwood	19.0	Binder V	PG 70-22	4 h @ 275°F (135°C)
Low-19-V-64-4h	Low-Brownwood	19.0	Binder V	PG 64-22	4 h @ 275°F (135°C)
High-9.5-A-70-2h	High-Georgetown	9.5	Binder A	PG 70-22	2 h @ 275°F (135°C)
High-9.5-A-64-2h	High-Georgetown	9.5	Binder A	PG 64-22	2 h @ 275°F (135°C)
High-9.5-A-70-4h	High-Georgetown	9.5	Binder A	PG 70-22	4 h @ 275°F (135°C)
High-9.5-A-64-4h	High-Georgetown	9.5	Binder A	PG 64-22	4 h @ 275°F (135°C)
High-19-A-70-2h	High-Georgetown	19.0	Binder A	PG 70-22	2 h @ 275°F (135°C)
High-19-A-64-2h	High-Georgetown	19.0	Binder A	PG 64-22	2 h @ 275°F (135°C)
High-19-A-70-4h	High-Georgetown	19.0	Binder A	PG 70-22	4 h @ 275°F (135°C)
High-19-A-64-4h	High-Georgetown	19.0	Binder A	PG 64-22	4 h @ 275°F (135°C)
High-9.5-V-70-2h	High-Georgetown	9.5	Binder V	PG 70-22	2 h @ 275°F (135°C)
High-9.5-V-64-2h	High-Georgetown	9.5	Binder V	PG 64-22	2 h @ 275°F (135°C)
High-9.5-V-70-4h	High-Georgetown	9.5	Binder V	PG 70-22	4 h @ 275°F (135°C)
High-9.5-V-64-4h	High-Georgetown	9.5	Binder V	PG 64-22	4 h @ 275°F (135°C)
High-19-V-70-2h	High-Georgetown	19.0	Binder V	PG 70-22	2 h @ 275°F (135°C)
High-19-V-64-2h	High-Georgetown	19.0	Binder V	PG 64-22	2 h @ 275°F (135°C)
High-19-V-70-4h	High-Georgetown	19.0	Binder V	PG 70-22	4 h @ 275°F (135°C)
High-19-V-64-4h	High-Georgetown	19.0	Binder V	PG 64-22	4 h @ 275°F (135°C)

The statistical analysis results by JMP statistical package (SAS product) for M_R as the response variable are presented in Figure C-1. As indicated by the "Effect Tests" table, among the 15 factor/level combinations considered, the main effects of binder absorption, NMAS, binder source, binder grade, and conditioning protocol on M_R were statistically significant at $\alpha = 0.05$. In addition, the effect on M_R from the two-way interactions between NMAS and binder source and between NMAS and binder grade were significant. Comparisons in terms of least squares means for significant factors of interests are presented in Table C-3. It can be observed from the table that, in general, mixtures with higher absorptive aggregate, larger NMAS, binder A, and longer short-term conditioning time may result in higher M_R values.

In the following five figures that present the difference in M_R results of asphalt mixtures due to different levels of binder absorption, NMAS, binder source, binder grade, and conditioning protocol, each bar represents the average value of three replicate specimens, and the error bars represent ± one standard deviation from the average value.

Response Resilient Modulus

Actual by Predicted Plot



Summary of Fit

RSquare	0.748566
RSquare Adj	0.700826
Root Mean Square Error	79.89199
Mean of Response	612.1905
Observations (or Sum Wgts)	95

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Ratio
Model	15	1501206.9	100080	15.6799
Error	79	504235.6	6383	Prob > F
C. Total	94	2005442.5		<.0001*

Lack Of Fit

Parameter Estimates

Effect Tests

Source	Nparm	DF	Sum of Squares	F Ratio	Prob > F
Binder Absorption	1	1	275506.19	43.1643	<.0001*
Max Agg Size	1	1	179624.99	28.1423	<.0001*
Binder Source	1	1	625390.39	97.9817	<.0001*
Binder Grade	1	1	45471.43	7.1241	0.0092*
Conditioning Protocols	1	1	190763.11	29.8874	<.0001*
Binder Absorption*Max Agg Size	1	1	1798.11	0.2817	0.5971
Binder Absorption*Binder Source	1	1	2149.24	0.3367	0.5634
Binder Absorption*Binder Grade	1	1	19898.67	3.1176	0.0813
Binder Absorption*Conditioning Protocols	1	1	525.76	0.0824	0.7749
Max Agg Size*Binder Source	1	1	82777.21	12.9689	0.0006*
Max Agg Size*Binder Grade	1	1	58037.84	9.0930	0.0034*
Max Agg Size*Conditioning Protocols	1	1	1941.95	0.3043	0.5828
Binder Source*Binder Grade	1	1	10029.17	1.5713	0.2137
Binder Source*Conditioning Protocols	1	1	10911.70	1.7096	0.1948
Binder Grade*Conditioning Protocols	1	1	6.50	0.0010	0.9746

Figure C-1. JMP output of fitting model for resilient modulus.

Table C-3. Least squares means results for resilient modulus.

Factors	Level Values	Least Squares Mean (ksi)
Binder Absorption	Brownwood	559.9
	Georgetown	667.7
NMAS	9.5 mm	570.2
	19 mm	657.3
Binder Source	Binder A	695.0
	Binder V	532.5
Binder Grade	PG 70-22	591.9
	PG 64-22	635.6
Conditioning Protocols	2 h @ 135°C (275°F)	568.9
	4 h @ 135°C (275°F)	658.6

As illustrated in Figure C-2, for all mixtures, the stiffness of mixtures with higher absorptive aggregates was higher than or equivalent to that of mixtures with lower absorptive aggregates. Since two types of aggregates had different levels of binder absorption, mixtures with those two aggregates were designed differently and, thus, with different optimum binder contents. A study at Virginia Polytechnic Institute and State University evaluated the effect of binder film thickness on mixture stiffness (Wang 2007). In the study, samples of two hemispheroid particles bonded by binder with different film thicknesses were fabricated and then tested under a direct compression condition. The vertical displacement and the resistance force of samples with different binder film thicknesses were recorded during the test. Test results indicated that the resistance force of the sample with thinner binder film thickness was higher than that of the sample with thicker binder film thickness. Therefore, it was concluded that asphalt mixtures with a thinner binder film thickness were stiffer than those with a thicker binder film thickness. The effective binder film thickness (FT_{be}) of all mixtures in this study were calculated based on mixture volumetrics and are shown in Table C-4. As inferred from the table, effective binder film thickness of mixtures with higher absorptive aggregates was thinner than that of mixtures with lower absorptive aggregates, when all other factors are kept constant. Therefore, the higher M_R values of mixtures with higher absorptive aggregates might be attributed to the thinner effective binder film thickness in the mixture.

The effect of NMAS on mixture M_R results is shown in Figure C-3. As illustrated, for the majority of the mixtures, the stiffness of mixtures with NMAS of 0.75 in. (19 mm) was higher than or equivalent to those of mixtures with NMAS of 0.37 in. (9.5 mm). Since the binder content used in mixtures with NMAS of 0.75 in. (19 mm) was lower than that used in mixtures with NMAS of 0.37 in. (9.5 mm), and the binder content had a significant effect on mixture stiffness (Hamzah and Yi 2008), it was anticipated that the observed differences in M_R results had a stronger correlation with binder content.

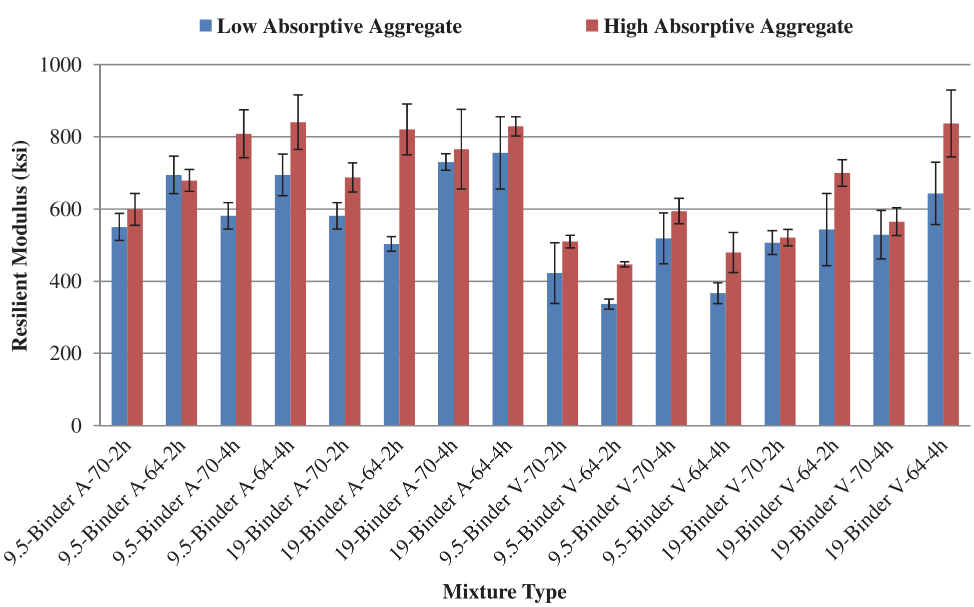


Figure C-2. Effect of binder absorption on mixture M_R stiffness.

Table C-4. Calculated effective binder film thickness results.

Mixture Label	Binder Absorption	FT_{be}
9.5-A-70-2h	Low-Brownwood High-Georgetown	18.8 14.6
9.5-A-64-2h	Low-Brownwood High-Georgetown	16.8 14.4
9.5-A-70-4h	Low-Brownwood High-Georgetown	18.8 14.6
9.5-A-64-4h	Low-Brownwood High-Georgetown	16.8 14.4
19-A-70-2h	Low-Brownwood High-Georgetown	19.6 14.3
19-A-64-2h	Low-Brownwood High-Georgetown	20.0 13.6
19-A-70-4h	Low-Brownwood High-Georgetown	19.6 14.3
19-A-64-4h	Low-Brownwood High-Georgetown	20.0 13.6
9.5-V-70-2h	Low-Brownwood High-Georgetown	19.1 14.6
9.5-V-64-2h	Low-Brownwood High-Georgetown	18.9 14.4
9.5-V-70-4h	Low-Brownwood High-Georgetown	19.1 14.6
9.5-V-64-4h	Low-Brownwood High-Georgetown	18.9 14.4
19-V-70-2h	Low-Brownwood High-Georgetown	20.1 13.7
19-V-64-2h	Low-Brownwood High-Georgetown	19.0 14.4
19-V-70-4h	Low-Brownwood High-Georgetown	20.1 13.7
19-V-64-4h	Low-Brownwood High-Georgetown	19.0 14.4

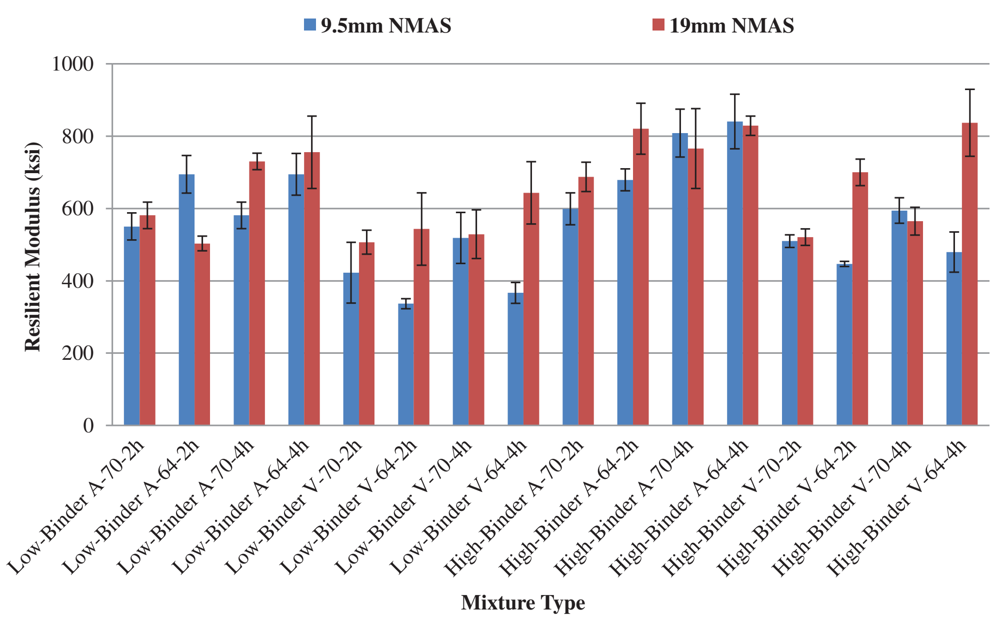


Figure C-3. Effect of NMAS on mixture M_R stiffness.

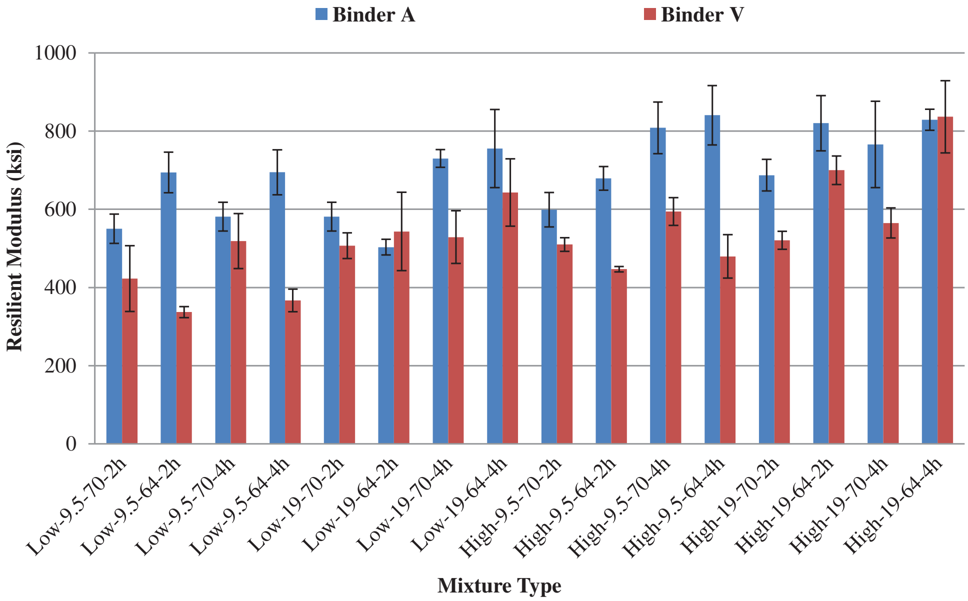


Figure C-4. Effect of binder source on mixture M_R stiffness.

The effect of binder source on mixture M_R results is illustrated in Figure C-4. All mixtures with Binder A were significantly stiffer than or equivalent to those made with Binder V. The effect of binder grade on mixture M_R values, as indicated in Figure C-5, was not consistent for all mixtures. In some cases, the incorporation of higher PG binders in the mixtures increased the mixture stiffness significantly, while the opposite trend was observed for other cases. Therefore, it was anticipated that the interaction between binder grade and other factors had a significant effect on the mixture M_R values.

Figure C-6 presents the effect of conditioning protocol on mixture M_R values. As anticipated, the stiffness of all mixtures conditioned with longer conditioning times were significantly higher or equivalent to those with shorter conditioning times.

The ranking of significant factors in terms of their influence effects on mixture M_R values was proposed as an indicator of which factors had the biggest influence in stiffness. This is obtained from the "Magnitude of Parameter Estimate" output from JMP listed in Table C-5.

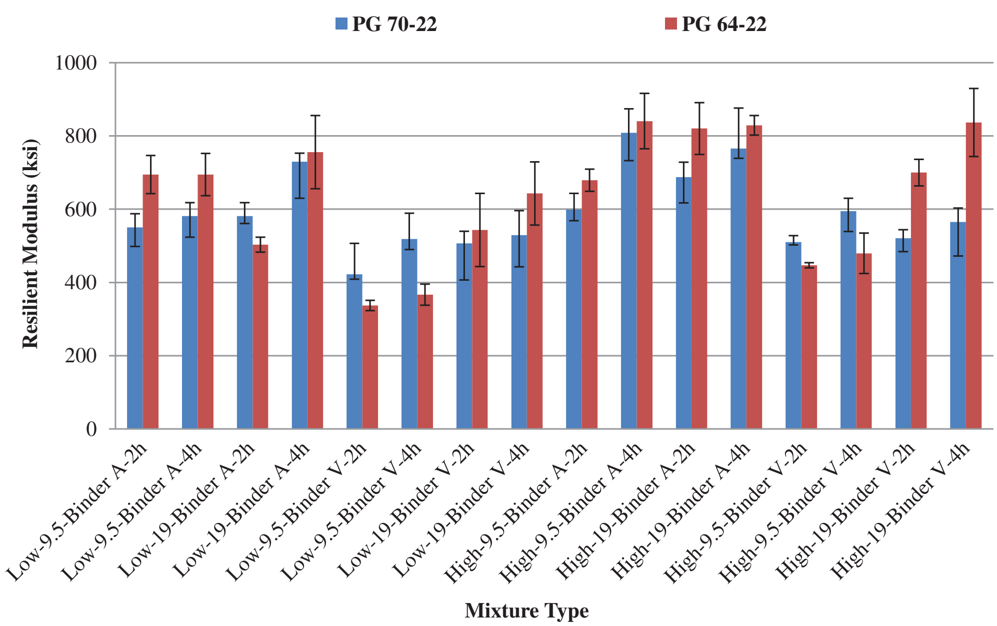


Figure C-5. Effect of binder grade on mixture M_R stiffness.

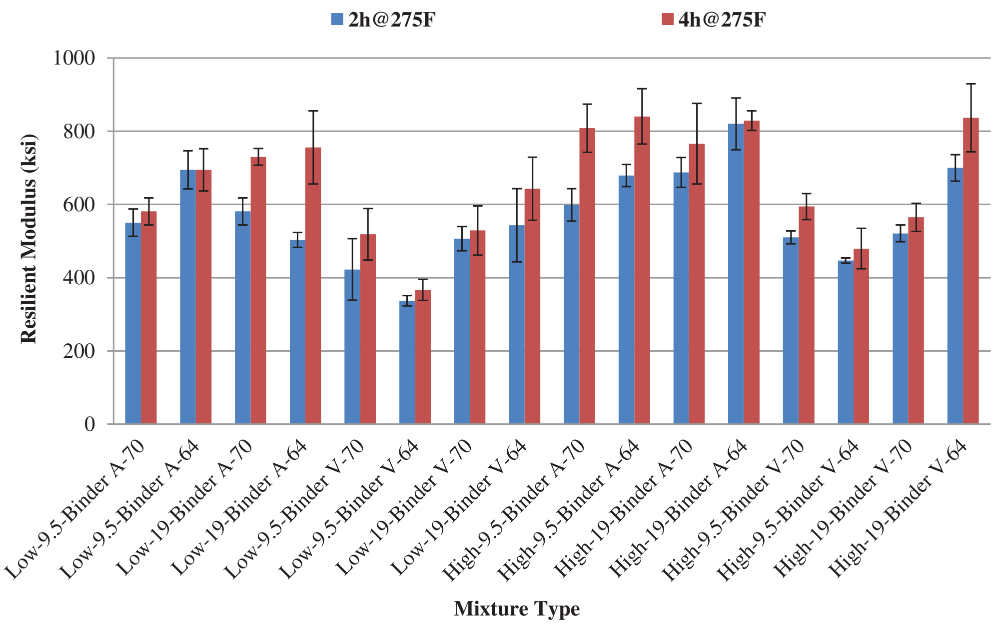


Figure C-6. Effect of conditioning protocol on mixture M_R stiffness.

The statistical analysis results for the IDT strength obtained by JMP are shown in Figure C-7. As indicated by the "Effect Tests" table in Figure C-7, among the 15 factor/level combinations considered, the main effects of binder absorption, binder source, binder grade, conditioning protocols, and the two-way interactions between binder absorption and NMAS, NMAS and binder source, and NMAS and binder grade on IDT strength are significant at $\alpha = 0.05$. Comparisons in terms of least squares means for significant factors of interests are presented in Table C-6. It can be observed from the table that, in general, higher absorptive aggregate, higher binder grade, and longer conditioning protocol may contribute to higher mixture IDT strength. Additionally, asphalt mixtures with Binder A are stronger than those that employ Binder V when all other factors are kept constant.

In the following five figures that present the difference in IDT strengths of asphalt mixtures due to different levels of binder absorption, NMAS, binder source, binder grade, and conditioning protocol, each bar represents the average value of three replicate specimens, and the error bars represent ± one standard deviation from the average value.

The effect of binder absorption on mixture IDT results, as shown in Figure C-8, was significant. More specifically, for the majority of the mixtures, those with higher absorptive aggregates are stiffer than those with lower absorptive aggregates, which is consistent with the trend obtained from the M_R stiffness results.

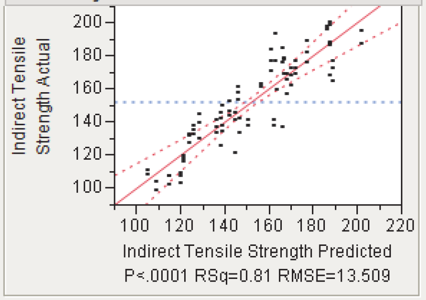
As illustrated in Figure C-9, the effect of NMAS on mixture IDT strength was not consistent for all mixtures. In some cases, the incorporation of aggregates with larger NMAS in

Table C-5. Linear fitting results for resilient modulus.

Factors	Magnitude of Parameter Estimate	Influence Effect Ranking
Binder Source	81.2	Significant 1st
Binder Absorption	53.9	Significant 2nd
Conditioning Protocol	44.9	Significant 3rd
NMAS	43.5	Significant 4th
NMAS & Binder Source	29.5	Significant 5th
NMAS & Binder Grade	24.7	Significant 6th
Binder Grade	21.9	Significant 7th

Response Indirect Tensile Strength

Actual by Predicted Plot



Summary of Fit

RSquare	0.806953
RSquare Adj	0.770757
Root Mean Square Error	13.50895
Mean of Response	152.1365
Observations (or Sum Wgts)	96

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Ratio
Model	15	61026.647	4068.44	22.2939
Error	80	14599.335	182.49	Prob > F
C. Total	95	75625.982		<.0001*

Lack Of Fit

Parameter Estimates

Effect Tests

Source	Nparm	DF	Sum of Squares	F Ratio	Prob > F
Binder Absorption	1	1	841.943	4.6136	0.0347*
Max Agg Size	1	1	309.243	1.6946	0.1967
Binder Source	1	1	37957.283	207.9946	<.0001*
Binder Grade	1	1	4340.315	23.7836	<.0001*
Conditioning Protocols	1	1	6024.586	33.0129	<.0001*
Binder Absorption*Max Agg Size	1	1	5820.378	31.8939	<.0001*
Binder Absorption*Binder Source	1	1	429.683	2.3545	0.1289
Binder Absorption*Binder Grade	1	1	452.836	2.4814	0.1191
Binder Absorption*Conditioning Protocols	1	1	181.775	0.9961	0.3213
Max Agg Size*Binder Source	1	1	2933.776	16.0762	0.0001*
Max Agg Size*Binder Grade	1	1	1617.863	8.8654	0.0038*
Max Agg Size*Conditioning Protocols	1	1	37.625	0.2062	0.6510
Binder Source*Binder Grade	1	1	8.108	0.0444	0.8336
Binder Source*Conditioning Protocols	1	1	61.921	0.3393	0.5619
Binder Grade*Conditioning Protocols	1	1	9.313	0.0510	0.8219

Figure C-7. JMP output of fitting model for IDT strength.

Table C-6. Least squares means results for IDT strength.

Factors	Level Values	Least Squares Mean (psi)
Binder Absorption	Brownwood	149.2
	Georgetown	155.1
NMAS	9.5 mm	150.3
	19 mm	153.9
Binder Source	Binder A	172.0
	Binder V	132.3
Binder Grade	PG 70-22	158.9
	PG 64-22	145.4
Conditioning Protocols	2 h @ 275°F (135°C)	144.2
	4 h @ 275°F (135°C)	160.1

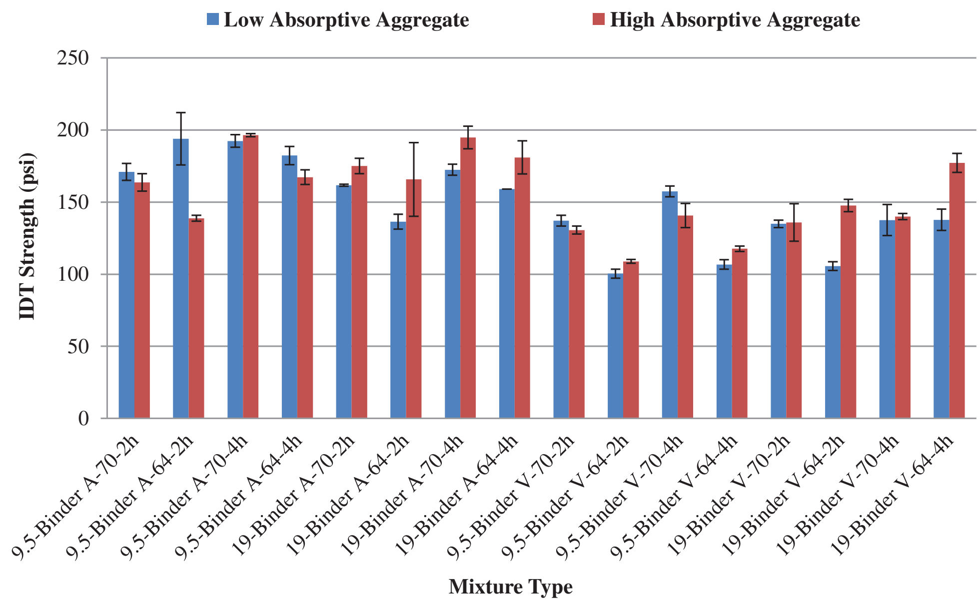


Figure C-8. Effect of binder absorption on mixture IDT strength.

the mixtures increased the mixture stiffness, while the opposite trend was observed for other cases. Therefore, it was anticipated that a more significant effect of NMAS on mixture IDT strength might be associated with the interaction with other main factors.

The effect of binder source on mixture IDT strength is illustrated in Figure C-10. All mixtures prepared using Binder A were significantly stronger than those prepared with Binder V. The effect of binder grade on mixture IDT strength is illustrated in Figure C-11. For the majority of the mixtures, the IDT strengths of mixtures with PG 70-22 were higher than those of mixtures with PG 64-22, for both Binder A and Binder V. It further verified the significant effect of binder stiffness on mixture property.

Figure C-12 presents the effect of conditioning protocol on mixture IDT strength results. As expected, all mixtures with longer conditioning (except that with "Low-9.5-Binder A-64") had IDT strengths that were higher or equivalent to those

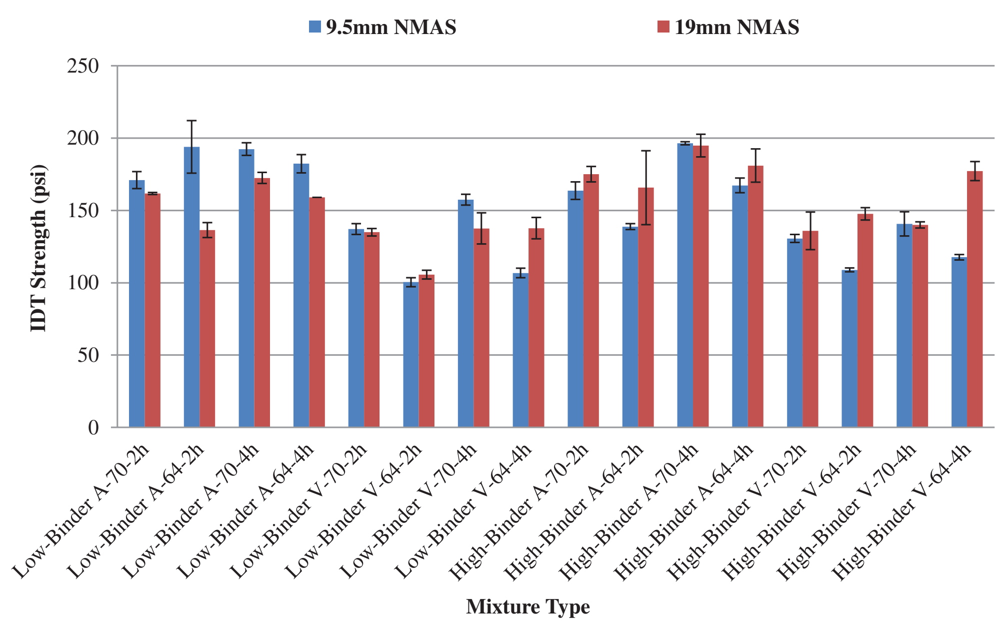


Figure C-9. Effect of NMAS on mixture IDT strength.

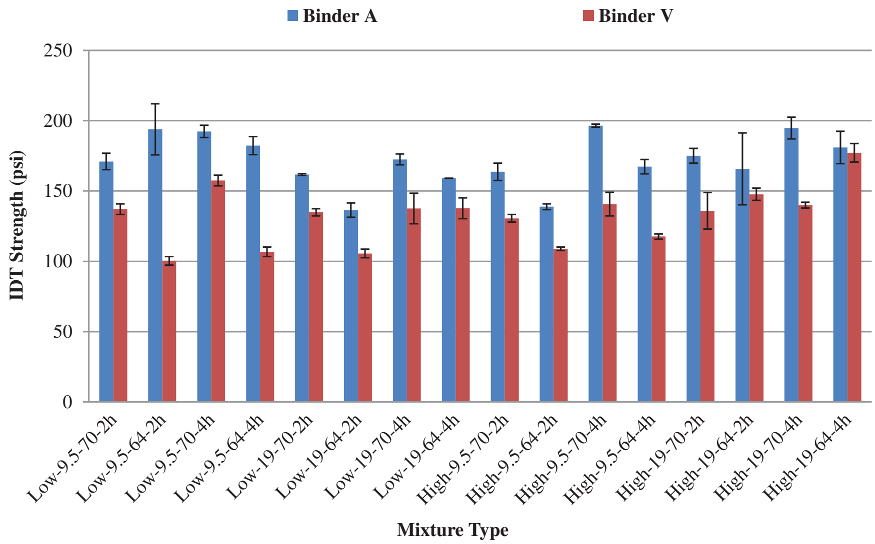


Figure C-10. Effect of binder source on mixture IDT strength.

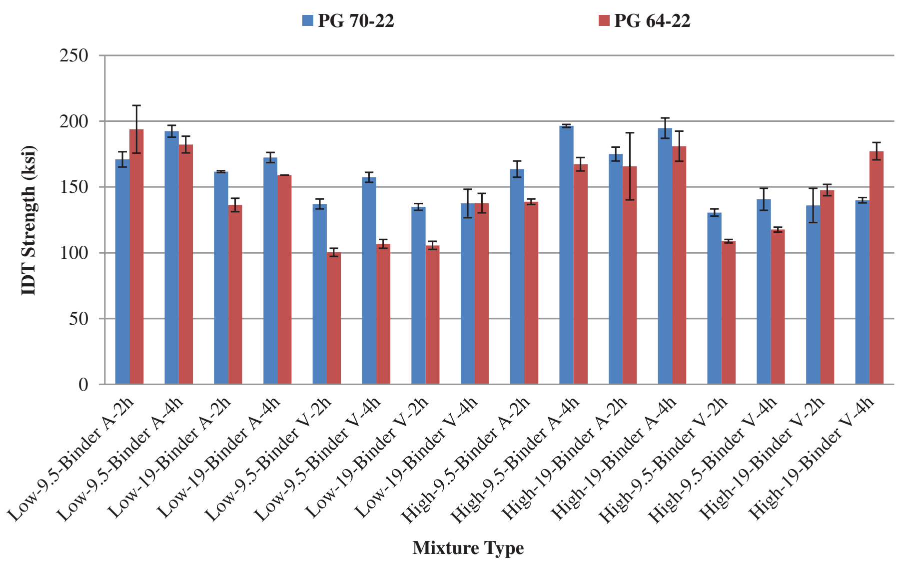


Figure C-11. Effect of binder grade on mixture IDT strength.

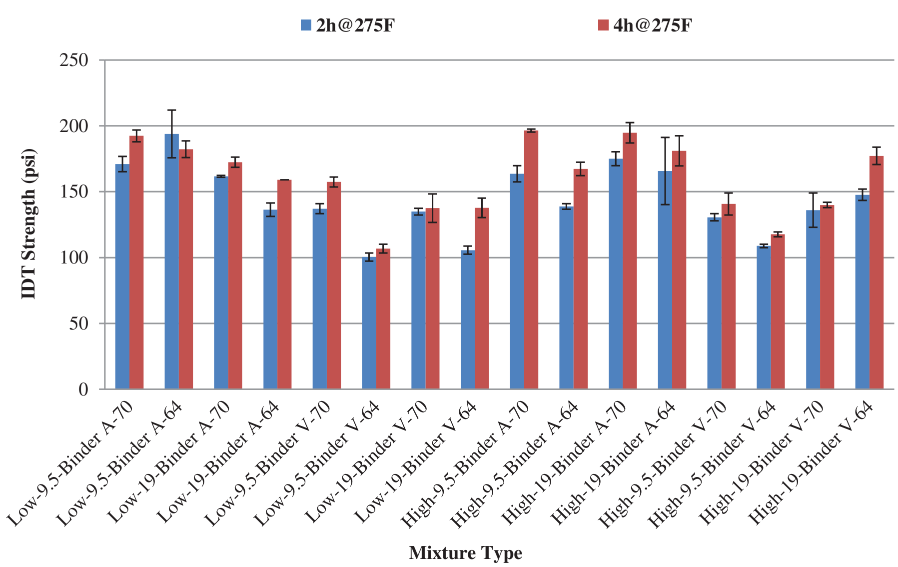


Figure C-12. Effect of conditioning protocol on mixture IDT strength.

Table C-7. Linear fitting results for IDT strength.

Factors	Level Values	Least Squares Mean (psi)
Binder Absorption	Brownwood	149.2
	Georgetown	155.1
NMAS	9.5 mm	150.3
	19 mm	153.9
Binder Source	WT	172.0
	SA	132.3
Binder Grade	PG 70-22	158.9
	PG 64-22	145.4
Conditioning Protocols	2 h @ 275°F (135°C)	144.2
	4 h @ 275°F (135°C)	160.1

with less conditioning time. Therefore, the extended short-term conditioning for loose mix was able to significantly increase the mixture stiffness and strength.

The ranking of significant factors in terms of their influence effects on mixture IDT strength was proposed based on the "Magnitude of Parameter Estimate" output from JMP listed in Table C-7.

On the basis of the statistical analysis of the laboratory test results, it can be concluded that binder source has a profound effect on both M_R and IDT strength. Specifically, mixtures with Binder A were stiffer than those with Binder V. There was also evidence that mixture M_R and IDT strength were sensitive to the laboratory conditioning protocol of the loose mix. NMAS was verified to have a significant effect on mixture stiffness while it showed no significant effect on mixture strength, and the difference was likely due to the compound effect of NMAS and binder content in the mix. The effects of binder grade and binder absorption on mixture M_R stiffness and IDT strength were significant. Additionally, two-way interactions between NMAS and binder source, and between NMAS and binder grade, also had a significant effect on mixture stiffness and strength, based on the statistical analysis.

Based on the findings from the small laboratory parametric study, there is strong evidence that mixture aging characteristics are strongly influenced by asphalt source and binder absorption by aggregate. NMAS and binder grade have lesser effects on mixture aging, and it is suspected that the effects of NMAS may be tied to mixture volumetrics. The laboratory conditioning protocol is of particular importance to the mixture stiffness and strength.

In the field trials, the research team made a particular effort to incorporate the mixture variables of binder source and aggregate absorption into the testing matrix as well as the plant characteristics of temperature and design. The resulting aging protocols for short-term and intermediate-term aging reflect the differences in these variables, which span a wide range of field and mixture conditions.

References

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Hamzah, M. O. and T. C. Yi (2008). "Effects of Temperature on Resilient Modulus of Dense Asphalt Mixtures Incorporating Steel Slag Subjected to Short Term Oven Ageing." World Academy of Science, Engineering and Technology, Vol. 2, No. 10, pp. 171–175.

Wang, D. (2007). "Binder Film Thickness Effect on Aggregate Contact Behavior." Master Thesis, Virginia Polytechnic Institute and State University, Blacksburg, VA.

APPENDIX D

Round Robin Study

This appendix summarizes the results of a round robin study between the University of California Pavement Research Center (UCPRC), Texas A&M Transportation Institute (TTI), and National Center for Asphalt Technology (NCAT) to assess the consistency, repeatability, and reproducibility of laboratory testing results between the different laboratories participating in this project. Results from the study indicate that specimen fabrication was generally consistent between the three laboratories. However, some of the results were inconsistent indicating that aspects of the testing procedures might be open to interpretation and hence followed differently between the laboratories. A few of the results (specific tests within individual laboratories) also appeared to have high variation in the results across the five specimens tested. The results of the round robin testing and the test procedures were reviewed to assess possible reasons for variability within and between laboratories. It was found that all of the laboratories were performing the tests in a similar manner, except for a few minor details. Appropriate statistical approaches for analyzing the results of field testing to meet the objectives of the project were also identified. The conclusion of the study was that project testing should proceed as planned.

Scope of the Study

To ensure consistency, repeatability, and reproducibility of laboratory testing results between the different laboratories participating in this project, round robin testing was conducted prior to starting the planned project testing. Given that the three laboratories participating in the study (UCPRC, TTI, and NCAT) all have AASHTO Materials Reference Laboratory (AMRL) accreditation for standard hot mix asphalt tests, only those tests that are not part of the AMRL accreditation system were evaluated. Tests included in the round robin testing program included unconfined flow number, resilient modulus (M_R), and dynamic modulus (E^*)/phase angle.

The round robin employed aggregate batches arranged by the UCPRC and sent to the participating laboratories for preparation and testing of specimens (referred to as in-laboratory fabricated in this appendix), as well as testing of specimens prepared and compacted at the UCPRC and sent to participating laboratories (referred to as prefabricated in this appendix). Testing of specimens prepared from supplied aggregate batches and binder assessed consistency, repeatability, and reproducibility between participating laboratories of mixing, compacting (gyratory), sawing/trimming, air-void (AV) content testing, and performance-related testing. Testing of specimens prepared at the UCPRC and sent to participating laboratories assessed the performance-related tests only. All results were submitted to the UCPRC for analysis and reporting.

The following tasks were completed to achieve the study's objective:

1. Prepare a test plan for specimen preparation, testing, and reporting.
2. Collect aggregate and binder samples. Prepare material batches and specimens for all laboratories.
3. Perform unconfined flow number, M_R, E^*/phase angle tests according to the test plan. Submit test results for analysis.
4. Analyze the results.

The results of the analysis were reviewed to identify potential sources of error and differences in practice due to possible gaps or different interpretations of the test procedure.

Table D-1. Specimen preparation parameters.

Temperature		Compaction
Binder/Mixture	145°C	M_R (61 x 150 mm) AV = 7.0 ± 0.5% (as compacted), Height ± 2.5 mm
Aggregate	160°C	E* (150 x 100 mm) AV = 7.0 ± 0.5% (cut specimen), Height ± 2.5 mm
Cure Mix 4 h	135°C	Flow # (150 x 100 mm) AV= 7.0 ± 0.5% (cut specimen), Height ± 2.5 mm
Compact	140°C	Compaction using height control mode
		Binder Content 5% by Dry Weight of Aggregate
		Aging/Curing of mix is per PP 60-09 (R 30 short-term aging)

Table D-2. Asphalt Mixture Performance Tester sample fabrication tolerances (AASHTO PP 60-09).

Parameter	Acceptable Tolerance
Air-Void Tolerance	7.0 ± 0.5%
Average Sample Diameter	100 to 104 mm
Standard Deviation of Sample Diameter	≤ 0.5 mm
Sample Height	147.5 to 152.5 mm
End Flatness	≤ 0.5 mm
End Perpendicularity	≤ 1.0 mm

Specimen Preparation and Testing Plan

One set of five test specimens for each test was produced by each laboratory from aggregate/binder material provided by UCPRC and a set of five test specimens for each test was prepared by UCPRC. All specimens were prepared from a California crushed granite aggregate source and a California coastal crude based PG 64-16 binder.

Tables D-1 and D-2 show the specimen preparation temperatures, dimensions, AV, and tolerances used by UCPRC for fabricating specimens. NCAT and TTI followed these guidelines for consistent specimen preparation among all three laboratories.

The flow number test was conducted according to NCHRP TP 79-12 with NCHRP 09-33 and 09-43 recommended parameters and modifications, as shown below:

- Test temperature: 55°C
- 30 kPa (4.35 psi) contact stress
- 600 kPa (87 psi) deviator stress
- Unconfined (0 psi)
- Five replicates each for both the laboratory-mixed, laboratory-compacted (LMLC) samples and the prefabricated samples
- Test ending criteria: 10,000 loading cycles or accumulated 30,000 microstrain
- Data analysis in accordance with AASHTO TP 79-12 (Francken model)

The output of the flow number test was reported according to Section 10.5 in TP 79-12. The flow number was calculated following the Francken model.

Resilient modulus or M_R stiffness was measured in accordance with the current ASTM D7369. TTI and NCAT replaced the on-specimen linear variable differential transducers (LVDT) setup with external LVDT aligned along the horizontal diametral plane (i.e., gauge length as a fraction of diameter of the specimen = 1.00). The M_R test was conducted through repetitive applications of compressive loads in a haversine waveform along a vertical diametral plane of cylindrical asphalt concrete specimens.

Testing parameters for the E* test are shown in Table D-3. Testing was performed in accordance with AASHTO TP 79-12 and under unconfined conditions (0 psi). Five replicates for each of the LMLC samples and the prefabricated samples were tested. Data analysis and presentation were done in accordance with Section 11 of AASHTO PP 61-10.

Data Analysis

The unconfined flow number, M_R, E*, phase angle, and AV content data from the specimens prepared from the supplied loose aggregate/binder samples were analyzed in accordance with ASTM E691-13 (Standard Practice for Conducting an Interlaboratory Study to Determine the Precision of a Test Method) to develop precision statements for each test. Following the same analysis procedure, the data

Table D-3. Temperatures and frequencies for E* testing.

Test Temperature (°C)	Loading Frequencies (Hz)
4.0	10, 1, 0.1
20.0	10, 1, 0.1
40.0 (for PG 64-16 binder)	10, 1, 0.1, 0.01

from the pre-fabricated specimens were analyzed to determine to what extent sample fabrication affected the precision of the tests. It should be noted that the purpose of the analysis was not to provide final statements of precision of the test methods. Since only three laboratories were involved in the study, conclusions regarding precision and bias in test results are not appropriate in that this study did not provide sufficient statistical evidence to identify laboratories with problematic specimen preparation and testing methods. However, it was possible to identify the levels of differences in test results based on the results of the analysis.

The following steps were followed to analyze the test results. Equations were adapted from ASTM E691 and *NCHRP Report 702* (Bonaquist 2011).

Step 1. Compute Consistency and Initial Estimate of Repeatability and Reproducibility Statistics

The following equations were used to compute the consistency and initial estimate of repeatability and reproducibility statistics:

$$s_r = \sqrt{\sum_1^p \frac{s^2}{p}} \qquad \text{Eq. (D-1)}$$

Where:
s_r = repeatability standard deviation,
s = within-laboratory standard deviation, and
p = number of laboratories in the interlaboratory study.

$$s_r\% = \frac{s_r}{\overline{\overline{X}}} \times 100 \qquad \text{Eq. (D-2)}$$

Where:
$s_r\%$ = repeatability coefficient of variation,
s_r = repeatability standard deviation, and
$\overline{X}$ = average of the laboratory averages.

$$s_R = \sqrt{(s_x)^2 + (s_r)^2\left(\frac{n-1}{n}\right)} \qquad \text{Eq. (D-3)}$$

Where:
s_R = reproducibility standard deviation,
s_x = standard deviation of the laboratory averages,
s_r = repeatability standard deviation, and
n = number of tests in each laboratory.

$$s_R\% = \frac{s_R}{\overline{\overline{X}}} \times 100 \qquad \text{Eq. (D-4)}$$

Where:
$s_R\%$ = reproducibility coefficient of variation,
s_R = reproducibility standard deviation, and
$\overline{X}$ = average of the laboratory averages.

$$k = \frac{s}{s_r} \qquad \text{Eq. (D-5)}$$

Where:
k = within-laboratory consistency statistic,
s = within-laboratory standard deviation, and
s_r = repeatability standard deviation.

$$h = \frac{(\bar{x} - \overline{X})}{s_x} \qquad \text{Eq. (D-6)}$$

Where:
h = between-laboratory consistency statistic,
s_x = standard deviation of the laboratory averages,
$\bar{x}$ = laboratory average, and
$\overline{X}$ = average of the laboratory averages.

Step 2. Evaluate the Consistency Statistics to Identify Questionable Data Groups

The consistency statistics, k and h, were used to evaluate the consistency of the data collected. The k statistic is a measure of consistency of the data within a laboratory, and the h statistic is a measure of the consistency of the data between laboratories. These values were compared to critical values that depend on the number of laboratories and number of tests per laboratory that were conducted. For the design used in this interlaboratory study (three laboratories with five tests per laboratory), the critical values of k and h are 1.56 and 1.15, respectively (ASTM E691). Plots of the k and h statistics were also used to identify data requiring further review and to understand the characteristics of the test variability.

Step 3. Investigate Difference in Between-Laboratory Test Results

Differences in between-laboratory test results were investigated using the Welch modified two-sample *t*-test, which is recommended for evaluating small datasets (Ruxton 2006). This method assumes unequal dataset variances and, if F1 and F2 are two distributions, the possible hypotheses and alternatives concerning these distributions are:

- H_0: $F_1(x) = F_2(x)$
- H_A: $F_1(x) \neq F_2(x)$

- Decision rule: Reject H_0 if p-value < 0.10; accept H_0 if p-value ≥ 0.10

For example, if the p-value from the Welch modified two-sample t-test for two distributions is equal to 0.07, H_0 will be rejected. In other words, distributions of the two test results are not equal. p-values closer to one suggest that the selected two test result distributions are similar for the laboratory test under consideration.

Step 4. Evaluate Trends in the Repeatability and Reproducibility Statistics

Repeatability is the variability between independent test results on the same material obtained in a single laboratory. Reproducibility is the variability between independent test results on the same material obtained in different laboratories. Repeatability and reproducibility statistics were calculated to identify trends in these statistics so that appropriate precision statements could be developed.

Step 5. Evaluate the Effect of AV Content on Test Results

The effects of AV content variability on test result precision and bias were also investigated.

Laboratory Results

Test result summaries are provided in this section. The laboratories are identified in the figures as follows:

- Lab 1: UCPRC
- Lab 2: TTI
- Lab 3: NCAT

Unconfined Flow Number

The unconfined flow number tests were conducted according to the AASHTO TP 79-12 test method, with NCHRP 09-33 and NCHRP 09-43 recommended parameters and modifications as previously mentioned.

Within-laboratory (k) consistency statistics for the unconfined flow number tests on in-laboratory fabricated and prefabricated specimens are shown in Figure D-1. None of the data exceeded the limit for the k statistic. However, due to the high variability in the UCPRC test results, within-laboratory consistency appeared to be lower. TTI and NCAT had closer within-laboratory consistency statistics, compared to the UCPRC statistics. The coefficients of variation of the flow number test results were also calculated to further investigate the levels of within-laboratory variability. Results are shown in Figure D-2. The test result variability for UCPRC was significantly higher than those for TTI and NCAT.

Between-laboratory (h) consistency statistics for the prefabricated and in-laboratory fabricated specimens are shown in Figure D-3. None of the data exceeded the limits for the h statistic. However, the h statistic for TTI appeared to be close to the limit, which suggests that the TTI flow number test results were different from the UCPRC and NCAT test results.

Differences in between-laboratory test results are listed in Table D-4. For the in-laboratory fabricated specimens, unconfined flow number test results from UCPRC can be considered as similar to those from NCAT. The results from TTI were statistically different to those from the UCPRC and NCAT laboratories. For the prefabricated specimens, the results from UCPRC were again similar to those from NCAT, but only marginally similar to those from TTI. When comparing results between TTI and NCAT, the results were statistically different. Comparisons of results for the different fabrication methods for each laboratory indicate that fabrication did not influence the TTI results, but did influence

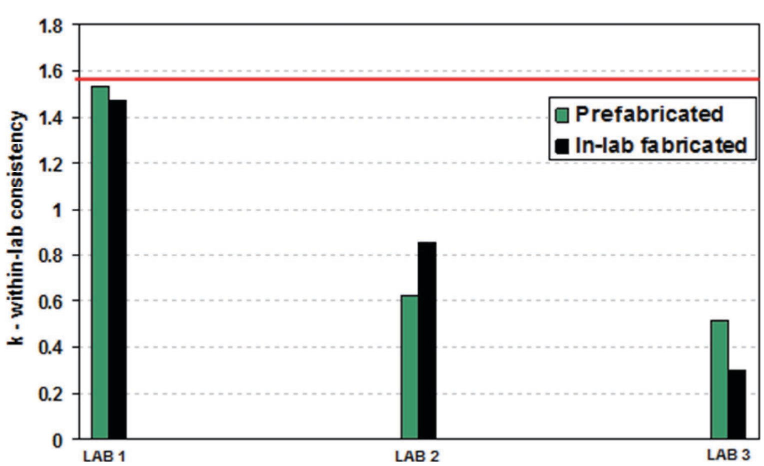


Figure D-1. Flow number: within-laboratory (k) consistency statistics.

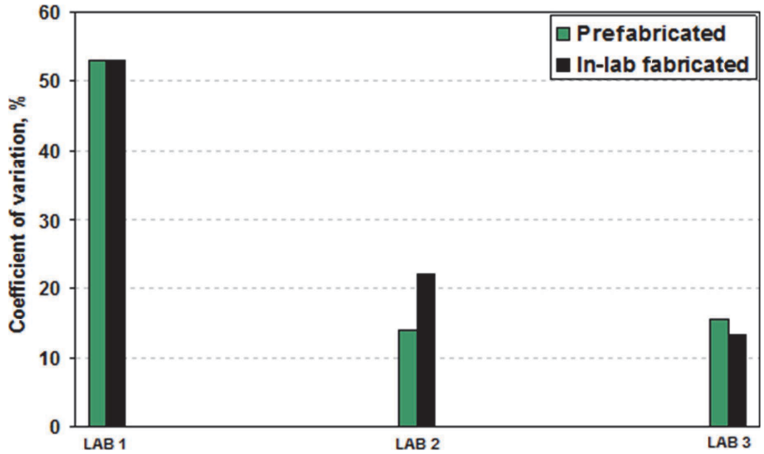


Figure D-2. Flow number: variability of test results.

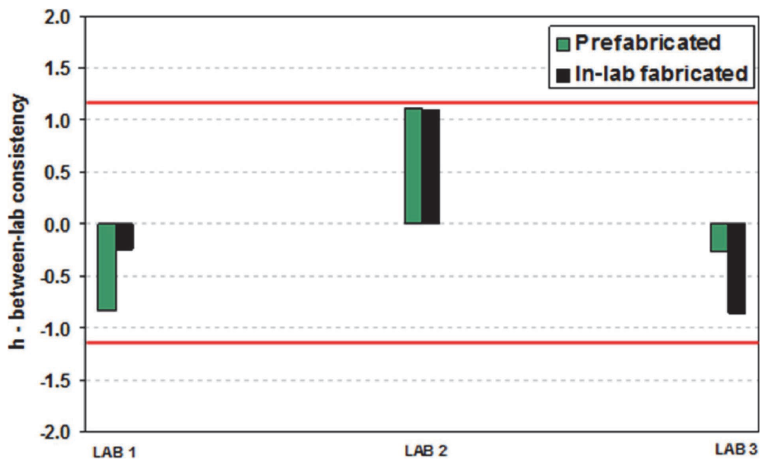


Figure D-3. Flow number: between-laboratory (h) consistency statistics.

Table D-4. Flow number: *p*-values from the Welch modified two-sample *t*-test.

Comparison	*p*-Value		
	In-Laboratory Fabricated	Prefabricated by UCPRC	Prefabricated vs. In-Lab Fabricated
UCPRC – TTI	0.19	0.09	–
UCPRC – NCAT	0.49	0.56	–
TTI – NCAT	0.01	0.02	–
TTI – TTI	–	–	0.45
NCAT – NCAT	–	–	0.01

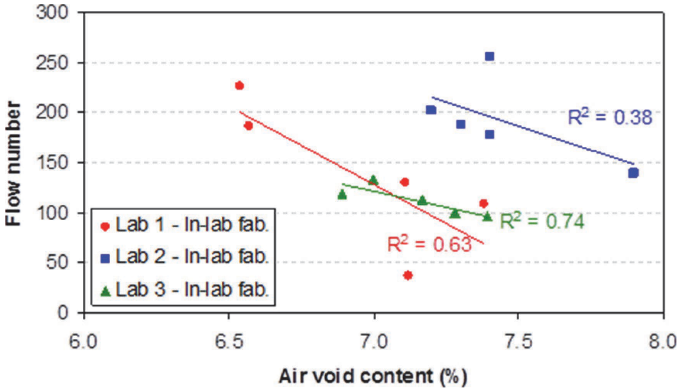


Figure D-4. Flow number: relationship with AV content for in-lab fabricated specimens.

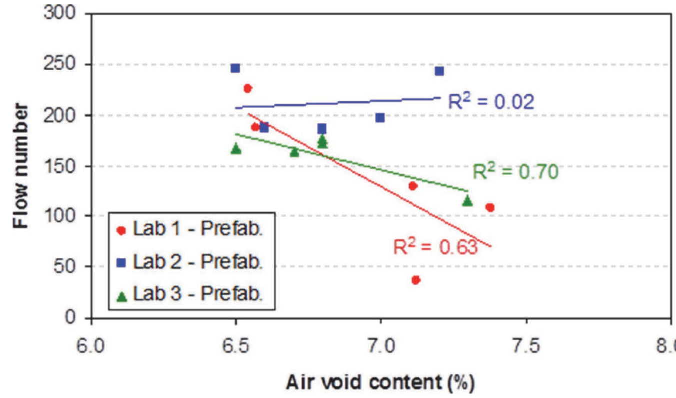


Figure D-5. Flow number: relationship with AV content for prefabricated specimens.

the NCAT results. This analysis was not conducted for the UCPRC laboratory since all specimens were prepared at the same time.

AV content variability can contribute to test result variability and bias between different laboratories. Specimen AV contents were plotted against flow numbers (Figures D-4 and D-5) to investigate this effect. AV content variability had an effect on measured flow number variability for the UCPRC and NCAT laboratories. The results from TTI were less influenced by specimen AV content.

Resilient Modulus

Within-laboratory (k) consistency statistics for the prefabricated and in-laboratory fabricated specimens are shown in Figure D-6. None of the data exceeded the limit for the k statistic. However, due to the high variability in the UCPRC and NCAT test results, within-laboratory consistency appeared to be lower overall. Within-laboratory consistency statistics for TTI were lower than those for UCPRC and NCAT. The coefficients of variation of the measured resilient modulus values were also calculated and are shown in Figure D-7. Test result variability was significantly smaller than the variability for the flow number test results discussed in the previous section. Although resilient modulus values from all three laboratories had similar variability, the TTI results appeared to have the lowest level of within-laboratory variability.

Between-laboratory (h) consistency statistics for the prefabricated and in-laboratory fabricated specimens are shown in Figure D-8. The prefabricated specimen results from TTI exceeded the limits for the h statistic while its results for the in-laboratory fabricated specimens were close to the limit. These results suggest that the TTI resilient modulus test results from prefabricated specimens were different from those from UCPRC and NCAT.

Results for the differences in between-laboratory test results are listed in Table D-5. For the in-laboratory fabricated specimens, the UCPRC results were statistically different from those

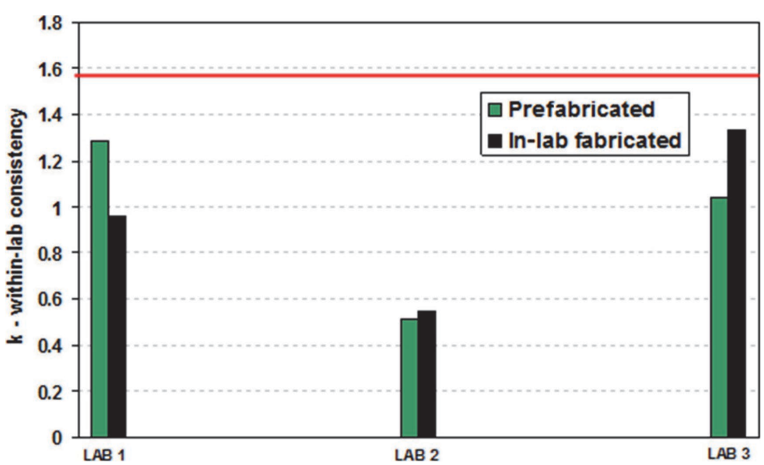


Figure D-6. Resilient modulus: within-laboratory (k) consistency statistics.

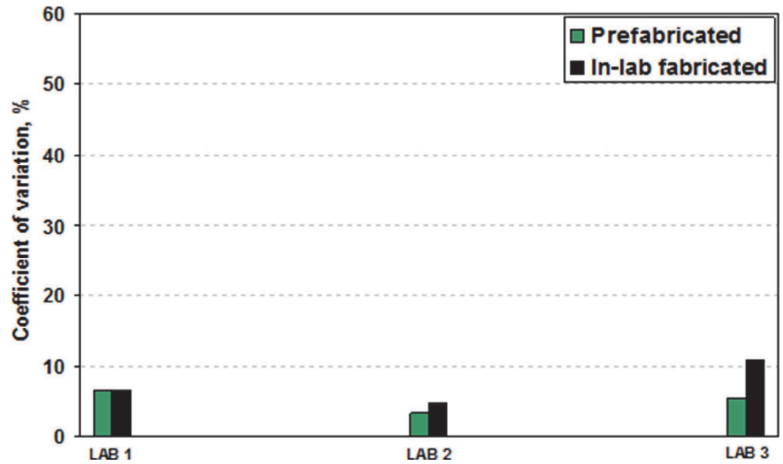


Figure D-7. Resilient modulus: variability of test results.

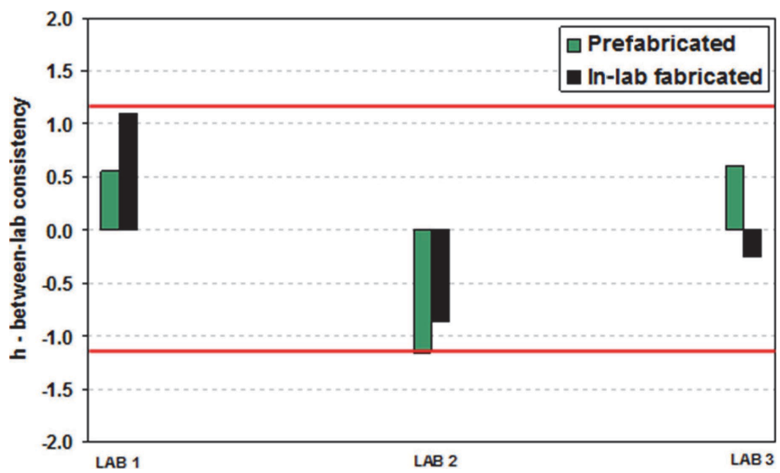


Figure D-8. Resilient modulus: between-laboratory (h) consistency statistics.

Table D-5. Resilient modulus: *p*-values from the Welch modified two-sample *t*-test.

Comparison	*p*-Value		
	In-Laboratory Fabricated	Prefabricated by UCPRC	Prefabricated vs. In-Lab Fabricated
UCPRC – TTI	0.00	0.00	–
UCPRC – NCAT	0.02	0.90	–
TTI – NCAT	0.18	0.00	–
TTI – TTI	–	–	0.72
NCAT – NCAT	–	–	0.01

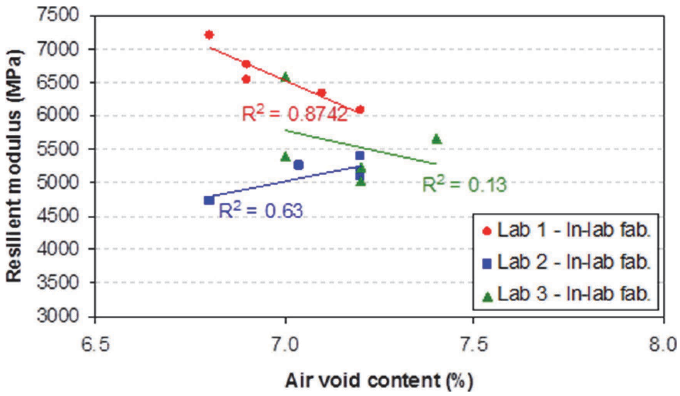


Figure D-9. Resilient modulus: relationship with AV content for in-lab fabricated specimens.

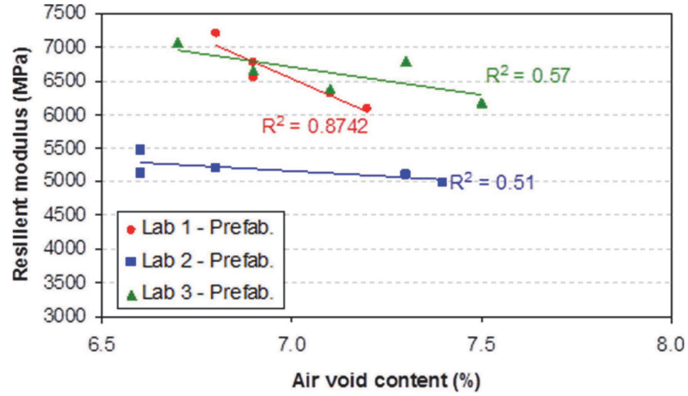


Figure D-10. Resilient modulus: relationship with AV content for prefabricated specimens.

from TTI and NCAT. The results from TTI show some similarity to those from NCAT. For the prefabricated specimens, the results from UCPRC and NCAT were statistically similar, but both these were statistically different from those from TTI. Comparisons of results for the different fabrication methods for each laboratory indicate that fabrication did not influence the TTI results but did influence the NCAT results. This analysis was not conducted for the UCPRC laboratory since all specimens were prepared at the same time.

Plots of AV content versus resilient modulus for in-laboratory fabricated and prefabricated specimens are shown in Figures D-9 and D-10, respectively. AV content variability had an effect on the measured resilient modulus variability for the prefabricated specimens, but was not evident in the test results for in-laboratory fabricated specimens shown in Figure D-9.

Dynamic Modulus

Within-laboratory (k) consistency statistics for the prefabricated and in-laboratory fabricated specimens are shown in Figure D-11. The test results on in-laboratory fabricated specimens from TTI exceeded the limit for the k statistic; all other results were below the limit, with the NCAT results for in-laboratory fabricated specimens appearing to have the lowest level of variability and highest level of within-laboratory consistency. Temperature and loading frequency did not affect the within-laboratory consistency. The coefficients of variation of the measured E* values for all temperatures and loading frequencies are shown in Figure D-12. Test result variability for E* was significantly higher than the variability for the resilient modulus test results (see Figure D-7).

Between-laboratory (h) consistency statistics for the in-laboratory fabricated and prefabricated specimens are shown in Figure D-13. Results from TTI for the in-laboratory fabricated specimens exceed the limit for the h statistic. The results from UCPRC for the prefabricated specimens are also close to the limit.

Results for the differences in between-laboratory test results are listed in Table D-6. For the in-laboratory fabricated specimens, the UCPRC results were similar to the NCAT results for six of the ten testing configurations and only similar to

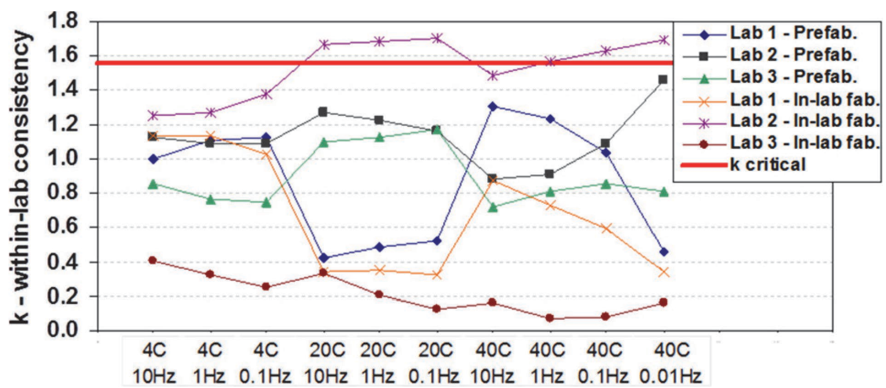


Figure D-11. Dynamic modulus: within-laboratory (k) consistency statistics.

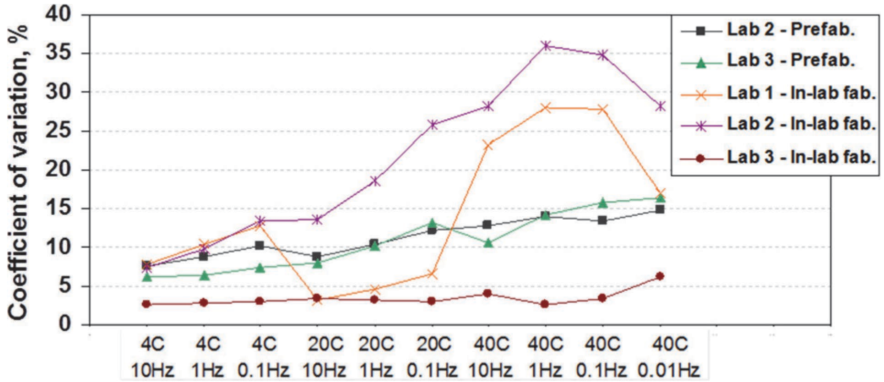


Figure D-12. Dynamic modulus: variability of test results.

one of the TTI results (20°C/1 Hz). The TTI and NCAT results were also only similar for one testing configuration (40°C/10 Hz). For the prefabricated specimens, the UCPRC results were similar to the NCAT results for four of the ten testing configurations and again only similar to one of the TTI results (40°C/10 Hz). The TTI results were similar to the NCAT results for eight of the ten testing configurations. Comparisons of results for the different fabrication methods for each laboratory indicate that fabrication did not influence the TTI results but did influence the NCAT results (three out of ten testing configurations were similar). This analysis was not conducted for the UCPRC laboratory since all specimens were prepared at the same time.

Figures D-14 and D-15, respectively, compare the repeatability and reproducibility statistics for the E* tests. Repeatability and reproducibility were both higher for the prefabricated specimens. Consequently, specimen fabrication could have affected the between-laboratory test result variability.

The effect of AV content on E* test results for the 20°C and 1 Hz loading frequency test configuration on the in-laboratory fabricated and prefabricated specimens are plotted in Figures D-16 and D-17, respectively. AV content variability had an effect on the measured E* variability for the test results from TTI and NCAT but did not appear to influence the UCPRC results.

Phase Angle

Within-laboratory (k) consistency statistics for the in-laboratory fabricated and prefabricated specimens are shown in Figure D-18. The TTI results again exceeded the limit for the k statistic. The NCAT test results for in-laboratory fabricated specimens appeared to have the lowest level of variability and a high level of within-laboratory consistency. Temperature and loading frequency did not affect any of the within-laboratory test result consistency.

Between-laboratory (h) consistency statistics for the in-laboratory fabricated and prefabricated specimens are shown in Figure D-19. The UCPRC phase angle values from prefabricated specimens were on the limit for the h statistic. TTI values for in-laboratory fabricated specimens were also close to the limit.

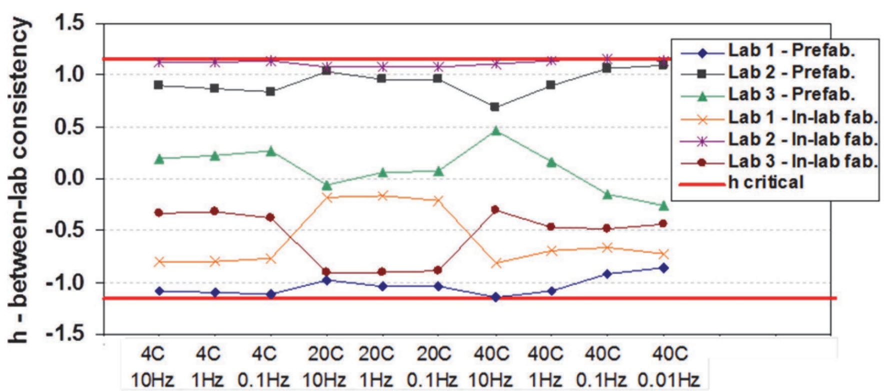


Figure D-13. Dynamic modulus: between-laboratory (h) consistency statistics.

Table D-6. Dynamic modulus: *p*-values from the Welch modified two-sample *t*-test.

Test Parameter	Comparison	*p*-Value		
		In-Laboratory Fabricated	Prefabricated by UCPRC	Prefabricated vs. In-Lab Fabricated
4°C, 10 Hz	UCPRC – TTI	0.02	0.03	–
	UCPRC – NCAT	0.39	0.09	–
	TTI – NCAT	0.04	0.35	–
	TTI – TTI	–	–	0.92
	NCAT – NCAT	–	–	0.15
4°C, 1 Hz	UCPRC – TTI	0.03	0.03	–
	UCPRC – NCAT	0.38	0.08	–
	TTI – NCAT	0.05	0.35	–
	TTI – TTI	–	–	0.79
	NCAT – NCAT	–	–	0.10
4°C, 0.1 Hz	UCPRC – TTI	0.02	0.02	–
	UCPRC – NCAT	0.40	0.06	–
	TTI – NCAT	0.05	0.36	–
	TTI – TTI	–	–	0.64
	NCAT – NCAT	–	–	0.05
20°C, 10 Hz	UCPRC – TTI	0.10	0.08	–
	UCPRC – NCAT	0.00	0.31	–
	TTI – NCAT	0.03	0.37	–
	TTI – TTI	–	–	0.57
	NCAT – NCAT	–	–	0.02
20°C, 1 Hz	UCPRC – TTI	0.11	0.09	–
	UCPRC – NCAT	0.00	0.27	–
	TTI – NCAT	0.03	0.47	–
	TTI – TTI	–	–	0.46
	NCAT – NCAT	–	–	0.02
20°C, 0.1 Hz	UCPRC – TTI	0.10	0.04	–
	UCPRC – NCAT	0.00	0.19	–
	TTI – NCAT	0.03	0.39	–
	TTI – TTI	–	–	0.48
	NCAT – NCAT	–	–	0.01
40°C, 10 Hz	UCPRC – TTI	0.09	0.10	–
	UCPRC – NCAT	0.36	0.12	–
	TTI – NCAT	0.17	0.75	–
	TTI – TTI	–	–	0.43
	NCAT – NCAT	–	–	0.15
40°C, 1 Hz	UCPRC – TTI	0.07	0.02	–
	UCPRC – NCAT	0.55	0.09	–
	TTI – NCAT	0.09	0.19	–
	TTI – TTI	–	–	0.52
	NCAT – NCAT	–	–	0.06
40°C, 0.1 Hz	UCPRC – TTI	0.02	0.00	–
	UCPRC – NCAT	0.41	0.02	–
	TTI – NCAT	0.03	0.00	–
	TTI – TTI	–	–	0.98
	NCAT – NCAT	–	–	0.03
40°C, 0.01 Hz	UCPRC – TTI	0.01	0.00	–
	UCPRC – NCAT	0.01	0.00	–
	TTI – NCAT	0.01	0.00	–
	TTI – TTI	–	–	0.17
	NCAT – NCAT	–	–	0.02

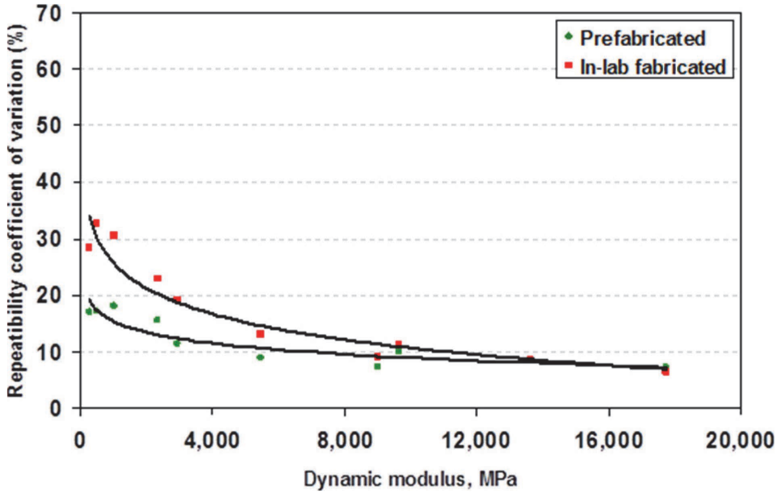


Figure D-14. Dynamic modulus: comparison of repeatability.

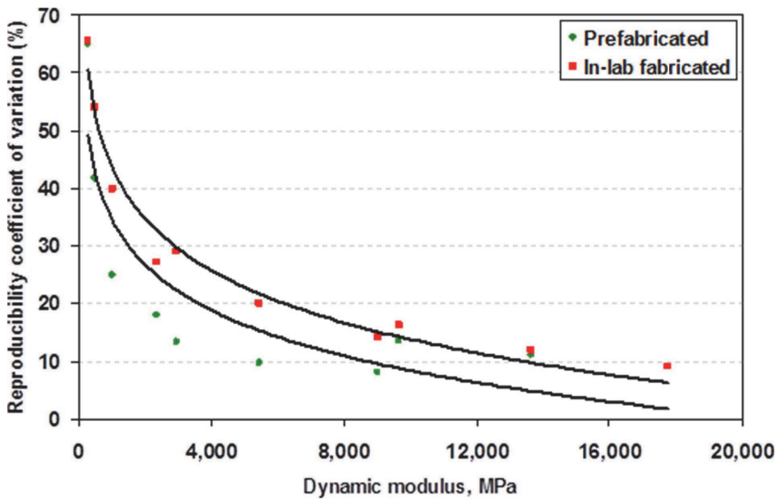


Figure D-15. Dynamic modulus: comparison of reproducibility.

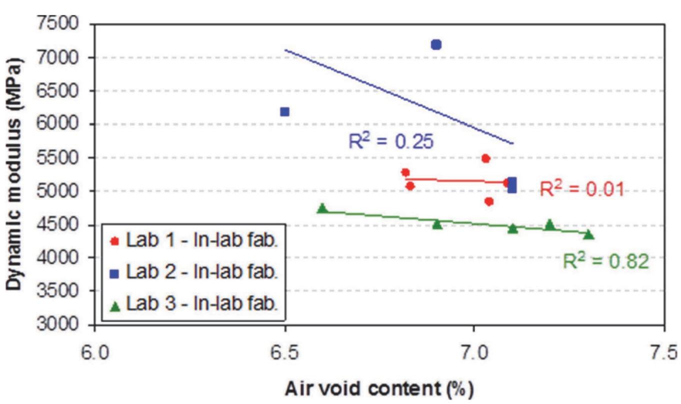


Figure D-16. Dynamic modulus: relationship with AV content for in-lab fabricated specimens.

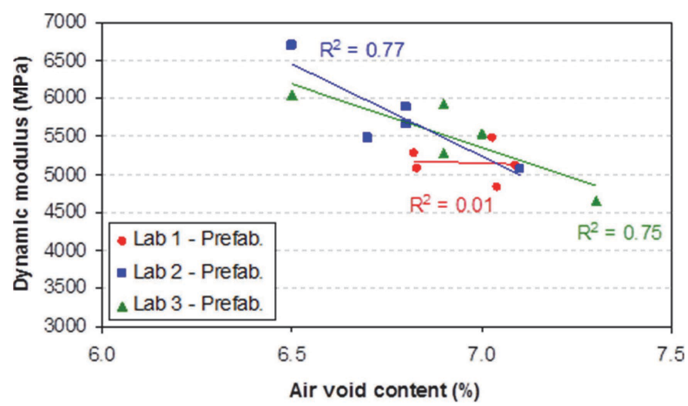


Figure D-17. Dynamic modulus: relationship with AV content for prefabricated specimens.

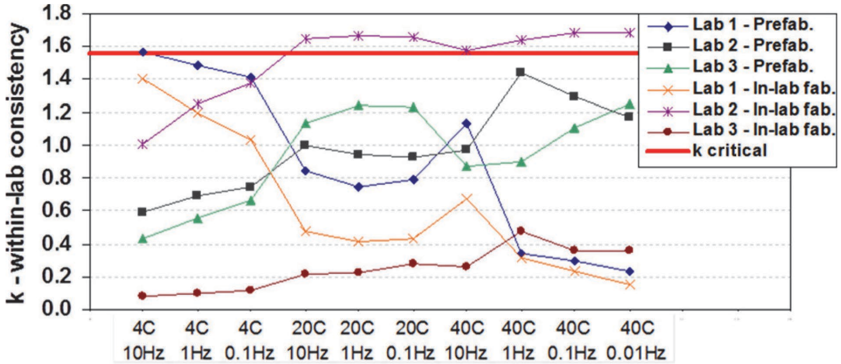


Figure D-18. Phase angle: within-laboratory (k) consistency statistics.

Results for the differences in between-laboratory test results are listed in Table D-7. For the in-laboratory fabricated specimens, the UCPRC results were similar to the TTI results for just two of the ten testing configurations (20°C/10 Hz and 20°C/1 Hz) and similar to the NCAT results for another two of the ten configurations (4°C/1 Hz and 4°C/0.1 Hz). The TTI and NCAT results were similar for five of the ten testing configurations. For the prefabricated specimens, the UCPRC results were statistically different from the TTI results for all testing configurations. The UCPRC results were similar to the NCAT results for three of the ten testing configurations, while the TTI results were similar to the NCAT results for six of the ten testing configurations. Comparisons of results for the different fabrication methods for each laboratory indicate that fabrication generally did not influence the TTI results (seven out of ten testing configurations were similar) but did influence the NCAT results (only two out of ten testing configurations were similar). This analysis was not conducted for the UCPRC laboratory since all specimens were prepared at the same time. The results for phase angle were generally inconsistent with the dynamic modulus test results, despite the tests being carried out on the same specimens and as part of the same testing sequence.

The repeatability and reproducibility statistics for phase angle are compared in Figures D-20 and D-21, respectively. Repeatability and reproducibility were both higher for the prefabricated specimens. Consequently, specimen fabrication could have affected the between-laboratory test result variability. Reproducibility for both the in-laboratory fabricated and prefabricated specimens appeared to be similar.

The effect of AV content on phase angle test results for the 20°C and 1 Hz loading frequency test configuration on the in-laboratory fabricated and prefabricated specimens are plotted in Figures D-22 and D-23, respectively. AV content variability did not have a significant and consistent effect on the measured phase angle variability for the test results from all three laboratories.

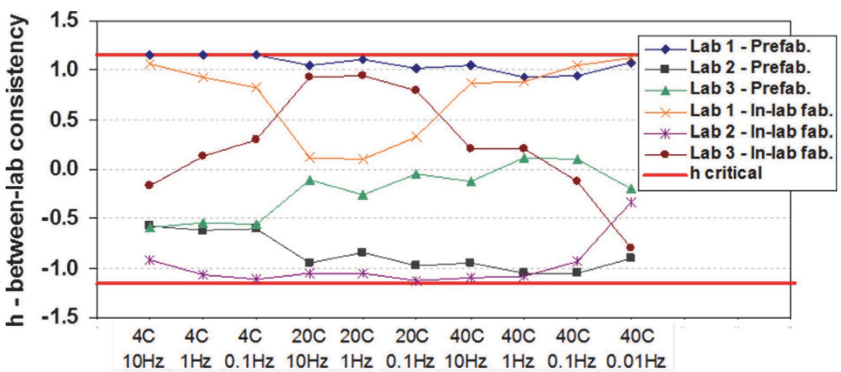


Figure D-19. Phase angle: between-laboratory (h) consistency statistics.

Table D-7. Phase angle: *p*-values from the Welch modified two-sample *t*-test.

Test Parameter	Comparison	*p*-Value: In-Laboratory Fabricated	*p*-Value: Prefabricated by UCPRC	*p*-Value: Prefabricated vs. In-Lab Fabricated
4°C, 10 Hz	UCPRC – TTI	0.03	0.05	–
	UCPRC – NCAT	0.09	0.04	–
	TTI – NCAT	0.21	0.92	–
	TTI – TTI	–	–	0.53
	NCAT – NCAT	–	–	0.07
4°C, 1 Hz	UCPRC – TTI	0.03	0.05	–
	UCPRC – NCAT	0.20	0.05	–
	TTI – NCAT	0.10	0.97	–
	TTI – TTI	–	–	0.36
	NCAT – NCAT	–	–	0.03
4°C, 0.1 Hz	UCPRC – TTI	0.04	0.03	–
	UCPRC – NCAT	0.28	0.03	–
	TTI – NCAT	0.10	0.99	–
	TTI – TTI	–	–	0.45
	NCAT – NCAT	–	–	0.01
20°C, 10 Hz	UCPRC – TTI	0.17	0.06	–
	UCPRC – NCAT	0.01	0.21	–
	TTI – NCAT	0.04	0.37	–
	TTI – TTI	–	–	0.65
	NCAT – NCAT	–	–	0.00
20°C, 1 Hz	UCPRC – TTI	0.13	0.07	–
	UCPRC – NCAT	0.00	0.26	–
	TTI – NCAT	0.03	0.56	–
	TTI – TTI	–	–	0.42
	NCAT – NCAT	–	–	0.01
20°C, 0.1 Hz	UCPRC – TTI	0.07	0.01	–
	UCPRC – NCAT	0.04	0.13	–
	TTI – NCAT	0.03	0.20	–
	TTI – TTI	–	–	0.47
	NCAT – NCAT	–	–	0.01
40°C, 10 Hz	UCPRC – TTI	0.01	0.00	–
	UCPRC – NCAT	0.03	0.00	–
	TTI – NCAT	0.06	0.01	–
	TTI – TTI	–	–	0.67
	NCAT – NCAT	–	–	0.02
40°C, 1 Hz	UCPRC – TTI	0.00	0.00	–
	UCPRC – NCAT	0.00	0.00	–
	TTI – NCAT	0.01	0.00	–
	TTI – TTI	–	–	0.03
	NCAT – NCAT	–	–	0.02
40°C, 0.1 Hz	UCPRC – TTI	0.01	0.00	–
	UCPRC – NCAT	0.00	0.00	–
	TTI – NCAT	0.11	0.00	–
	TTI – TTI	–	–	0.02
	NCAT – NCAT	–	–	0.25
40°C, 0.01 Hz	UCPRC – TTI	0.02	0.00	–
	UCPRC – NCAT	0.00	0.00	–
	TTI – NCAT	0.29	0.01	–
	TTI – TTI	–	–	0.02
	NCAT – NCAT	–	–	0.71

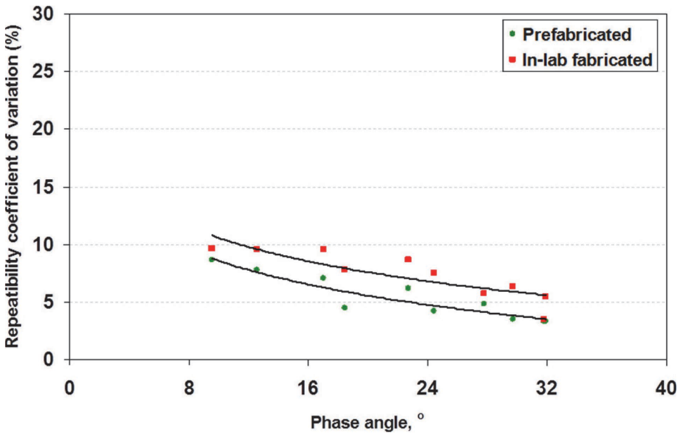


Figure D-20. Phase angle: comparison of repeatability.

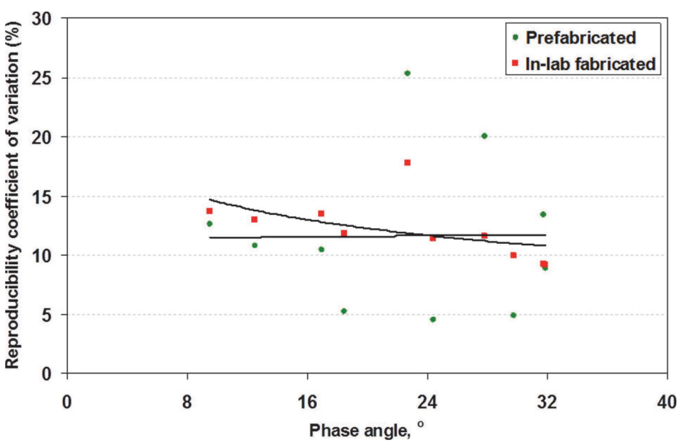


Figure D-21. Phase angle: comparison of reproducibility.

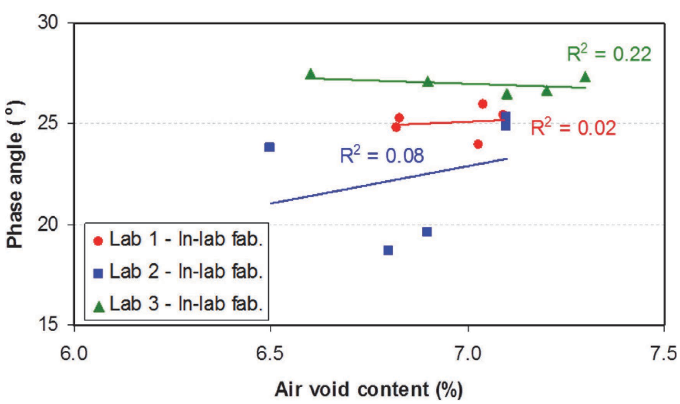


Figure D-22. Phase angle: relationship with AV content for in-lab fabricated specimens.

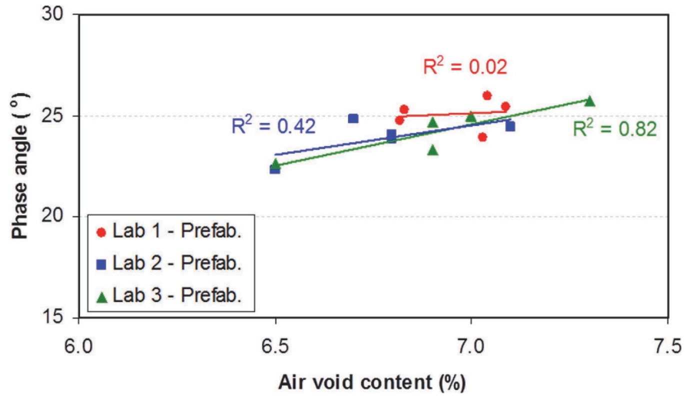


Figure D-23. Phase angle: relationship with AV content for prefabricated specimens.

Sources of Variability and Bias

The reasons behind the differences in TTI, NCAT, and UCPRC results for the flow number, resilient modulus, and, to a lesser degree, E* and the phase angle were analyzed. Possible sources of variability and bias introduced during specimen preparation and testing included test and specimen preparation procedures, test setup, and equipment differences.

Specimen Preparation

All three laboratories outlined their step-by-step specimen preparation procedures. No significant differences were identified in these. Details of the specimen preparation for all three laboratories are listed in Table D-8.

Unconfined Flow Number

None of the laboratories exceeded the limit for the within- and between-laboratory consistency statistics. TTI results were slightly higher than the other two laboratories (not statistically significant). Possible causes of bias and their resolution are:

- Testing with or without steel ball (top platen free to rotate)—none of the laboratories used the steel ball during the tests.
- Friction reducers—all three laboratories used latex friction reducers.

Resilient Modulus

Load Level

NCAT and UCPRC ran their resilient modulus testing with loading set at 150 lb, while TTI ran tests at 75 lb. This was discussed earlier in the study; however, the different load levels were agreed upon given that the NCAT and UCPRC equipment achieved reasonable signal-to-noise ratios only at 150 lb

Table D-8. Comparison of specimen preparation variables.

Procedure	NCAT	UCPRC	TTI
Binder break down	145°C for 2 hours	145°C for 2 hours	145°C for 2 hours
Compaction temp. and time	135°C for 2 hours	140°C for 2 hours	140°C for 2 hours
Aggregate mixing temp. and time	160°C for 2 hours	160°C for 2 hours	160°C for 2 hours
Binder mixing temp. and time	145°C for 2 hours	145°C for 2 hours	145°C for 2 hours
Short-term aging	135°C for 4 hours	135°C for 4 hours	135°C for 4 hours

load level while noise was high at 75 lb. On the other hand, TTI's test equipment is capable of testing at only 75 lb. This difference in load levels was expected to be a source of bias in the test results. However, NCAT tested some samples at both 75-lb and 150-lb to determine the effect of load level on test results and did not observe any significant effect on average resilient modulus. At 75 lb load level, UCPRC could not get a reasonable wave form for the determination of the resilient modulus.

LVDT Configuration

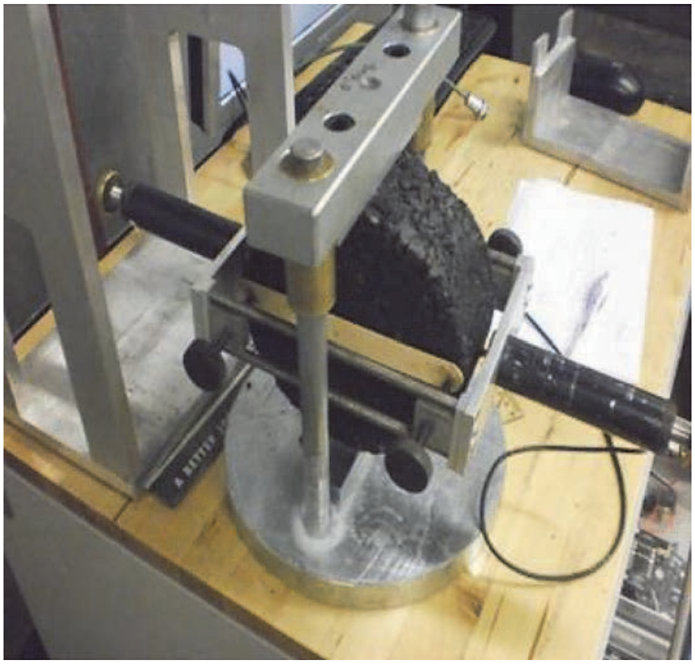
NCAT and TTI used LVDT around the sample while UCPRC had LVDTs mounted on the sample (Figure D-24). Although this difference in LVDT position is expected to affect the test results due to the non-uniformity in AV distributions within the specimens, results were not conclusive since the average resilient modulus results measured by NCAT and UCPRC were close.

Different load levels and LVDT position might be the sources of bias between the three laboratories. However, since the mixtures from the identified projects/asphalt plants will be tested at one laboratory only and the focus of the project was to identify the differences in properties for the mixture variables, possible sources of bias were not expected to affect the statistical analysis of the test results in the project as a whole.

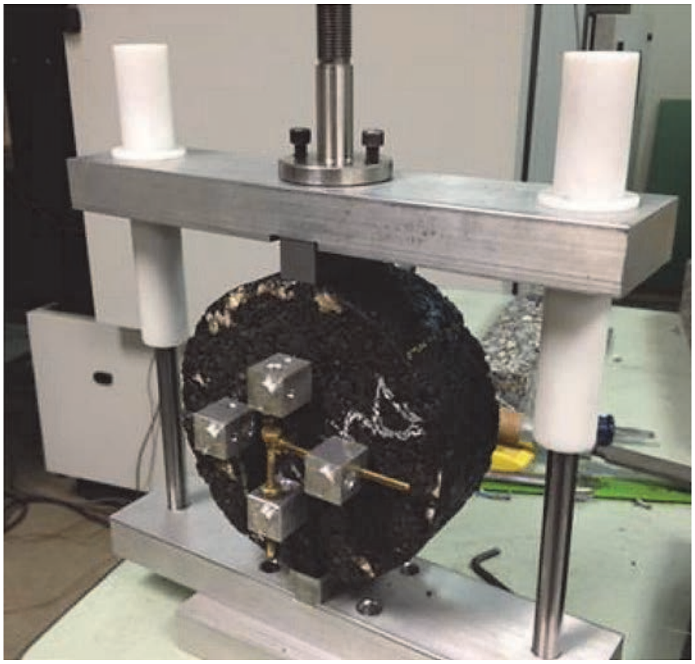
Dynamic Modulus

TTI test results for in-laboratory fabricated specimens slightly exceeded the limits for both the within-laboratory and between-laboratory consistency statistics. However, the difference was not very high. Possible causes of bias and their resolution are:

- Equipment differences—all laboratories have IPC Global equipment.
- Data quality—all laboratories checked data quality statistics requirements given in AASHTO TP 79 and each met the requirements.
- Testing with or without steel ball (top platen free to rotate)—all laboratories used the steel ball.



(a) TTI and NCAT



(b) UCPRC (according to ASTM D7369)

Figure D-24. LVDT configurations.

- Friction reducers—NCAT used Teflon friction reducers for E* testing (which is allowed according to the specification) while TTI and UCPRC used latex friction reducers.
- Temperature conditioning time—all laboratories followed the procedures given in the specifications and no important differences were noted.

Summary

The conclusion from the analysis of sources of variability and bias was that all of the laboratories were performing the tests in a similar manner, except for a few minor details. The minor differences did not have major influence on the results, and were corrected. Statistical approaches for analyzing the results of field testing to meet the objectives of the project were also identified.

Summary and Recommendations

The following lists summarize the results from the different tests.

Unconfined Flow Number

- None of the data exceeded the limit for the within-laboratory consistency statistic.
- The UCPRC test result variability was significantly higher than those of TTI and NCAT.
- None of the data exceeded the limit for the between-laboratory consistency statistic.
- The TTI flow number test results were different from those from UCPRC and NCAT.
- Specimen fabrication did not appear to significantly affect the flow number test results from TTI. The in-laboratory fabricated and prefabricated specimen test results from NCAT were different.
- AV content variability influenced the UCPRC and NCAT results but had limited effect on the TTI test results.

Resilient Modulus

- None of the data exceeded the limit for the within-laboratory consistency statistic.
- Due to the high variability in the UCPRC and NCAT test results, within-laboratory consistency appeared to be lower.
- Within-laboratory test result variability was significantly smaller than the variability for the flow number test results.
- TTI test results on prefabricated specimens exceeded the limit for the between-laboratory consistency statistic. The UCPRC and NCAT test results for the prefabricated specimens were similar to each other, but both were different from those from TTI.
- For the in-laboratory fabricated specimens, TTI and NCAT results were similar, but both were different from those from the UCPRC.
- Specimen fabrication did not appear to significantly affect the test results from TTI. The in-laboratory fabricated and prefabricated specimen test results from NCAT were different.
- AV content variability had some effect on measured resilient modulus variability for the prefabricated specimens. This effect was not evident in test results from the in-laboratory fabricated specimens.

Dynamic Modulus

- TTI test results for in-laboratory fabricated specimens exceeded the limits for both the within-laboratory and between-laboratory consistency statistics.
- NCAT test results for in-laboratory fabricated specimens appeared to have the lowest level of variability.
- Temperature and loading frequency did not affect the within-laboratory consistency.
- Variability in the E* test results between and within the laboratories was significantly higher than the variability for the resilient modulus test results.
- UCPRC and NCAT test results for the in-laboratory fabricated specimens were generally similar, but both were statistically different from those from TTI.
- Test results for the prefabricated specimens from TTI and NCAT were generally similar. UCPRC results were significantly different from those from TTI for all loading frequencies and temperatures and different from those from NCAT for six of the ten testing configurations.
- Specimen fabrication did not appear to significantly affect the TTI E* test results. The NCAT results for the in-laboratory fabricated and prefabricated specimen test results were different.
- Repeatability and reproducibility were both better for the prefabricated specimens.
- AV content variability appeared to have some effect on measured E* variability at TTI and NCAT but did not appear to influence the test results from UCPRC.

Phase Angle

- The TTI test results for in-laboratory fabricated specimens exceeded the limit for the within-laboratory consistency statistic.
- NCAT test results for in-laboratory fabricated specimens appeared to have the lowest level of variability and highest level of within-laboratory consistency.
- Temperature and loading frequency did not affect the within-laboratory consistency.

- The UCPRC phase angle values for prefabricated specimens were on the limit for the between-laboratory consistency statistic.
- Test results from all three laboratories appeared to be different.
- Specimen fabrication did not appear to significantly affect the phase angle results from TTI. The in-laboratory fabricated and prefabricated specimen phase angle results from NCAT were different.
- Repeatability was better for the prefabricated specimens. Reproducibility for sets of specimens appeared to be similar for all laboratories.
- AV content variability did not appear to have a significant and consistent effect on measured phase angle variability at all three laboratories.

Based on the outcome of the variability and bias analysis, the recommendation was to proceed with field testing, and use standard statistical procedures to handle the levels of between-laboratory bias and variability identified in this study and to arrive at sound conclusions for the overall project.

References

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APPENDIX E

Statistical Analysis

Phase I Experiment

The objective of the statistical analysis for Phase I was to identify factors with significant effects on resilient modulus (M_R) stiffness of short-term aged asphalt mixtures. The factors of interest were:

1. WMA technology (HMA vs. WMA)
2. Production temperature (high temperature vs. control temperature)
3. Plant type (DMP vs. BMP)
4. Recycled materials (no RAP/RAS vs. RAP/RAS)
5. Aggregate absorption (low absorption vs. high absorption)
6. Binder source (Binder 1 vs. Binder 2)

In addition to the aforementioned factors, the information on field site (Connecticut, Florida, Indiana, Iowa, New Mexico, South Dakota, Texas I, Texas II, Wyoming), specimen type (plant-mixed, plant-compacted [PMPC]; laboratory-mixed, laboratory compacted [LMLC]; cores at construction [Cores@Const]), nominal maximum aggregate size (NMAS; 9.5 mm, 12.5 mm, 19 mm), and air-void (AV) content (3.6 to 11.2 percent) was also available and incorporated into the analyses. Six separate experiments and analyses were performed to assess the effects of each of the six factors of interest listed above.

WMA Technology (HMA vs. WMA)

There were 343 non-missing (correct and recorded) M_R stiffness measurements obtained from eight different field sites (Connecticut, Florida, Indiana, Iowa, New Mexico, South Dakota, Texas I, Wyoming), three specimen types, and two levels of WMA technology in the data for this analysis. Other relevant variables for the analysis were NMAS and AV. The analysis of covariance (ANCOVA) having WMA technology and specimen type as main effects along with a two-way interaction effect between them (specimen type*WMA technology), NMAS and AV as covariates, and field site as a random effect was fitted to the data.

Table E-1 contains the analysis outputs obtained by the restricted maximum likelihood (REML) method implemented in the JMP statistical package (SAS product). It can be observed from Table E-1 (see Fixed Effects Tests) that the effects of specimen type, WMA technology, specimen type*WMA technology, and AV were statistically significant at $\alpha = 0.05$, while the effect of NMAS was not.

When there is a significant interaction effect, the effect of each factor involved in the interaction needs to be assessed conditional on the levels of the other factor because the effect might be different for each level of the other factor. Therefore, the effect of WMA technology was assessed for each level of specimen type. Figure E-1 contains the interaction plot for WMA technology and specimen type. An interaction plot is a graph of predicted mean responses (least squares means, which will be explained later) that are connected line segments. The *y*-axis is the predicted mean response, and the *x*-axis displays the levels of one of the factors (here WMA technology). Separate line segments are drawn for each level of the other factor (specimen type). It can be observed from the plot that except for Cores@Const, the predicted M_R stiffness was lower for WMA than for HMA. For Cores@Const, the predicted M_R did not seem to be different.

Table E-2 contains the analysis output obtained by the REML method implemented in JMP. Table E-2 presents the predicted values (least squares [LS] means) for M_R stiffness for each level of significant factors along with their standard errors. When there are multiple factors in the model, it is not fair to make comparisons between raw cell means in data because raw cell means do not compensate for other factors in the model. The LS means are the predicted values of the

Table E-1. Analysis results of the effect of WMA technology based on the ANCOVA model with main effects and a two-way interaction effect.

Response M_R Stiffness (ksi)

Summary of Fit

RSquare	0.770825
RSquare Adj	0.765336
Root Mean Square Error	111.2762
Mean of Response	435.519
Observations (or Sum Wgts)	343

Fixed Effect Tests

Source	Nparm	DF	DFDen	F Ratio	Prob > F
Specimen Type	2	2	329	12.7708	<0.0001*
WMA Technology	1	1	329.2	15.4774	0.0001*
NMAS (mm)	2	2	4.98	0.4257	0.6750
AV (%)	1	1	329.4	22.1249	<0.0001*
Specimen Type*WMA Technology	2	2	329	4.9565	0.0076*

response (M_R stiffness) for each level of a factor that have been adjusted for the other factors in the model. Note that the "Least Sq Mean" denotes the least squares means for M_R stiffness. The predicted values (LS means) and the corresponding standard errors for M_R stiffness for each factor-level combination of WMA technology and specimen type are presented in Table E-2. For specimen type*WMA technology, a multiple comparison procedure (Tukey's honest significant difference [HSD]) was also carried out to test which of those factor levels were statistically different. It can be seen that the difference between HMA and WMA was statistically significant for PMPC while not for LMLC and Cores@Const.

The ANCOVA model without the two-way interaction effect term (specimen type*WMA technology), but with WMA technology and specimen type as main effects, NMAS and AV as covariates, and field site as a random effect, was also fitted to the same data for comparison purposes. Note that this model is meaningful if the two-way interaction effect presented in Table E-2 can be considered to be practically insignificant. Table E-3 contains the analysis outputs. Again, it can be observed from Table E-3 that the effects of specimen type, WMA technology, and AV were statistically significant at $\alpha = 0.05$, while the effect of NMAS was not.

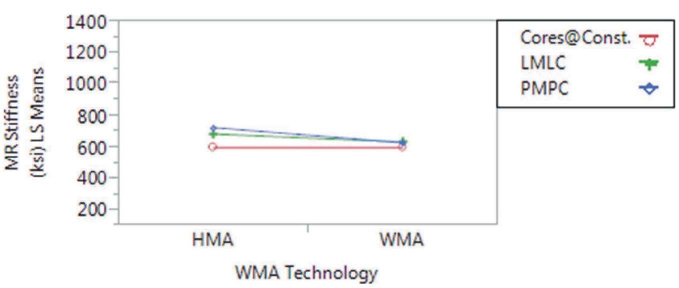


*Figure E-1. Least squares means or interaction plot for specimen type*WMA technology.*

Production Temperature (High vs. Control)

There were 117 non-missing (correct and recorded) M_R stiffness measurements obtained from two different field sites (Iowa, Wyoming), three specimen types, two levels of WMA technology, and two levels of production temperature in the data for this analysis. Another relevant variable that could be included in the analysis was AV content. The value of NMAS was held constant at 12.5 mm for all of the M_R stiffness measurements in the data, so it was excluded from the analysis. The ANCOVA model including production temperature, WMA technology, and specimen type as main effects along with all possible two-way interactions, AV as a covariate, and field site as a random effect was first fitted to the data, but none of the two-way interaction effects were statistically significant. Thus, two-way interaction effects were removed from the model, and the ANCOVA model with production temperature, WMA technology and specimen type as main effects, AV as a covariate, and field site as a random effect was fitted again to the data. Table E-4 contains the analysis output obtained by the REML method implemented in JMP. It can be observed from Table E-4 (see Fixed Effects Tests) that the effects of specimen type and WMA technology were statistically significant at $\alpha = 0.05$, while the effect of production temperature and AV were not.

Plant Type (BMP vs. DMP)

There were 83 non-missing (correct and recorded) M_R stiffness measurements obtained from two different field sites (Indiana, Texas II), three specimen types, and two levels of plant type (DMP vs. BMP) in the data for this analysis. Another relevant variable that was included in the analysis was AV. The value of NMAS was held constant at 9.5 mm for all of the M_R stiffness measurements in the data, so it was excluded from the analysis. The ANCOVA model including plant type and

Table E-2. Effect details for model in Table E-1.

Effect Details
Specimen Type
Least Squares Means Table

Level	Least Sq Mean	Std Error
Cores@Const.	598.85750	222.91111
LMLC	659.23066	222.81605
PMPC	677.15810	222.80865

WMA Technology
Least Squares Means Table

Level	Least Sq Mean	Std Error
HMA	669.82037	222.74650
WMA	620.34380	222.75855

NMAS (mm)
Least Squares Means Table

Level	Least Sq Mean	Std Error
9.5	645.08209	222.66375
12.5	419.98336	99.67253
19	456.93319	157.55805

Specimen Type*WMA Technology
Least Squares Means Table

Level	Least Sq Mean	Std Error
Cores@Const., HMA	599.59226	223.18301
Cores@Const., WMA	598.12274	223.17083
LMLC, HMA	684.67667	223.12468
LMLC, WMA	633.78465	223.01576
PMPC, HMA	725.19219	223.10975
PMPC, WMA	629.12402	223.00803

LS Means Differences Tukey HSD
$\alpha = 0.050$

Level				Least Sq Mean
PMPC, HMA	A			725.19219
LMLC, HMA	A	B		684.67667
LMLC, WMA		B	C	633.78465
PMPC, WMA		B	C	629.12402
Cores@Const., HMA			C	599.59226
Cores@Const., WMA			C	598.12274

Levels not connected by same letter are significantly different.

Table E-3. Analysis results of the effect of WMA technology based on the model with main effects only.

Response M_R Stiffness (ksi)
Summary of Fit

RSquare	0.763931
RSquare Adj	0.759715
Root Mean Square Error	112.5958
Mean of Response	435.519
Observations (or Sum Wgts)	343

Fixed Effect Tests

Source	Nparm	DF	DFDen	F Ratio	Prob > F
Specimen Type	2	2	331	11.1538	<0.0001*
WMA Technology	1	1	331.2	15.7227	<0.0001*
NMAS (mm)	2	2	4.979	0.4185	0.6792
AV (%)	1	1	331.4	18.5589	<0.0001*

Effect Details
Specimen Type
Least Squares Means Table

Level	Least Sq Mean	Std Error
Cores@Const.	600.08926	223.27652
LMLC	658.35884	223.17444
PMPC	673.17074	223.16766

(continued on next page)

Table E-3. (Continued).

LS Means Differences Tukey HSD
$\alpha = 0.050$

Level			Least Sq Mean
PMPC	A		673.17074
LMLC	A		658.35884
Cores@Const.		B	600.08926

Levels not connected by same letter are significantly different.

WMA Technology
Least Squares Means Table

Level	Least Sq Mean	Std Error
HMA	669.09403	223.10848
WMA	618.65186	223.12016

NMAS (mm)
Least Squares Means Table

Level	Least Sq Mean	Std Error
9.5	643.87295	223.02364
12.5	420.33217	99.83569
19	458.19937	157.81461

Table E-4. Analysis results of the effect of production temperature.

Response M_R Stiffness (ksi)
Summary of Fit

RSquare	0.336584
RSquare Adj	0.30587
Root Mean Square Error	82.03103
Mean of Response	260.1842
Observations (or Sum Wgts)	114

Fixed Effect Tests

Source	Nparm	DF	DFDen	F Ratio	Prob > F
Specimen Type	2	2	107.4	9.4183	0.0002*
WMA Technology	1	1	106.2	4.4461	0.0373*
Production Temperature	1	1	107.5	0.0071	0.9329
AV (%)	1	1	107.7	3.3372	0.0705

Effect Details
Specimen Type
Least Squares Means Table

Level	Least Sq Mean	Std Error
Cores@Const.	210.45968	23.428893
LMLC	305.14533	22.979005
PMPC	267.66671	23.042212

LS Means Differences Tukey HSD
$\alpha = 0.050$

Level			Least Sq Mean
LMLC	A		305.14533
PMPC	A		267.66671
Cores@Const.		B	210.45968

Levels not connected by same letter are significantly different.

WMA Technology
Least Squares Means Table

Level	Least Sq Mean	Std Error
HMA	279.41688	22.815326
WMA	242.76426	20.164048

Production Temperature
Least Squares Means Table

Level	Least Sq Mean	Std Error
Control Temperature	261.74940	20.787180
High Temperature	260.43174	21.583983

specimen type as main effects, plant type*specimen type as a two-way interaction effect, AV as a covariate, and field site as a random effect was first fitted to the data. However, the two-way interaction effect plant type*specimen type was not statistically significant, and the ANCOVA model with plant type and specimen type as main effects, AV as a covariate, and field site as a random effect was fitted again to the data. Table E-5 contains the analysis outputs obtained by the REML method implemented in JMP. It can be observed from Table E-5 (see Fixed Effects Tests) that the effects of specimen type and AV were statistically significant at $\alpha = 0.05$, while the effect of plant type was not.

Inclusion of Recycled Materials (RAP/RAS vs. No RAP/RAS)

There were 79 non-missing M_R stiffness measurements obtained from two different field sites (Texas I, New Mexico), three specimen types, two levels of WMA technology, and two levels of recycled materials (no RAP/RAS vs. RAP/RAS) in the data for this analysis. The other relevant variable that was included in the analysis was AV. The value of NMAS was held constant at 19 mm for all of the M_R stiffness measurements in the data and was excluded from this analysis. The ANCOVA model including recycled materials, specimen type, and WMA technology as main effects along with all possible two-way interaction effects among them, AV as a covariate, and field site as a random effect was first fitted to the data. Because the WMA technology*recycled materials interaction effect was not statistically significant at $\alpha = 0.05$, it was removed, and the ANCOVA model with recycled materials, specimen type, and WMA technology as main effects, specimen type*WMA technology and specimen type*recycled materials as two-way interactions, AV as a covariate, and field site as a random effect was fitted again to the data.

Table E-6 contains the analysis output obtained by the REML method implemented in JMP. It can be observed from Table E-6 (see Fixed Effects Tests) that the effects of specimen type, WMA technology, recycled materials, specimen type*WMA technology, and specimen type*recycled materials were statistically significant at $\alpha = 0.05$, while the effect of AV was not. Table E-6 also presents the predicted values (least squares means) for M_R stiffness for each level of significant factors along with their standard errors. For specimen type*WMA technology and specimen type*recycled materials, a multiple comparison procedure (Tukey's HSD) was also carried out to test which of those factor levels were statistically different. It can be seen that for this dataset the difference between HMA and WMA was statistically significant for Cores@Const and PMPC but not for LMLC (Figure E-2a). It can also be observed from the interaction plot and Tukey's HSD test result that the difference between no RAP/RAS and RAP/RAS was statistically significant for each of Cores@Const, LMLC, and PMPC although the amount of

Table E-5. Analysis results of the effect of plant type.

Response M_R Stiffness (ksi)
Summary of Fit

RSquare	0.79484
RSquare Adj	0.784319
Root Mean Square Error	64.00318
Mean of Response	576.9036
Observations (or Sum Wgts)	83

Fixed Effect Tests

Source	Nparm	DF	DFDen	F Ratio	Prob > F
Specimen Type	2	2	77.02	34.5118	<0.0001*
Plant Type	1	1	77.01	0.3386	0.5623
AV (%)	1	1	77.06	19.5165	<0.0001*

Effect Details
Specimen Type
Least Squares Means Table

Level	Least Sq Mean	Std Error
Cores@Const.	487.23342	69.019735
LMLC	552.51341	68.516970
PMPC	662.81736	68.683869

Plant Type
Least Square Means Table

Level	Least Sq Mean	Std Error
BMP	563.30111	68.075887
DMP	571.74168	68.038493

Table E-6. Analysis results of the effect of recycled materials.

Response M_R Stiffness (ksi)
Summary of Fit

RSquare	0.738961
RSquare Adj	0.704913
Root Mean Square Error	96.6455
Mean of Response	473.443
Observations (or Sum Wgts)	79

Fixed Effect Tests

Source	Nparm	DF	DFDen	F Ratio	Prob > F
Specimen Type	2	2	68	9.1956	0.0003*
WMA Technology	1	1	68.07	33.9737	<0.0001*
Recycled Materials	1	1	68.34	132.8675	<0.0001*
AV (%)	1	1	68.03	0.8223	0.3677
Specimen Type*WMA Technology	2	2	68	3.1880	0.0475*
Specimen Type*Recycled Materials	2	2	68	9.4862	0.0002*

Effect Details
Specimen Type
Least Squares Means Table

Level	Least Sq Mean	Std Error
Cores@Const.	503.05058	85.383434
LMLC	395.63973	85.251826
PMPC	487.16675	85.259816

WMA Technology
Least Squares Means Table

Level	Least Sq Mean	Std Error
HMA	528.38576	84.750304
WMA	395.51894	84.505516

Recycled Materials
Least Squares Means Table

Level	Least Sq Mean	Std Error
No RAP/RAS	324.35229	84.888197
RAP/RAS	599.55241	84.516152

Specimen Type*WMA Technology
Least Squares Means Table

Level	Least Sq Mean	Std Error
Cores@Const., HMA	600.34646	88.702788
Cores@Const., WMA	405.75470	87.615965
LMLC, HMA	423.06789	87.716629
LMLC, WMA	368.21156	86.931067
PMPC, HMA	561.74294	87.675827
PMPC, WMA	412.59056	87.029846

LS Means Differences Tukey HSD
$\alpha = 0.050$

Level			Least Sq Mean
Cores@Const., HMA	A		600.34646
PMPC, HMA	A		561.74294
LMLC, HMA		B	423.06789
PMPC, WMA		B	412.59056
Cores@Const., WMA		B	405.75470
LMLC, WMA		B	368.21156

Levels not connected by same letter are significantly different.

Specimen Type*Recycled Materials
Least Squares Means Table

Level	Least Sq Mean	Std Error
Cores@Const., No RAP/RAS	307.57200	87.934962
Cores@Const., RAP/RAS	698.52915	87.629086
LMLC, No RAP/RAS	318.03673	87.970726
LMLC, RAP/RAS	473.24272	86.911187
PMPC, No RAP/RAS	347.44813	88.040685
PMPC, RAP/RAS	626.88537	86.917262

Table E-6. (Continued).

LS Means Differences Tukey HSD
$\alpha = 0.050$

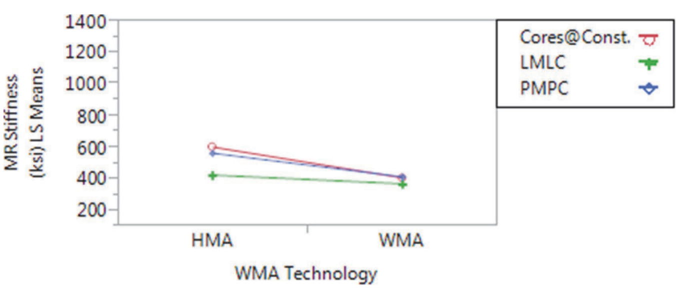
Level				**Least Sq Mean**
Cores@Const., RAP/RAS	A			698.52915
PMPC, RAP/RAS	A			626.88537
LMLC, RAP/RAS		B		473.24272
PMPC, No RAP/RAS			C	347.44813
LMLC, No RAP/RAS			C	318.03673
Cores@Const., No RAP/RAS			C	307.57200

Levels not connected by same letter are significantly different.

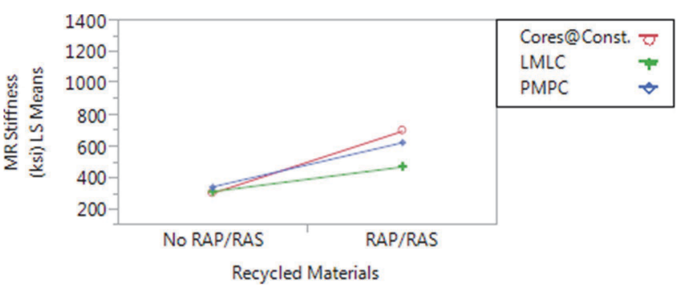
difference varied with specimen type (Figure E-2b). It can be concluded that, in general, mixtures with RAP/RAS had statistically higher M_R stiffness than mixtures with no RAP/RAS.

Aggregate Absorption (High- vs. Low-Absorptive Aggregate)

There were 119 non-missing M_R stiffness measurements obtained from two different field sites (Iowa, Florida), three specimen types, two levels of WMA technology, and two levels of aggregate absorption (low absorption vs. high absorption) in the data for this analysis. Another relevant variable that was included in the analysis was AV. The value of NMAS was held constant at 12.5 mm for all of the M_R stiffness measurements in the data and was excluded from this analysis. The ANCOVA model including aggregate absorption, specimen type, and WMA technology as main effects, AV as a covariate, and field site as a random effect was fitted to these data. Table E-7 contains the analysis output obtained by the REML method implemented in JMP. It can be observed from Table E-7 (see Fixed Effects Tests) that the effects of specimen type, WMA technology, and aggregate absorption were statistically significant at $\alpha = 0.05$, while the effect of AV was not. Table E-7 also presents the predicted values (least squares means) for M_R stiffness for each level of significant factors along with their standard errors.



(a) Specimen type*WMA technology



(b) Specimen type*recycled materials

Figure E-2. Least squares means or interaction plots.

Binder Source (Binder A vs. Binder V)

There were 35 non-missing M_R stiffness measurements obtained from three specimen types and two levels of binder source (Binder 1 vs. Binder 2) in the data for this analysis. The other relevant variable included in the analysis was AV. The value of NMAS was held constant at 9.5 mm and the field site was Texas II for all of the M_R stiffness measurements in the data. The ANCOVA model including binder source and specimen type as main effects and AV as a covariate was fitted to these data. Originally, a model including a two-way interaction, binder source*specimen type, was fitted, but the interaction was not statistically significant. Table E-8 contains the analysis outputs obtained by the REML method implemented in JMP. It can be observed from Table E-8 that the effects of binder source, specimen type, and AV were all statistically significant at $\alpha = 0.05$. Table E-8 also presents the predicted values for M_R stiffness for each level of significant factors. Specifically, Binder A yielded significantly higher M_R stiffness than Binder V.

Specimen Type

In addition to the analyses for the identification of significant factors, the analysis of the effect of specimen type on M_R stiffness was also conducted to determine if equivalent mixture stiffness is achieved by PMPC specimens and LMLC specimens as well as by cores at construction and LMLC specimens.

Table E-7. Analysis results of the effect of aggregate absorption.

Response M_R Stiffness (ksi)

Summary of Fit

RSquare	0.906554
RSquare Adj	0.902419
Root Mean Square Error	75.768
Mean of Response	442.0672
Observations (or Sum Wgts)	119

Fixed Effect Tests

Source	Nparm	DF	DFDen	F Ratio	Prob > F
Specimen Type	2	2	112	54.9220	<0.0001*
WMA Technology	1	1	112	9.7778	0.0023*
Aggregate Absorption	1	1	112	63.5877	<0.0001*
AV (%)	1	1	112	0.1203	0.7293

Effect Details

Specimen Type

Least Squares Means Table

Level	Least Sq Mean	Std Error
Cores@Const.	358.61853	207.00726
LMLC	561.80383	206.93033
PMPC	526.57987	206.93214

WMA Technology

Least Squares Means Table

Level	Least Sq Mean	Std Error
HMA	504.09964	206.78238
WMA	460.56851	206.77821

Aggregate Absorption

Least Squares Means Table

Level	Least Sq Mean	Std Error
High Absorption	425.22727	206.78724
Low Absorption	539.44088	206.78706

Table E-8. Analysis results of the effect of binder source.

Response M_R Stiffness (ksi)

Summary of Fit

RSquare	0.911424
RSquare Adj	0.899613
Root Mean Square Error	35.99742
Mean of Response	503.3714
Observations (or Sum Wgts)	35

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Ratio
Model	4	400005.74	100001	77.1726
Error	30	38874.43	1296	Prob > F
C. Total	34	438880.17		<0.0001*

Lack of Fit

Source	DF	Sum of Squares	Mean Square	F Ratio
Lack of Fit	15	27284.935	1819.00	2.3543
Pure Error	15	11589.500	772.63	Prob > F
Total Error	30	38874.435		0.0540
				Max RSq

Effect Tests

Source	Nparm	DF	Sum of Squares	F Ratio	Prob > F
Specimen Type	2	2	226092.08	87.2394	<0.0001*
Binder Source	1	1	80832.73	62.3799	<0.0001*
AV (%)	1	1	16571.39	12.7884	0.0012*

Effect Details

Specimen Type

Least Squares Means Table

Level	Least Sq Mean	Std Error	Mean
Cores@Const.	427.14738	11.302509	411.250
LMLC	470.13260	10.641587	478.333
PMPC	627.21056	11.136625	631.182

Table E-8. (Continued).

Binder Source			
Least Squares Means Table			
Level	**Least Sq Mean**	**Std Error**	**Mean**
Binder 1 (Binder A)	556.83935	8.8053350	553.000
Binder 2 (Binder V)	459.48767	8.5257073	456.500

There were 378 non-missing M_R stiffness measurements obtained from nine field sites, three specimen types, and three levels of NMAS (9.5 mm, 12.5 mm, 19 mm) in the data for this analysis. Another relevant variable that was included in the analysis was AV. The ANCOVA model including specimen type as a main effect, NMAS and AV as covariates, and field site as a random effect was fitted to the data. Table E-9 contains the analysis output obtained by the REML method implemented in JMP. It can be observed from Fixed Effects Tests that the effects of specimen type and AV were statistically significant at $\alpha = 0.05$, while the effect of NMAS was not. It also appears that higher values of AV are associated with lower values of M_R stiffness (see "Parameter Estimates" where the coefficient for AV is negative), as expected. Table E-9 also presents the predicted

Table E-9. Analysis results of the effect of specimen type.

Response M_R Stiffness (ksi)

Summary of Fit

RSquare	0.747895
RSquare Adj	0.744507
Root Mean Square Error	112.459
Mean of Response	441.8016
Observations (or Sum Wgts)	378

Parameter Estimates

Term	Estimate	Std Error	DFDen	t Ratio	Prob>|t|
Intercept	795.80309	156.8864	7.24	5.07	0.0013*
Specimen Type [Cores@Const.]	−45.92381	8.992407	366.1	−5.11	<0.0001*
Specimen Type [LMLC]	9.4336149	8.301336	366	1.14	0.2565
NMAS (mm) [12.5–9.5]	−163.0417	176.9445	5.979	−0.92	0.3925
NMAS (mm) [19–12.5]	40.390787	176.9656	5.982	0.23	0.8271
AV (%)	−30.53477	6.363505	366.5	−4.80	<0.0001*

Fixed Effect Tests

Source	Nparm	DF	DFDen	F Ratio	Prob > F
Specimen Type	2	2	366	14.6422	<0.0001*
NMAS (mm)	2	2	5.981	0.4252	0.6720
AV (%)	1	1	366.5	23.0248	<0.0001*

Effect Details

Specimen Type

Least Squares Means Table

Level	Least Sq Mean	Std Error
Cores@Const.	530.99022	149.83759
LMLC	586.34765	149.73227
PMPC	613.40423	149.74051

LS Means Differences Tukey HSD

$\alpha = 0.050$

Level			Least Sq Mean
PMPC	A		613.40423
LMLC	A		586.34765
Cores@Const.		B	530.99022

Levels not connected by same letter are significantly different.

NMAS (mm)

Least Squares Means Table

Level	Least Sq Mean	Std Error
9.5	576.91403	149.52323
12.5	413.87236	94.60771
19	454.26315	149.56134

values for M_R stiffness for each level of factors. For specimen type, a multiple comparison procedure (Tukey's HSD) was also carried out to test which levels were statistically different. It can be observed from "LS Means Differences Tukey HSD" that PMPC and LMLC lead to significantly higher M_R stiffness than Cores@Const while PMPC and LMLC are not statistically different from each other at $\alpha = 0.05$.

Phase II Experiment

The objective of the statistical analysis for Phase II was to identify factors with significant effects on M_R ratio of long-term aged asphalt mixtures. The factors of interest were:

1. WMA technology (HMA vs. WMA)
2. Production temperature (high temperature vs. control temperature)
3. Plant type (DMP vs. BMP)
4. Recycled materials (no RAP/RAS vs. RAP/RAS)
5. Aggregate absorption (low absorption vs. high absorption)

In addition to the aforementioned factors, the information on field site (Florida, Indiana, Iowa, New Mexico, South Dakota, Texas I, Wyoming) and aging level (Lab LTOA 1, Lab LTOA 2, Field Aging 1, Field Aging 2, and Field Aging 3—where each of three field aging levels are nested within the field site) was also available and incorporated into the analyses. Five separate experiments and analyses were performed to assess the effects of each of the five factors of interest listed above.

WMA Technology (HMA vs. WMA)

There were 105 non-missing M_R stiffness ratio measurements obtained from seven different field sites (Florida, Indiana, Iowa, New Mexico, South Dakota, Texas I, and Wyoming) and two levels of WMA technology in the data for this analysis. Another variable included in the analysis was aging level. Analysis of variance (ANOVA) having WMA technology and aging level as main effects along with a two-way interaction effect between them, and field site as a random effect was fitted to the data. However, the two-way interaction effect WMA technology*aging level was not statistically significant, and the ANOVA model with WMA technology and aging level as main effects and field site as a random effect was fitted again to the data. Table E-10 contains the analysis outputs obtained by the REML method implemented in the

Table E-10. Analysis results of the effect of WMA technology.

Response M_R Stiffness Ratio

Summary of Fit

RSquare	0.722275
RSquare Adj	0.68605
Root Mean Square Error	0.241657
Mean of Response	1.603867
Observations (or Sum Wgts)	105

Fixed Effect Tests

Source	Nparm	DF	DFDen	F Ratio	Prob > F
WMA Technology	1	1	86.53	9.2668	0.0031*
Aging Level	11	11	89.51	11.1913	<0.0001*

Effect Details

WMA Technology

Least Squares Means Table

Level	Least Sq Mean	Std Error
HMA	1.5874780	0.09972780
WMA	1.7371166	0.09737346

Aging Level

Least Squares Means Table

Level	Least Sq Mean	Std Error
Field Aging 1_FL	1.9166531	0.16887818
Field Aging 1_IA	1.4450960	0.13757681
Field Aging 1_IN	1.6608835	0.16887818
Field Aging 1_NM	1.0857561	0.16887818
Field Aging 1_SD	1.5925310	0.16888933
Field Aging 1_TX I	1.5054495	0.16476408
Field Aging 1_WY	0.8898908	0.16010955
Field Aging 2_FL	2.0004031	0.16887818
Field Aging 2_TX I	2.0336069	0.18696559
Field Aging 3_TX I	2.5919403	0.18696559
Lab LTOA 1	1.4584678	0.09949980
Lab LTOA 2	1.7668895	0.09927131

JMP statistical package. It can be observed from Table E-10 (see Fixed Effects Tests) that the effects of WMA technology and aging level were statistically significant at $\alpha = 0.05$. Table E-10 also presents the predicted values for M_R stiffness for each level of factors (see "Least Squares Means Table"). It can be seen that WMA led to a higher predicted M_R ratio value than HMA. Also, in general, the predicted M_R ratio seemed to increase as aging level increased.

Production Temperature (High vs. Control)

There were 36 non-missing M_R stiffness ratio measurements obtained from two different field sites (Iowa, Wyoming) and two levels of production temperature in the data for this analysis. Other relevant variables included in the analysis were WMA technology and aging level. The ANOVA model including production temperature, WMA technology, and aging level as main effects along with all possible two-way interactions and field site as a random effect was first fitted to the data, but none of the two-way interaction effects were statistically significant. Thus, two-way interaction effects were removed from the model, and the ANOVA model with main effects, production temperature, WMA technology and aging level, as fixed effects and field site as a random effect was fitted again to the data.

Table E-11 contains the analysis output obtained by the REML method implemented in JMP. It can be observed from Table E-11 (see "Fixed Effects Tests") that the effect of the factor of interest, production temperature, was not statistically significant at $\alpha = 0.05$.

Plant Type (BMP vs. DMP)

There were 12 non-missing M_R stiffness measurements obtained from one field site (Indiana) and two levels of plant type in the data for this analysis. Other relevant variables that could be included in the analysis were aging level and WMA technology. The ANOVA model including plant type, aging level, and WMA technology as main effects along with all possible two-way interactions among them was first fitted to the data. However, none of the two-way interaction effects were statistically significant, and the ANOVA model with main effects only was fitted again to the data. Table E-12 contains the analysis outputs obtained by the REML method implemented

Table E-11. Analysis results of the effect of production temperature.

Response M_R Stiffness Ratio
Summary of Fit

RSquare	0.683867
RSquare Adj	0.631178
Root Mean Square Error	0.206745
Mean of Response	1.400333
Observations (or Sum Wgts)	36

Fixed Effect Tests

Source	Nparm	DF	DFDen	F Ratio	Prob > F
Production Temperature	1	1	29	2.3413	0.1368
Aging Level	3	3	19.74	15.1443	<0.0001*
WMA Technology	1	1	29.84	3.8217	0.0600

Effect Details
Production Temperature
Least Squares Means Table

Level	Least Sq Mean	Std Error
Control Temperature	1.2538520	0.05916754
High Temperature	1.3619241	0.06452429

Aging Level
Least Squares Means Table

Level	Least Sq Mean	Std Error
Field Aging 1_IA	1.3328748	0.11401034
Field Aging 1_WY	0.8594769	0.12539081
Lab LTOA 1	1.3461258	0.07461122
Lab LTOA 2	1.6930748	0.07322558

WMA Technology
Least Squares Means Table

Level	Least Sq Mean	Std Error
HMA	1.2328373	0.07171734
WMA	1.3829388	0.05453665

Table E-12. Analysis results of the effect of plant type.

Response M_R Stiffness Ratio
Summary of Fit

RSquare	0.528446
RSquare Adj	0.258987
Root Mean Square Error	0.131003
Mean of Response	1.446583
Observations (or Sum Wgts)	12

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Ratio
Model	4	0.13462633	0.033657	1.9611
Error	7	0.12013258	0.017162	Prob > F
C. Total	11	0.25475892		0.2052

Effect Tests

Source	Nparm	DF	Sum of Squares	F Ratio	Prob > F
Plant Type	1	1	0.00156408	0.0911	0.7715
Aging Level	2	2	0.12721017	3.7062	0.0798
WMA Technology	1	1	0.00585208	0.3410	0.5776

Effect Details
Plant Type
Least Squares Means Table

Level	Least Sq Mean	Std Error	Mean
BMP	1.4351667	0.05348177	1.43517
DMP	1.4580000	0.05348177	1.45800

Aging Level
Least Squares Means Table

Level	Least Sq Mean	Std Error	Mean
Field Aging 1_IN	1.4935000	0.06550152	1.49350
Lab LTOA 1	1.3037500	0.06550152	1.30375
Lab LTOA 2	1.5425000	0.06550152	1.54250

WMA Technology
Least Squares Means Table

Level	Least Sq Mean	Std Error	Mean
HMA	1.4245000	0.05348177	1.42450
WMA	1.4686667	0.05348177	1.46867

in JMP. It can be observed from Table E-12 (see "Fixed Effects Tests") that none of the factor effects (as well as the factor of interest plant type) were statistically significant at $\alpha = 0.05$.

Inclusion of Recycled Material (RAP/RAS vs. No RAP/RAS)

There were 17 non-missing M_R stiffness ratio measurements obtained from two different field sites (Texas I, New Mexico), and two levels of recycled materials (no RAP/RAS vs. RAP/RAS) in the data for this analysis. Other relevant factors included in the analysis were aging level and WMA technology. Because field site was confounded with aging level for this dataset, field site could not be included as a random effect in the model in this case. The ANOVA model including recycled materials, aging level, and WMA technology as main effects along with all possible two-way interaction effects among them was first fitted to the data. Because none of the two-way interaction effects was statistically significant at $\alpha = 0.05$, the ANOVA model including main effects only was used.

Table E-13 contains the analysis output obtained by JMP. It can be observed from Table E-13 (see "Fixed Effects Tests") that the effects of recycled materials, aging level, and WMA technology were all statistically significant at $\alpha = 0.05$. Table E-13 also presents the predicted values (least squares means) for M_R ratio for each level of factors along with their standard errors. From this set of results, it can be concluded that mixtures with no RAP/RAS have a higher M_R ratio compared to mixtures with RAP/RAS.

Aggregate Absorption (High- vs. Low-Absorptive Aggregate)

There were 39 non-missing M_R stiffness ratio measurements obtained from two different field sites (Iowa, Florida) and two levels of aggregate absorption in the data for this analysis. Other relevant factors that were included in the analysis were aging level and WMA technology. The ANOVA model including aggregate absorption, aging level, and WMA technology as main effects along with all possible

Table E-13. Analysis results of the effect of recycled materials.

Response M_R Stiffness Ratio
Summary of Fit

RSquare	0.832989
RSquare Adj	0.757074
Root Mean Square Error	0.214076
Mean of Response	1.789824
Observations (or Sum Wgts)	17

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Ratio
Model	5	2.5143317	0.502866	10.9728
Error	11	0.5041147	0.045829	Prob > F
C. Total	16	3.0184465		0.0006*

Lack of Fit

Source	DF	Sum of Squares	Mean Square	F Ratio
Lack of Fit	7	0.38521025	0.055030	1.8512
Pure Error	4	0.11890450	0.029726	Prob > F
Total Error	11	0.50411475		0.2880
				Max RSq

Effect Tests

Source	Nparm	DF	Sum of Squares	F Ratio	Prob > F
Recycled Materials	1	1	0.8570860	18.7020	0.0012*
Aging Level	3	3	1.7065448	12.4125	0.0007*
WMA Technology	1	1	0.8227346	17.9524	0.0014*

Effect Details
Recycled Materials
Least Squares Means Table

Level	Least Sq Mean	Std Error	Mean
No RAP/RAS	2.1391094	0.09578452	1.94733
RAP/RAS	1.6087969	0.06723247	1.70391

Aging Level
Least Squares Means Table

Level	Least Sq Mean	Std Error	Mean
Field Aging 1_NM	1.6020781	0.11134181	1.46950
Field Aging 1_TX I	1.5340781	0.09763560	1.63420
Lab LTOA 1	2.0215781	0.11134181	1.88900
Lab LTOA 2	2.3380781	0.11134181	2.20550

WMA Technology
Least Squares Means Table

Level	Least Sq Mean	Std Error	Mean
HMA	1.6385000	0.07568736	1.63850
WMA	2.1094063	0.08138591	1.92433

two-way interaction effects among them, and field site as a random effect was first fitted to these data. Because the WMA technology*aging level interaction effect was not statistically significant at $\alpha = 0.05$, it was removed, and the ANOVA model with aggregate absorption, aging level, and WMA technology as main effects, aging level*aggregate absorption and WMA technology*aggregate absorption as two-way interaction effects, and field site as a random effect was fitted again to the data.

Table E-14 contains the analysis output obtained by the REML method implemented in JMP. It can be observed from Table E-14 (see "Fixed Effects Tests") that the effects of aggregate absorption, aging level, aging level*aggregate absorption, and WMA technology* aggregate absorption were statistically significant at $\alpha = 0.05$. Table E-14 also presents the predicted values (least squares means) for M_R ratio for each level of factors along with their standard errors. For WMA technology*aggregate absorption and aggregate aging level*absorption, a multiple comparison procedure (Tukey's HSD) was also carried out to test which of those factor levels are statistically different. It can be seen that for this dataset the difference between high absorption and low absorption was statistically significant for WMA but not for HMA (see "LS Means Differences Tukey HSD" for WMA technology*aggregate absorption and Figure E-3a). It can also be observed from the interaction plot (Figure E-3b) and Tukey's HSD test result that, although high absorption leads to higher M_R ratio for each level of aging, in general (except for Field Aging 1_IA), the difference between high absorption and low absorption was statistically significant only for Lab LTOA 2.

Table E-14. Analysis results of the effect of aggregate absorption.

Response M_R Stiffness Ratio

Summary of Fit

RSquare	0.764182
RSquare Adj	0.668108
Root Mean Square Error	0.155178
Mean of Response	1.454256
Observations (or Sum Wgts)	39

Fixed Effect Tests

Source	Nparm	DF	DFDen	F Ratio	Prob > F
Aggregate Absorption	1	1	26	12.9133	0.0013*
Aging Level	4	4	26.41	10.7705	<0.0001*
WMA Technology	1	1	26.01	2.8022	0.1061
Aging Level*Aggregate Absorption	4	4	26	3.0540	0.0345*
WMA Technology*Aggregate Absorption	1	1	26.01	5.0244	0.0337*

Effect Details

Aggregate Absorption

Least Squares Means Table

Level	Least Sq Mean	Std Error
High Absorption	1.6080909	0.09268907
Low Absorption	1.4084768	0.09291451

Aging Level

Least Squares Means Table

Level	Least Sq Mean	Std Error
Field Aging 1_FL	1.7168638	0.12146761
Field Aging 1_IA	1.2385112	0.10837145
Field Aging 2_FL	1.8006138	0.12146761
Lab LTOA 1	1.2523017	0.09692408
Lab LTOA 2	1.5331287	0.09618523

WMA Technology

Least Squares Means Table

Level	Least Sq Mean	Std Error
HMA	1.4665357	0.09216360
WMA	1.5500320	0.09182246

Aging Level*Aggregate Absorption

Least Squares Means Table

Level	Least Sq Mean	Std Error
Field Aging 1_FL, High Absorption	1.8803638	0.14413327
Field Aging 1_FL, Low Absorption	1.5533638	0.14413327
Field Aging 1_IA, High Absorption	1.2031362	0.12146761
Field Aging 1_IA, Low Absorption	1.2738862	0.12146761
Field Aging 2_FL, High Absorption	1.9583638	0.14413327
Field Aging 2_FL, Low Absorption	1.6428638	0.14413327
Lab LTOA 1, High Absorption	1.2865454	0.10610500
Lab LTOA 1, Low Absorption	1.2180580	0.10948715
Lab LTOA 2, High Absorption	1.7120454	0.10610500
Lab LTOA 2, Low Absorption	1.3542121	0.10610500

LS Means Differences Tukey HSD

$\alpha = 0.050$

Level			Least Sq Mean
Field Aging 2_FL, High Absorption	A		1.9583638
Field Aging 1_FL, High Absorption	A		1.8803638
Lab LTOA 2, High Absorption	A		1.7120454
Field Aging 2_FL, Low Absorption	A	B	1.6428638
Field Aging 1_FL, Low Absorption	A	B	1.5533638
Lab LTOA 2, Low Absorption		B	1.3542121
Lab LTOA 1, High Absorption		B	1.2865454
Field Aging 1_IA, Low Absorption		B	1.2738862
Lab LTOA 1, Low Absorption		B	1.2180580
Field Aging 1_IA, High Absorption		B	1.2031362

Levels not connected by same letter are significantly different.

Table E-14. (Continued).

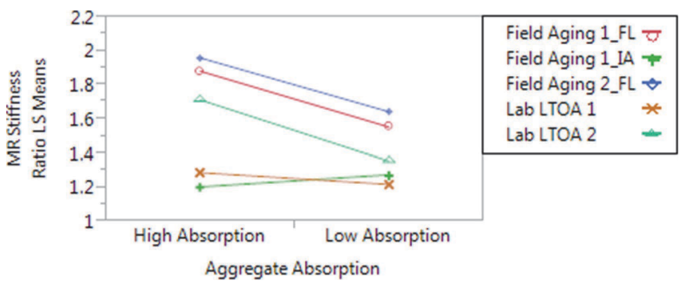
WMA Technology*Aggregate Absorption
Least Squares Means Table

Level	Least Sq Mean	Std Error
HMA, High Absorption	1.5104409	0.09897104
HMA, Low Absorption	1.4226305	0.10016495
WMA, High Absorption	1.7057409	0.09897104
WMA, Low Absorption	1.3943230	0.09899987

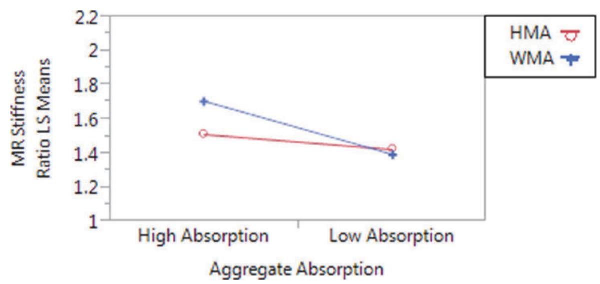
LS Means Differences Tukey HSD
α=0.050

Level			Least Sq Mean
WMA, High Absorption	A		1.7057409
HMA, High Absorption		B	1.5104409
HMA, Low Absorption		B	1.4226305
WMA, Low Absorption		B	1.3943230

Levels not connected by same letter are significantly different.



(a) Aging level*aggregate absorption



(b) WMA technology*aggregate absorption

Figure E-3. Least squares means or interaction plots.

APPENDIX F

Proposed AASHTO Recommended Practice for Conducting Plant Aging Studies

Standard Recommended Practice for

Measuring the Effects of Asphalt Plant Mixing and Processing on Binder Absorption by Aggregate and Asphalt Mixture Characteristics

AASHTO Designation: X xx-xx

INTRODUCTION

The effects of plant production on asphalt material properties have been researched for many years resulting in a number of changes in mixture design procedures to produce laboratory specimens with the same degree of aging as mixtures processed through a batch or drum mixing plant. The importance of being able to simulate plant aging in the laboratory is high for mixture volumetric considerations such as asphalt absorption, and it is absolutely essential for performance testing of mixtures for moisture, rutting, and fatigue susceptibility.

National Cooperative Highway Research Program (NCHRP) Project 9-52 provided guidance on laboratory short-term oven aging (STOA) techniques for warm mix asphalt (WMA) and hot mix asphalt (HMA) that were able to simulate plant aging over a wide range of variables, including WMA technology, aggregate asphalt absorption, recycled material content, and asphalt source. Recent and potential future changes in the production and formulations of asphalt mixtures such as the use of new WMA additives, higher recycled material content, and the use of asphalt recycling agents may require evaluations of their effects upon short-term aging.

1. SCOPE

1.1. This recommended practice provides a structure for conducting studies of mixture plant aging effects on asphalt mixtures. It is intended as a general guidance document that may need to be adapted for specific variables and conditions. It reflects the experience of the research team in NCHRP Project 9-52. An overall flowchart for a short-term aging study is shown in Figure 1.

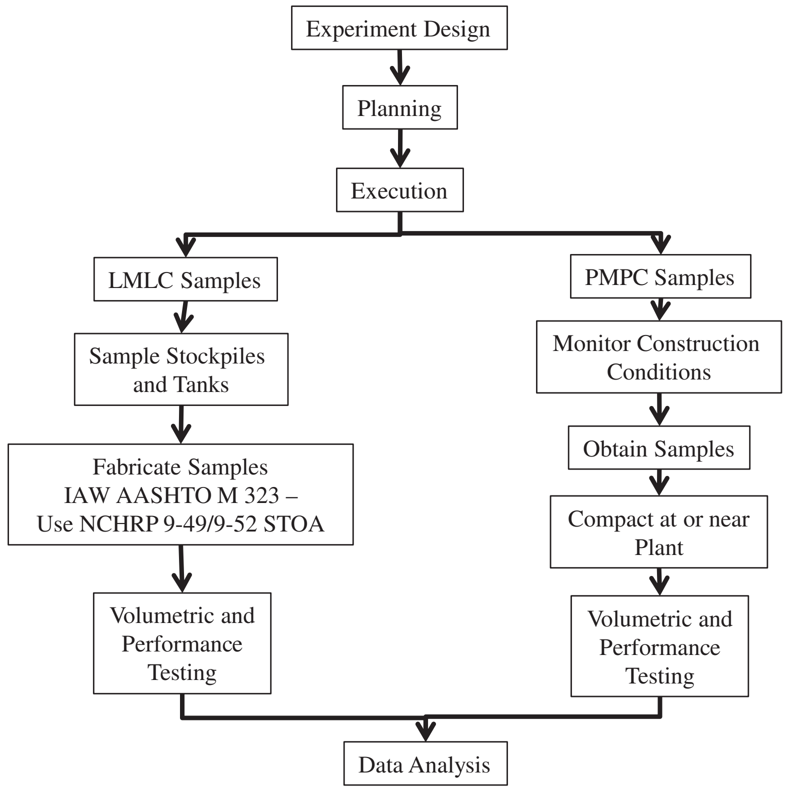


Figure 1. Overall Flow Chart for Conducting Short-Term Aging Studies of Asphalt Mixtures

1.2. *This standard may involve hazardous materials, operations, and equipment. This standard does not propose to address all safety problems associated with its usage. It is the duty and responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

Note 1—The values stated in SI units are to be regarded as the standard.

2. REFERENCED DOCUMENTS

2.1. Newcomb, D., A. Epps Martin, F. Yin, E. Arambula, E.S. Park, A. Chowdhury, R. Brown, C. Rodezno, N. Tran, E. Coleri, D. Jones, J.T. Harvey, and J.M. Signore. *NCHRP Report 815: Short-Term Laboratory Conditioning of Asphalt Mixtures*, Transportation Research Board, 2015.

2.2. Hogg, R.V. and J. Ledolter. *Engineering Statistics*, Macmillan, 1987.

3. EXPERIMENTAL DESIGN

3.1. The most important part of conducting an aging study is to define the goals and desired outcomes in terms of what questions are to be answered. Variables that may be of interest could include the effect of:

- Aggregate sources
- Asphalt sources
- WMA technologies
- Presence and concentration of recycled binder replacement
- Asphalt grades
- Aggregate gradations

3.2. If a relatively few factors and levels of factors are included in the study, a full-factorial experiment will give the most definitive results. Each factor and level will comprise one cell in the experiment design. The number of replicate samples to be prepared for each cell should be no less than three, and should be increased for larger variability. It should be noted that the addition of a factor with two levels doubles the number of cells in the experiment design matrix and a full-factorial experiment design will soon become unwieldy. If there are a moderate to high number of cells, then greater cost and time efficiency can be gained by a partial factorial experiment wherein only some fraction of the cells in the factorial are populated.

3.3. The types of samples to be obtained should also be considered in the experiment design. It is possible that, for the laboratory portion of the study [laboratory-mixed, laboratory-compacted (LMLC) samples], more factors and levels may be desirable to obtain a more detailed, controlled set of data on the effects of factors at various levels. In fact, a preliminary laboratory study may highlight certain variables of interest and discount others so that a more robust field study may be conducted.

3.4. The field portion of the study should be designed to obtain the needed comparison between factors and levels as efficiently as possible. Thus, trends in the laboratory data from LMLC samples should be studied before a given factor or levels of a factor are considered for production in an asphalt plant. Samples taken during production are referred to as plant-mixed, plant-compacted (PMPC) specimens and are taken as near as practical to the point of production. The factors and levels for the plant portion of the study should be selected to be as efficient as possible to limit costs and the number of plant changes required in production.

3.5. An example of a simple experiment design to study the differences in short-term aging between WMA and HMA from two different asphalt binder sources (S1 and S2) are shown in Figure 2. Here there are only two factors—asphalt mix type and asphalt binder source—with two levels each. For the sake of illustration, three replicates (R1, R2, and R3) represent the individual tests in each cell. The number of replicates in the cells should be established based upon the reproducibility of the test (within-laboratory variability). The greater the variability, the greater the number of samples required.

Factor		Asphalt Mix Type	
		WMA	HMA
Binder Source	S1	R1, R2, R3	R1, R2, R3
	S2	R1, R2, R3	R1, R2, R3

Figure 2. An Experimental Design for Asphalt Mix Type and Binder Source

3.6. The data analysis of the matrix is performed through an analysis of variance (ANOVA) to determine if the means of the cells are significantly different from each other. Following this, a pair-wise comparison of means [such as Tukey's honest significant difference (HSD) method] may be done to rank and group the levels of factors into those that are the same and those that are different. A good reference on experiment design is Hogg and Ledolter (1987).

4. PLANNING

4.1. Once the experiment design has been established, planning the schedule for sampling, sample preparation, and testing may begin. This process ensures that the proper amount of material for LMLC and PMPC samples are obtained from the specified points of sampling.

4.2. A checklist should be made to ensure that all the needed laboratory equipment and supplies are ready. This checklist should include all the equipment and supplies for volumetric testing and any desired performance testing.

4.3. Sampling for LMLC specimens should include aggregate and reclaimed asphalt pavement (RAP) stockpiles to be used in the mixtures as well as the virgin asphalt binder and any additives. Sampling and testing these materials should be as close as possible to the time of production. This will help ensure that the materials sampled will be those used in production.

4.4. Sampling asphalt mixtures at the plant should be done as close to the point of loadout from the silo as possible. Ideally, the plant asphalt mix for PMPC should be taken from a sampling platform prior to trucks exiting the plant site. A platform may not always be available, especially in the case of a portable plant. In these instances, the sample may be taken from the pavement site, preferably from a windrow, provided the job site is no more than a 20-minute drive from the laboratory where the PMPC sample will be compacted.

4.5. For each cell in the experiment design, the engineer or technician will need to determine the amount of material needed in each cell. Normally, a conservative approach of ordering 20 to 30 percent over the amount required for each cell works best to account for potential errors in fabrication and handling of specimens.

5. LABORATORY-MIXED, LABORATORY-COMPACTED SPECIMENS

5.1. Preparation:

5.1.1. As stated above, materials need to be sampled from aggregate stockpiles, RAP stockpiles, asphalt binder tanks, and additive bins or tanks in quantities sufficient to prepare enough LMLC samples for the study. The mixture design procedure in AASHTO M 323 should be followed in the preparation of laboratory samples with the exception of the STOA protocol for both volumetric and performance testing specimens.

5.1.2. NCHRP Projects 9-49 and 9-52 have shown that using an STOA process of storing the loose mix in an oven at 240°F (116°C) for WMA and at 275°F (135°C) for HMA at two hours prior to compaction produces volumetric, stiffness, and rutting resistance values which match plant-produced mixtures very closely. This differs from AASHTO M 323 which requires an STOA of 275°F (135°C) for two hours for volumetric testing and four hours for performance testing regardless of the mixture type. Thus, for LMLC specimens, the STOA of 240°F (116°C) for WMA and at 275°F (135°C) for HMA at two hours should be used for both volumetric and performance testing.

5.1.3. Compaction of the specimens should follow the number of required gyrations for 96 percent of maximum density for volumetric testing and 93 percent of maximum density for performance testing. Once the LMLC specimens have been compacted, they should be tested for volumetric mix parameters and appropriately shaped for performance testing.

5.2. Testing:

5.2.1. Volumetric and performance testing should follow sample preparation as closely as possible. Normally, testing should be accomplished within 48 hours of sample fabrication if stored in a 77°F (25°C) room or within two weeks if stored in a 60°F (15°C) chamber.

5.2.2. Volumetric parameters that may be of interest to compare LMLC short-term aged with PMPC specimens include maximum specific gravity (G_{mm}), effective binder content (P_{be}), percent binder absorbed (P_{ba}), and film thickness (FT). As shown in Figures 2 and 3, volumetric properties tracked very well between LMLC specimens and PMPC specimens with the exception of one mixture with a very highly absorptive aggregate. However, this procedure can be followed if there is a suspicion that the volumetrics between mixture design and production are significantly different.

5.2.3. Performance testing may include stiffness measurements such as resilient modulus or dynamic modulus; a rutting test such as Hamburg wheel-tracking test, asphalt pavement analyzer, or flow number; or a cracking test such as the semi-circular beam, disk compact-shaped tension, or the overlay tester. In NCHRP 9-52 it was noted that stiffness measurements were the best discriminators of performance between different mix parameters and plant characteristics. Although cracking evaluation was not a part of NCHRP 9-52, a complete field study is being design for cracking in NCHRP 9-57.

6. PLANT-MIXED, PLANT-COMPACTED SPECIMENS

6.1. Plant Operations: The field portion of the aging study may be conducted using either a drum mix plant or a batch plant. NCHRP Project 9-52 confirmed that both types of plants are capable of producing similar mixtures. Ensure that the asphalt plant is operating well and the proper materials are being introduced in the quantities determined from the mixture design. A minimum run of 500 tons of asphalt mix is required to be sure that the plant is operating normally before sampling the material for each condition to be studied. So, for the experiment design presented earlier, at least 500 tons of mix would need to be produced at each combination of asphalt mixture type and asphalt binder source.

6.2. Sampling and Preparation:

6.2.1. The plant-produced mixture should be sampled out of the back of a dump truck from a sampling platform positioned just after the trucks pull off the weigh scale at the plant. If circumstances prevent sampling from the truck, loose mix samples may be taken from the paving site and transported to the field lab for compaction provided that the time between production and sampling is no longer than 20 minutes and that the travel time from the paving site to the laboratory is no longer than 20 minutes.

6.2.2. After sampling the plant-produced mixture, it should be immediately transported to the field testing laboratory. Once at the laboratory, individual samples for compaction should be weighed into metal bowls and placed in an oven until the material reaches the field compaction temperature. The samples should then be left in the oven until they have reached the compaction temperature. Once the mixture has achieved the field compaction temperature, the samples should be compacted in a Superpave gyratory compactor to 96 percent of G_{mm} for volumetric testing and 93 percent of G_{mm} for performance testing.

6.3. Testing:

6.3.1. As with the LMLC mixtures, volumetric and performance testing should follow sample preparation as closely as possible. Normally, testing should be accomplished within 48 hours of sample fabrication if stored in a 77°F (25°C) room or within two weeks if stored in a 60°F (15°C) chamber.

6.3.2. The same volumetric parameters measured for the LMLC short-term aged specimens should be measured for the PMPC samples including G_{mm}, P_{be}, P_{ba}, and FT. A comparison of these parameters with the LMLC volumetric measurements will verify that the plant-produced materials are sufficiently similar to those produced in mixture design so that the comparison performance testing results will not be affected by extraneous variables. Performance testing should include the same stiffness measurements, rutting tests, and cracking tests that were used in evaluating the LMLC material.

7. DATA ANALYSIS

7.1. The first step in data analysis should be to compare the results of LMLC volumetric and performance testing against that of the PMPC. A cell-by-cell comparison can be made by plotting the means of the cells for the two sample types against one another along a 45° line of equality as shown in Figure 3. This will show whether there is a bias toward either the lab-produced or field-produced material. If the data points fall close to the line of equality, then it is likely that there are minimal differences between the two types of preparation.

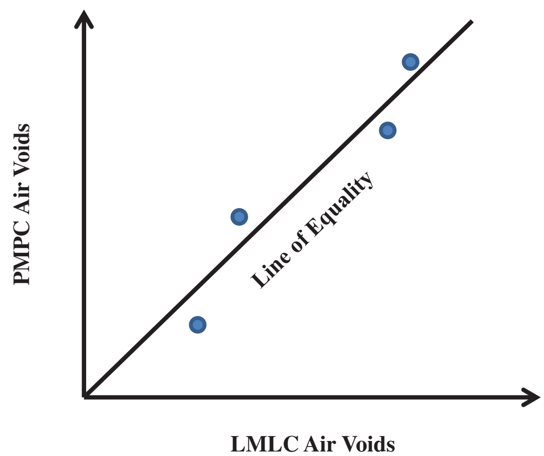


Figure 3. Comparison of overall LMLC and PMPC results.

7.2. Next, a factor analysis can be done by comparing the results by using ANOVA as previously described to compare the means of different levels of factors. This will establish whether a factor influences the measured parameter. For instance, Figure 4 shows a hypothetical outcome in which WMA is shown to be less stiff than HMA.

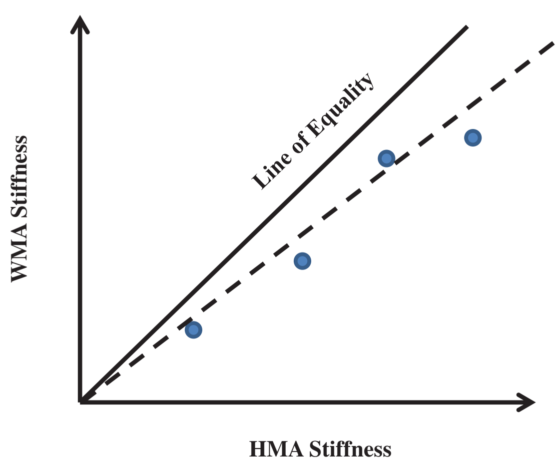


Figure 4. Stiffness Comparison for HMA versus WMA

7.3. Once the ANOVA has been completed, an additional step of ranking the data can be accomplished using Tukey's method, for instance. This type of ranking is very beneficial when there are a number of levels of each factor.

7.4. It is strongly suggested that, as more factors and levels are considered, the advice of a statistics expert should be sought at the experiment design and data analysis stages.

APPENDIX G

Recommended Changes to Standard Practice for Mixture Conditioning of Hot Mix Asphalt (HMA)

AASHTO Designation: R 30-02 (~~2010~~20XX)[i]

1. SCOPE

1.1. This standard practice describes procedures for mixture conditioning of compacted and uncompacted hot mix asphalt (HMA). ~~Three~~ Two types of conditioning are described: (1) short-term mixture conditioning for volumetric mixture design ~~; (2) short-term conditioning for mixture~~and mechanical property testing (both of which simulate the precompaction phase of the construction process); and (~~3~~2) long-term conditioning for mixture mechanical property testing to simulate the aging that occurs over the service life of a pavement. The procedures for long-term conditioning for mixture mechanical property testing are preceded by the procedure for short-term conditioning ~~for mixture mechanical property testing~~.

1.2. *This standard practice may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. REFERENCED DOCUMENTS

2.1. *AASHTO Standards*:

- PP 3, Preparing Hot Mix Asphalt (HMA) Specimens by Means of the Rolling Wheel Compactor[ii]
- T 312, Preparing and Determining the Density of Hot Mix Asphalt (HMA) Specimens by Means of the Superpave Gyratory Compactor
- ~~T 316, Viscosity Determination of Asphalt Binder Using Rotational Viscometer~~

3. SUMMARY OF PRACTICE

For short-term mixture conditioning ~~for volumetric mixture design~~, a mixture of aggregate and binder is conditioned in a forced-draft oven for 2 h at ~~the mixture's specified compaction temperature~~116°C for warm mix asphalt (WMA) and at 135°C for hot mix asphalt (HMA). ~~For~~

~~short-term mixture conditioning for mechanical property testing, a mixture of aggregate and binder is conditioned in a forced-draft oven for 4 h at 135°C.~~ For long-term mixture conditioning for mechanical property testing, a compacted mixture of aggregate and binder is conditioned in a forced-draft oven for 5 days at 85°C.

4. SIGNIFICANCE AND USE

The properties and performance of HMA can be more accurately predicted by using conditioned test samples. The short-term mixture conditioning ~~for the volumetric mixture design procedure~~ is designed to allow for binder absorption during the mixture design~~. The short-term mixture conditioning for the mechanical property testing procedure is designed~~ and to simulate the plant-mixing ~~and construction~~ effects on the mixture. The long-term mixture conditioning for the mechanical property testing procedure is designed to simulate the aging the compacted mixture will undergo during ~~7 to 10~~the first 1 to 3 years of service.

5. APPARATUS

5.1. *Oven*—A forced-draft oven, thermostatically controlled, capable of maintaining any desired temperature setting from room temperature to 176°C within ±3°C.

5.2. *Thermometers*—Thermometers having a range from 50 to 260°C and readable to 1°C.

5.3. *Miscellaneous*—A metal pan for heating aggregates, a shallow metal pan for heating uncompacted HMA, a metal spatula or spoon, timer, and gloves for handling hot equipment.

6. HAZARDS

6.1. This standard involves the handling of hot binder, aggregate, and HMA, which can cause severe burns if allowed to contact skin. Follow standard safety precautions to avoid burns.

7. MIXTURE CONDITIONING PROCEDURES

7.1. *Short-Term Mixture Conditioning ~~for Volumetric Mixture Design~~*:

7.1.1. The short-term mixture conditioning ~~for the volumetric mixture design procedure~~ applies to laboratory- prepared, loose mixture only. No mixture conditioning is required when conducting quality control or quality assurance testing on plant-produced mixture.

Note 1—The agency may identify the need to ~~condition~~ heat the plant-produced mixture to ~~be more~~ its compaction temperature to represent~~ative of~~ field conditions~~, particularly where absorptive aggregates are used~~.

7.1.2. Place the mixture in a pan, and spread it to an even thickness ranging between 25 and 50 mm. Place the mixture and pan in a forced-draft oven for 2 h ± 5 min at a temperature ~~equal to the mixture's compaction~~of 116 ± 3°C for WMA or 135 ± 3°C for HMA. ~~temperature ±3°C. The compaction temperature range of a HMA mixture is defined as the range of temperatures where the unaged binder has a kinematic viscosity of 280 ± 30 mm²/s (approximately 0.28 ± 0.03~~

~~Pa·s) measured in accordance with T 316 (Note 2). The target compaction temperature is generally the midpoint of this range.~~

Note 2—~~Modified binders may not adhere to the equi-viscosity requirements noted. T~~For modified binders, the agency should consider the manufacturer's recommendations when establishing the mixing and compaction temperatures for modified binders. Practically, the mixing temperature should not exceed 165°C and the compaction temperature should not be lower than 115°C.

7.1.3. Stir the mixture after 60 ± 5 min to maintain uniform conditioning.

7.1.4. After 2 h ± 5 min, remove the mixture from the forced-draft oven. The conditioned mixture is now ready for compaction or testing.

~~*Short-Term Conditioning for Mixture Mechanical Property Testing*:~~

~~The short-term conditioning for the mixture mechanical property testing procedure applies to laboratory-prepared, loose mix only.~~

~~Place the mixture in a pan, and spread it to an even thickness ranging between 25 and 50 mm. Place the mixture and pan in the conditioning oven for 4 h ± 5 min at a temperature of 135 ± 3°C.~~

~~Stir the mixture every 60 ± 5 min to maintain uniform conditioning.~~

~~After 4 h ± 5 min, remove the mixture from the forced-draft oven. The conditioned mixture is now ready for further conditioning or testing as required.~~

7.2. *Long-Term Mixture Conditioning ~~for Mixture Mechanical Property Testing~~*:

7.2.1. The long-term conditioning for the mixture mechanical property testing procedure applies to laboratory-prepared mixtures that have been subjected to the short-term conditioning ~~for the mixture mechanical property testing~~ procedure described in Section 7.2, plant-mixed HMA, and compacted roadway specimens.

7.2.2. *Preparing Specimens from Loose HMA*:

7.2.2.1. Specimens Compacted Using the Superpave Gyratory Compactor:

7.2.2.1.1. Compact the specimens in accordance with T 312. Cool the test specimen at room temperature for 16 ± 1 h.

Note 3—Extrude the specimen from the compaction mold after cooling for 2 to 3 h.

Note 4—Specimen cooling is usually scheduled as an overnight step. Cooling may be accelerated by placing the specimen in front of a fan.

7.2.2.2. Specimens Compacted Using the Rolling Wheel Compactor:

7.2.2.2.1. Compact the specimens in accordance with PP 3.

7.2.2.2.2. Cool the test specimen at room temperature for 16 ± 1 h.

7.2.2.2.3. Remove the slab from the mold, and saw or core the required specimens from the slab.

7.2.3. *Preparing Compacted Roadway Specimens*:

7.2.3.1. Cool test specimens at room temperature for 16 ± 1 h.

7.2.4. *Long-Term Conditioning of Prepared Test Specimens*—Place the compacted test specimens in the conditioning oven for 120 ± 0.5 h at a temperature of 85 ± 3°C.

7.2.5. After 120 ± 0.5 h, turn the oven off; open the doors and allow the test specimen to cool to room temperature. Do not touch or remove the specimen until it has cooled to room temperature.
Note 5—Cooling to room temperature will take approximately 16 h.

7.2.6. After cooling to room temperature, remove the test specimen from the oven. The long-term-conditioned specimen is now ready for testing as required.

8. Report

8.1. Report the binder grade, binder content (nearest 0.1 percent), and the aggregate type and gradation, if applicable.

8.2. Report the following short-term mixture conditioning information~~ for the volumetric mixture design conditions, if applicable~~:

8.2.1. Mixture conditioning temperature in laboratory (~~compaction temperature, ~~nearest 1°C);

8.2.2. Mixture conditioning duration in laboratory (nearest minute); and

8.2.3. Laboratory compaction temperature (nearest 1°C).

~~Report the following short-term conditioning information for the mixture mechanical property testing conditions, if applicable:~~

~~Short-term mixture conditioning temperature in laboratory (nearest 1°C);~~

~~Short-term mixture conditioning duration in laboratory (nearest minute); and~~

~~Laboratory compaction temperature (nearest 1°C).~~

8.3. Report the following long-term conditioning information~~ for the mixture mechanical property testing conditions~~, if applicable:

8.3.1. Laboratory compaction temperature (nearest 1°C);

8.3.2. Long-term mixture conditioning temperature in laboratory (nearest 1°C); and

8.3.3 Long-term mixture conditioning duration in laboratory (nearest 5 min).

9. Keywords

9.1 Conditioning; hot mix asphalt; long-term conditioning; short-term conditioning.

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[i] This standard is based on ~~SHRP Product 1031~~the results of NCHRP Project 9-52.
[ii] PP 3-94 was last printed in the May 2002 Edition of the AASHTO Provisional Standards.

Abbreviations and acronyms used without definitions in TRB publications:

A4A	Airlines for America
AAAE	American Association of Airport Executives
AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ACI–NA	Airports Council International–North America
ACRP	Airport Cooperative Research Program
ADA	Americans with Disabilities Act
APTA	American Public Transportation Association
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
ATA	American Trucking Associations
CTAA	Community Transportation Association of America
CTBSSP	Commercial Truck and Bus Safety Synthesis Program
DHS	Department of Homeland Security
DOE	Department of Energy
EPA	Environmental Protection Agency
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
FMCSA	Federal Motor Carrier Safety Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
HMCRP	Hazardous Materials Cooperative Research Program
IEEE	Institute of Electrical and Electronics Engineers
ISTEA	Intermodal Surface Transportation Efficiency Act of 1991
ITE	Institute of Transportation Engineers
MAP-21	Moving Ahead for Progress in the 21st Century Act (2012)
NASA	National Aeronautics and Space Administration
NASAO	National Association of State Aviation Officials
NCFRP	National Cooperative Freight Research Program
NCHRP	National Cooperative Highway Research Program
NHTSA	National Highway Traffic Safety Administration
NTSB	National Transportation Safety Board
PHMSA	Pipeline and Hazardous Materials Safety Administration
RITA	Research and Innovative Technology Administration
SAE	Society of Automotive Engineers
SAFETEA-LU	Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users (2005)
TCRP	Transit Cooperative Research Program
TDC	Transit Development Corporation
TEA-21	Transportation Equity Act for the 21st Century (1998)
TRB	Transportation Research Board
TSA	Transportation Security Administration
U.S.DOT	United States Department of Transportation

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