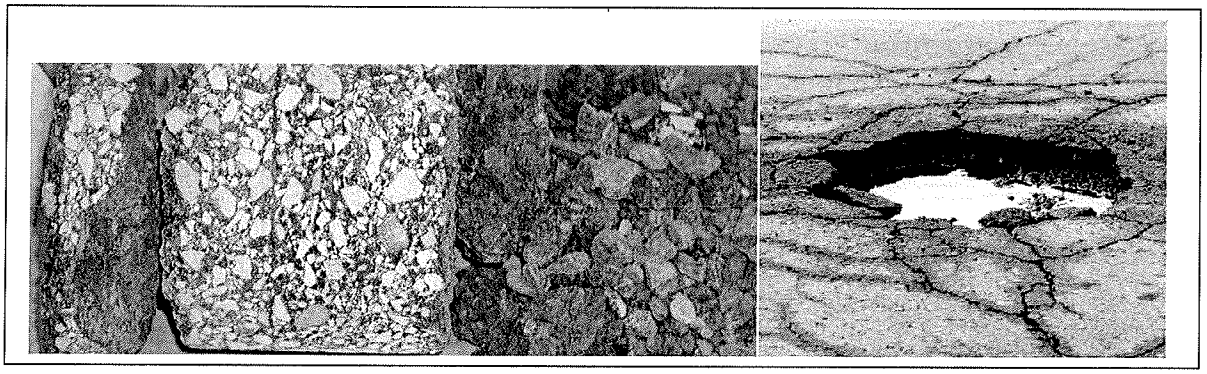
2.2 Moisture Susceptibility

Moisture susceptibility of asphalt concrete paving materials indicates the proclivity for specific combinations of binder and aggregate to sustain damage or a loss in functionality due to the detrimental effects of moisture under repetitive traffic loading. Moisture required to cause this type of damage is present in pavements in both liquid and vapor form, entering the pavement through infiltration of surface water by gravity, tire action, or irrigation; permeation of water vapor; capillary rise of subsurface water; and through the mixture itself (wet aggregates) (Arambula Mercado, 2007; Santucci, 2010). Figure 2.2 shows that moisture damage can manifest in a range of overall and localized distresses (premature permanent deformation or fatigue cracking, accelerated formation of potholes, or delamination between pavement layers) caused by a loss of cohesion in the binder (stiffness reduction) and/or a loss of adhesion between the component materials (stripping) (Kiggundu and Roberts, 1988, Santucci, 2010; Stroup-Gardiner, 1995).



(a) (b)

Figure 2.2. Examples of Moisture Damage: (a) Stripping and (b) Pothole (Santucci, 2010)

Moisture damage has been a major concern for many years, with exclusive attention focused on this topic in several national and international conferences. Examples of such events are the American Society for Testing and Material (ASTM) Symposium on Water Damage of Asphalt Pavements: Its Effect and Prevention in Williamsburg, Virginia in 1984; the Moisture Damage Symposium in Laramie Wyoming in 2002; the Moisture Sensitivity of Asphalt Pavements: A National Seminar in San Diego, California in, 2003; and the International Workshops on Moisture Induced Damage of Asphaltic Mixes in The Netherlands in 2005 and in College Station, Texas in 2007. Several state-of-the-art and synthesis reports (Hicks, 1991; Kiggundu and Roberts, 1988; Majidzadeh and Brovold, 1968; Stuart, 1990; Taylor and Khosla, 1983) also dealt

with this issue. More recent comprehensive summaries come from Santucci (2010) who provides an excellent technology transfer update on minimizing moisture damage and Caro et al. (2008a; 2008b) who provide a technical overview of the mechanisms of moisture damage. This section provides a summary of the literature on mechanisms and minimization strategies and laboratory characterization, including moisture conditioning protocols, tests, and corresponding relationships to field performance.

2.2.1 Mechanisms and Minimization Strategies

To take effective measures to preclude moisture damage, a comprehensive understanding of the chemical and mechanical mechanisms for the causes of damage is needed (Scherocman et al. 1985). There are two major causes of moisture damage: (1) the loss of adhesive bonding between the binder or the mastic (including fine aggregates) and the aggregates and (2) the loss of cohesion in the mastic due the presence of moisture (Little and Jones, 2003). Research over many years has identified the following six processes that contribute, usually in combination, to these causes (Santucci, 2010; Sebaaly et al., 2010; Taylor and Khosla, 1983):

- detachment of the binder film from the aggregate without film rupture
- displacement of the binder film from the aggregate through film rupture
- spontaneous emulsification and formation of an inverted emulsion of water in binder
- pore pressure-induced damage due to repeated traffic loading
- hydraulic scour at the surface due to tire-pavement interaction
- pH instability of the contact water that affects the binder-aggregate interface
- environmental factors such as excessive rainfall, large temperature fluctuations, and Freeze-Thaw (F/T) conditions

Adhesive bonding between the binder and aggregate results from the physical and chemical interaction between the two component materials and the non-uniform and opposite charge distributions on their surfaces. The most durable, tenacious, and moisture-resistant bonds with the aggregates are formed by different functional groups in the binder that are not necessarily strongly adsorbed on dry aggregates (Caro et al., 2008a). The binder-aggregate adhesive bond is also affected by aggregate mineralogy and corresponding surface charge and adsorbed cations on the aggregate surface, with clay particles degrading the adhesive bond (Tarrer and Wagh, 1991).

Recent research has focused on calculating adhesive bond strengths from measured surface energies of the component materials in both wet and dry conditions as part of a tiered approach to select compatible binder-aggregate combinations with adequate resistance to moisture damage (Howson et al., 2006). Physical properties of the aggregate (such as surface texture, porosity, shape, and gradation) and of the binder (such as viscosity and modification) are also important in terms of their effect on binder film thickness and wettability, with thicker films providing a physical barrier to moisture damage and lower viscosity providing deeper penetration into the aggregate surface and a stronger mechanical bond (Santucci, 2010). Anti stripping additives such as hydrated lime, Portland cement, and chemical liquid agents can be added to improve adhesive bond strengths between binders and aggregates, and polymer modifiers can also result in improved adhesion due to thicker binder films (Santucci, 2010). Thus, both favorable chemical bonding and the ability of the binder to wet and permeate the aggregate surface are required for favorable adhesive bonding between aggregates and binders.

Moisture damage is not limited to adhesive failure, but weakening of the cohesive strength of the mastic due to moisture infiltration is equally important. Some research suggests that the incorporation of anti stripping agents can also enhance mixture cohesion (Sebaaly et al., 2010). Lytton et al. (1993) used micromechanics to assess the relationship between the binder film thickness and the failure type due to moisture. They found that mixtures with thin binder films fail in tension by adhesive bond rupture while those with thicker binder films fail due to cohesive damage within the mastic.

Strategies to minimize moisture damage by either loss of adhesion or cohesion include incorporation of anti stripping agents and/or good construction practices. Commonly used anti stripping agents include hydrated lime and liquid anti stripping agents. Most liquid anti stripping additives are amine-based compounds that are usually added at a rate of 0.25 to 1.00% by weight of binder. They are designed to act as coupling agents to promote better adhesion at the binder-aggregate interface (Curtis et al., 1993). Other less common liquid anti stripping agents include silane-based additives that are added at a rate of 0.04 to 0.40% by weight of binder directly to the heated binder (Kim and Moore, 2009a; 2009b). Lime treatment is widely used throughout the United States by agencies to improve the moisture damage resistance of asphalt concrete paving materials. This anti stripping agent is generally added at a rate of 1.0 to 2.0% by weight of dry aggregate or 20 to 40% by weight of binder. Dry lime can be added to

dry or damp aggregate (with or without marination), and lime slurry marination is a fourth effective method of adding this anti stripping agent (Santucci, 2010). Related research has verified that lime treatment helps reduce the probability of moisture damage by loss of adhesion or cohesion and stiffen the mixture, therefore extending pavement life (Sebaaly et al., 2010)

In addition to the use of anti stripping agents, good construction practices can also alleviate moisture damage issues. The primary factor is good compaction to high density (93 to 96% maximum theoretical density) to reduce the access of moisture into the mixture and minimize other types of distress that are exacerbated by the effects of moisture (Santucci, 2010). Another practice aimed at reducing the presence of moisture is to ensure complete drying of aggregates. Finally, construction should not result in trapped moisture such as placement of a drainage layer over a distressed pavement or between two dense layers or placement of an impermeable surface treatment over a moisture-susceptible mixture.

2.2.2 Laboratory Characterization

To evaluate the moisture susceptibility of specific binder-aggregate combinations and the effectiveness of minimization strategies including the use of anti stripping agents and good construction practices, laboratory testing is utilized in the mix design stage on LMLC specimens or as a forensic tool on PMFC cores. As part of these laboratory testing methods, laboratory moisture-conditioning protocols are followed before testing mixtures in a wet or conditioned state. In some cases, the measured properties are compared with corresponding properties measured in a dry or unconditioned state. An ideal laboratory moisture-conditioning protocol should accelerate the penetration of moisture through the binder film and, at the same time, minimize complicating effects such as damaging the structure of the mixture.

The laboratory approaches shown in Table 2.2 with their corresponding test methods include boiling the mixture in a loose state or utilizing Ultrasonic Accelerated Moisture Conditioning (UAMC), vacuum saturation of the compacted mixture, soaking the compacted mixture in a hot water bath, freezing the mixture, and cycling pore water pressure to more closely simulate field conditions. Other approaches apply repetitive loading in the presence of moisture. These conditioning protocols and associated laboratory tests for assessing mixture susceptibility to moisture damage are described in this section based on categorization of the representative tests as shown in Table 2.2 as: (1) tests on loose uncompacted mixtures or component materials, (2)

tests that mechanically measure stiffness or tensile strength of laboratory-compacted specimens or field cores before and after moisture conditioning to simulate field conditions, and (3) tests that utilize repetitive loading of compacted mixtures in the presence of water.

Table 2.2. Laboratory Tests for Characterizing Moisture Susceptibility

Category	Tests & Standards	Moisture Conditioning	Output
Uncompacted Loose Mixtures or Component Materials	Boiling Water Test ASTM D3652	Boiling water, 10 min	Level of stripping by visual rating
	Ultrasonic Accelerated Moisture Conditioning (UAMC)	Ultrasonic conditioning in 140°F (60°C) water bath, 5hr	Mass loss
	Net Adsorption Test (NAT)	Wet condition (presence of water)	Amount of asphalt remaining on the aggregate surface after desorption
	Surface Free Energy (SFE)	Wet condition (calculated)	Conditioned to unconditioned adhesive bond strength ratio
	Bitumen Bond Strength (BBS)	Wet condition (presence of water) or conditioned specimens	Maximum pullout tensile force
Comparison of Conditioned and Unconditioned Mixtures	Modified Lottman Test AASHTO T283	Partial vacuum saturation, 1 optional F/T cycle, and 140°F (60°C) water bath	Conditioned Indirect Tensile Strength (ITS), unconditioned ITS, conditioned to unconditioned tensile strength ratio (TSR)
	Immersion-Compression Test AASHTO T165	140°F (60°C) water bath	Conditioned and unconditioned compressive strength ratio
	Energy Ratio (ER)	Vacuum saturation and cyclic pore pressure with hot water	Dissipated creep strain energy - DCSE
	E*/ECS AASHTO TP 62 AASHTO TP 34	Environmental Conditioning System (ECS)	Conditioned to unconditioned E* stiffness ratio (ESR)
	Resilient Modulus ASTM D4123	Partial vacuum saturation, optional F/T cycle, and hot water bath	Conditioned M_R, unconditioned M_R, conditioned to unconditioned M_R ratio
	Dynamic Mechanical Analyzer (DMA)	Partial vacuum saturation, 1hr	Conditioned to unconditioned crack growth index ratio at 10,000 cycles
Repetitive Loading in the Presence of Water	Hamburg Wheel-Tracking Test (HWTT) AASHTO T324	122°F (50°C) water bath	Rut depth at 20,000 load cycles and Stripping Inflection Point (SIP)
	Asphalt Pavement Analyzer (APA) AASHTO TP 63	Partial vacuum saturation, 1 F/T cycle, 140°F (60°C) water bath and testing water bath at PG high temperature	Conditioned to unconditioned rut depth ratio
	Model Mobile Load Simulator 3 (MMLS3)	140°F (60°C) water bath	Visual stripping evaluation, conditioned to unconditioned rut depth ratio, and conditioned to unconditioned TSR
	Moisture Induced Stress Tester (MIST)	Unsaturated specimen with water at 140°F (60°C) under compressed air and vacuum cycles	Visual stripping evaluation, change in bulk specific gravity, and conditioned to unconditioned TSR

The relationship with field performance is the ultimate test of laboratory characterization methods for identifying moisture-susceptible asphalt concrete paving mixtures and the effectiveness of materials (binder-aggregate compatibility and/or anti stripping agents) and methods (increased density) to combat the deteriorative effects of moisture. Table 2.2 is not exhaustive, but the representative list captures the commonly used and currently available tests that were adopted as national standards and recently developed and promising methods. Solaimanian et al. (2003) and Santucci (2010) provide a more extensive list that includes less commonly used and older test methods.

2.2.2.1 Uncompacted Loose Mixtures or Component Materials

The Boiling Water Test (ASTM D3625) is a national test standard from the first category that visually evaluates a loose mixture after boiling in water for ten minutes. Numerous prior research studies have indicated that the Boiling Water Test is not an ideal test method since the results are subjective, but some reasonable correlation to field performance has been shown for Alabama mixtures (Parker and Wilson, 1986). Additionally, the Boiling Water Test does not take into consideration the void structure, permeability, or gradation of the asphalt concrete mixtures (Aschenbrener et al., 1995). As a result, the Boiling Water Test is not a popular test method.

More recent developments include the use of ultrasonic energy to more quantitatively assess displacement and detachment of the binder from the aggregate (McCann and Sebaaly, 2001; McCann et al., 2006). In applying UAMC, ultrasonic energy is utilized on a loose mixture in a 140°F (60°C) water bath and the loss of weight through a fine sieve [No. 16 (1.18mm)] over a five-hour period is continuously monitored. The rate of material lost is a repeatable result that quantifies mixture moisture susceptibility: larger rates are associated with a greater susceptibility to moisture damage. The UAMC method distinguished between different aggregate type, binder types and contents, and the use of lime as an anti stripping agent (McCann and Sebaaly, 2001). In addition, this testing method was correlated with retained TSR, described in the next section, after a single F/T cycle and with the decay rate of the TSR after multiple F/T cycles (McCann et al., 2006).

More theoretical approaches involve evaluating the adsorption characteristics of different aggregate-binder combinations with and without the presence of water. The Net Adsorption Test (NAT) was developed during the Strategic Highway Research Program (SHRP) to assess

moisture susceptibility of asphalt concrete mixtures (Curtis et al., 1991, Curtis et al., 1993; Perry and Curtis, 1993). This test is based on adsorption of a binder of varying concentrations in solutions of toluene onto an aggregate surface and subsequent desorption of the binder from the aggregate surface in the presence of water. A specific fraction of fine aggregate is used in the NAT, and initial and net adsorptions are reported. Adequate correlation to field performance has been shown for Minnesota mixtures (Stroup-Gardiner et al., 1995).

More recent advances utilize a calculated energy ratio of the strength of the adhesive bond between a specific binder and a specific aggregate with and without water present based on surface energies of the binder and aggregate measured separately (Howson et al., 2007). This calculation is part of the first tier in a three-tiered approach to select binder and aggregate components with adequate moisture susceptibility. Successful correlation to field performance has been shown for multiple mixtures from multiple states, and limits on the ratio have been suggested (Bhasin et al., 2006).

To assess more directly the adhesive bond between specific binders and aggregates, the Bitumen Bond Strength (BBS) Test was recently developed as a draft AASHTO standard and submitted for review during the 2009 Federal Highway Administration (FHWA) Emulsion Task Force meeting in Scottsdale, Arizona. This test utilizes the Pneumatic Adhesion Tensile Testing Instrument (PATTI) to pull an aggregate stub off a solid substrate coated with binder (Kanitpong and Bahia, 2003; Youtcheff and Aurilio, 1997, Copeland et al., 2006). This test successfully showed increased adhesion with anti stripping additives, especially in the presence of water, that correlated with improved overall pavement performance and specific distresses associated with moisture damage (raveling and rutting) (Kanitpong and Bahia, 2008). This test was also utilized with a new moisture conditioning process for binders to test the binder-aggregate combination wet to evaluate adhesion after soaking binder-coated substrate in a heated water bath at 147°F (64°C) overnight and compare with the result from testing dry to evaluate cohesion (Wasiuddin et al., 2011). An additional test to measure cohesion of the mastic as a function of moisture and allow for comparison of the adhesive (through BBS testing) and cohesive causes of failure predicted for specific binder-aggregate combinations was also recently developed (Kringos et al., 2011).

These test methods that provide a qualitative or quantitative measure of the compatibility or stripping potential of a specific binder-aggregate combination are best used to screen these combinations and preclude them from use in the field. They are generally simpler and less costly than those described subsequently that include measurement of mixture properties. However, as Solaimanian et al. (2003) and Aschenbrener et al. (1995) suggested, they do not account for mixture mechanical behavior, the effects of traffic, or the internal structure of the mixture.

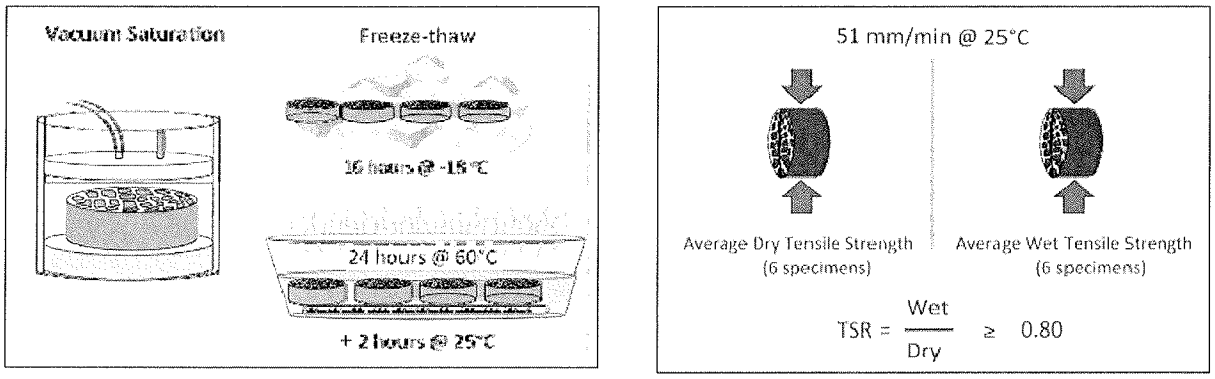
2.2.2.2 Comparison of Conditioned and Unconditioned Mixtures

Compared to more subjective, qualitative, and pass-fail tests on uncompacted loose mixtures or component materials, quantitative tests provide a more objective ratio of a measured mechanical parameter (stiffness or strength) after moisture conditioning to that in an unconditioned or dry state.

The most common national test standard in the second category is the Modified Lottman Test (AASHTO T283) included in AASHTO M323 as part of the Superpave volumetric mix design method and a similar ASTM method (ASTM D4867, Asphalt Institute 2001). These methods utilize a value of 0.7 or 0.8 (Superpave criteria) for the retained indirect TSR measured at 77°F (25°C) and 2in/min (50mm/min) after moisture conditioning. The moisture-conditioning protocol consists of partial vacuum saturation (70 to 80%), an optional F/T cycle for 16 hours at 0°F (-18°C), and soaking for 24 hours at 140°F (60°C) before bringing the conditioned specimens to the test temperature of 77°F (25°C). A schematic of the AASHTO T283 moisture conditioning and testing protocol is presented in Figure 2.3. The specimen fabrication protocol also calls for loose mix conditioning/curing of 16 hours at 140°F (60°C) followed by 2 hours at 275°F (135°C) and a compacted AV content of 7%. The loose mix conditioning/curing and the use of a F/T cycle were recommended to transition from smaller specimens for which the test was originally developed to larger specimens compacted in the Superpave Gyratory Compactor (SGC) and utilized in the Superpave mix design (Epps et al., 2000).

Experience shows that this common test method is able to distinguish highly moisture-resistant mixtures from those that are extremely susceptible to moisture damage (Lottman, 1982; Aschenbrener, 1995; Scherocman et al., 1986; Sebaaly et al., 2010). Multiple mixtures in Colorado with large differences in performance were identified by differences in TSR measured on conditioned specimens with high saturation levels (Aschenbrener, 1995). Successful use of

this test to discriminate the effectiveness of different anti stripping agents has also been shown with multiple F/T cycles for mixtures from multiple states (Scherocman et al., 1985 and 1986). A more recent extensive laboratory testing program for the National Lime Association (NLA) determined TSR results for mixtures from multiple states with a range in performance and reliably predicted moisture susceptibility (Sebaaly et al., 2010).



(a) (b)

Figure 2.3. Modified Lottman Test AASHTO T 283: (a) Moisture Conditioning Sequence and (b) IDT Test (Santucci et al. 2010)

However, there are also some disadvantages to the Modified Lottman Test, including a lack of correlation with field performance for mixtures that are marginally moisture susceptible, especially when the AV content is less than 6%, and a large variability in test results (Epps et al., 2000; Aschenbrener et al., 1995; Stuart, 1998; Kanitpong and Bahia, 2008). Improvements to the anti stripping treatment processes were suggested to reduce variability (Solaimanian and Kennedy, 2002). Higher limits for the TSR (85%) were suggested to preclude the exacerbation of both rutting and fatigue cracking (Lottman, 1990). Increasing the TSR threshold to 87% and increasing the number of F/T cycles to three were also suggested for larger SGC specimens (Bausano et al., 2006).

A recent study to develop a precision and bias statement for this test method identified large variability in the test results when comparing two different compaction methods and two different aggregate types (a moisture-resistant limestone and a moisture-susceptible sandstone) and their corresponding field performance (Azari, 2010). The reasons for the associated variability were also evaluated using analysis of X-ray Computed Tomography (CT) images and finite element simulation of the moisture infiltration process. Results indicated that moisture infiltrates to the center of smaller specimens compacted with the Marshall hammer faster than to

that of larger SGC specimens due to differences in air void size and distribution or internal structure. In addition, the lack of correspondence between laboratory and field results was attributed to the fact that the laboratory moisture-conditioning protocol was not representative of the moisture damage time frame that occurred in the field. This moisture-conditioning protocol and resulting micro cracking due to the vacuum saturation process may also contribute to the variability of the test results. Other related research demonstrated that there are likely differences in the long-term damage processes in the field and damage processes in the laboratory caused by volumetric expansion when water turns to ice and embrittlement of materials that could also contribute to the variability in the test results (Kringos et al., 2009).

The Immersion-Compression Test (AASHTO T165/ASTM D1075) is another national test standard that employs a value of 0.75 for the index of retained stability (or ratio of direct compressive strength) after static soaking of compacted specimens in heated water to that without moisture conditioning. This is no longer a popular method for characterizing moisture susceptibility because tensile properties (stiffness or strength) are likely more related or sensitive to the adhesive and cohesive causes of failure due to moisture damage.

The Environmental Conditioning System (ECS) was developed during SHRP to more realistically simulate field conditions using repeated hydraulic loading and repeated load cycles. This ECS system was utilized with a retained resilient modulus ratio (ECS-M_R ratio) with and without multiple moisture conditioning cycles (vacuum saturation, hot water, and optional freeze cycle) (AASHTO TP34) (Terrel and Al-Swailmi, 1994). This non-destructive test parameter was measured after each moisture-conditioning cycle, and specifications require a minimum retained resilient modulus of 70% of conditioned specimens to unconditioned specimens. Several modifications to the original ECS conditioning parameters and M_R measurement protocols have been made to provide a better correlation between test results and field performance (Aschenbrener et al., 1995; Alam et al., 1998).

More recently the ECS was evaluated for use with another non-destructive test, the compressive dynamic modulus (E*) test (AASHTO TP62), to again assess the effects of moisture conditioning on the same specimen (Solaimanian et al., 2006). The E* stiffness ratio (ESR) with and without ECS conditioning at a reported threshold between 75 and 80% showed good correlation to field performance for mixtures from multiple states, but further work to

simplify and shorten the testing protocol and/or add an evaluation of the effects of moisture and load separately and/or possibly continuously monitor mixture response during conditioning was recommended (Solaimanian et al., 2007). Nadkarni et al. (2009) also utilized the ESR but with and without AASHTO T283 moisture conditioning and recommended a minimum required retained ESR of 70% for conventional mixtures in Arizona. Bausano and Williams (2009) also utilized and recommended the ESR with AASHTO T283 moisture conditioning (with a minimum retained E* ratio of 60% based on an equivalent percentage of AASHTO T283 results below the 80% limit for different mixtures from Iowa), but they tested the conditioned specimens in a saturated condition. A recently completed extensive laboratory testing program for the NLA tracked E* with up to 15 multiple F/T cycles (AASHTO T283) for mixtures from multiple states with a range in performance and found that these results correlated with other moisture susceptibility results (Sebaaly et al., 2010).

The NLA study also utilized more advanced tools using the Dynamic Mechanical Analyzer (DMA) to characterize the effect of lime on the fine aggregate matrix (FAM) (with aggregate less than the No. 16 sieve (1.18mm), as recommended in a tiered approach for moisture susceptibility evaluation (Howson et al., 2007). For compatible binder-aggregate combinations identified in the first tier, the DMA is utilized in the second tier in a repeated loading test with and without moisture conditioning using a protocol developed by Kim et al. (2004) with vacuum saturation for 1 hr. The final tier consists of repeated direct tension tests on full-scale mixtures with and without moisture conditioning by the AASHTO T283 protocol without the F/T cycle. A criterion for a retained crack growth index was recommended for both the second and third tier (Howson et al., 2006). The DMA and associated analysis identified both good and poor field performance in multiple states, and the NLA study showed consistent results between the DMA G* results and E* with multiple F/T cycles (Lytton et al., 2005; Sebaaly et al. 2010).

More recent advances include development of the energy ratio (ER) by Birgisson et al. (2004; 2007). The ER quantifies the effects of moisture damage on the fracture resistance of mixtures measured using the Superpave Indirect Tension (IDT) Test (AASHTO T322) that includes both creep and strength testing. The ER is calculated from parameters including the Dissipated Creep Strain Energy (DCSE), a minimum DCSE threshold for adequate cracking performance, and creep and strength parameters. While originally developed using the moisture-conditioning protocol from AASHTO T283, a new protocol using application of cyclic pore

pressure is now included (Birgisson et al., 2007). This technique creates a more representative mechanism that accelerates both long-term moisture intrusion through the binder film and the effects of expansive water pressure while minimizing other confounding damage effects. Results indicated that the ER was capable of detecting moisture susceptibility in terms of the effects on mixture fracture resistance and detecting the positive contribution of anti stripping agents in improving moisture susceptibility.

2.2.2.3 Repetitive Loading in Presence of Water

Laboratory tests that utilize repetitive loading in the presence of water include wheel tracking tests that measure combined mixture resistance to moisture susceptibility and rutting. These tests are also more objective in nature, but may confound resistance to rutting and moisture susceptibility. Some states have switched to this type of test from a comparison of conditioned and unconditioned mixtures based on a significantly improved relationship with field performance and an ability to identify premature failures (Epps Martin et al., 2003).

In the last decade, some agencies have moved to the Hamburg Wheel-Tracking Test (HWTT) (AASHTO T324) to evaluate moisture susceptibility of asphalt concrete mixtures due to good repeatability and correlation with field performance (Epps Martin et al., 2003; Izzo and Tahmoressi, 1999). As shown in Figure 2.4, specimens are submerged in hot water and subjected to 50 passes of a steel wheel per minute. Each sample is loaded for a maximum of 20,000 passes or until 0.8in (20mm) of deformation occurs at a temperature of (113°F) 45°C or 122°F (50°C). Some states have different requirements. For example, Texas specifications require a rut depth of less than 0.5in (12.5mm) after 20,000 passes when a Performance Graded (PG) PG 76 binder or higher grade is used.

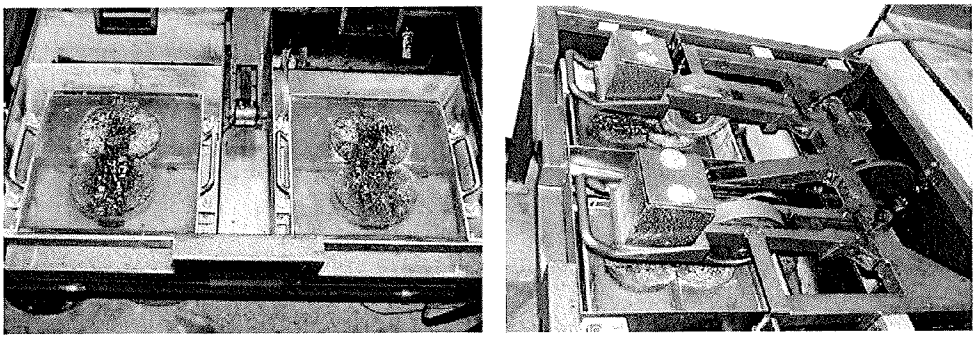


Figure 2.4. Submerged Specimens in the HWTT and HWTT Testing (Solaimanian et al., 2007)

Results from the test as shown in Figure 2.5 include creep slope, stripping slope, and SIP. Creep slope relates to rutting from plastic flow, the stripping slope represents the number of passes required to create 0.04in (1mm) deformation due to stripping (which relates to the severity of moisture damage), and the SIP is the number of passes at the intersection of the creep slope and the stripping slope (which relates to the resistance to moisture damage).

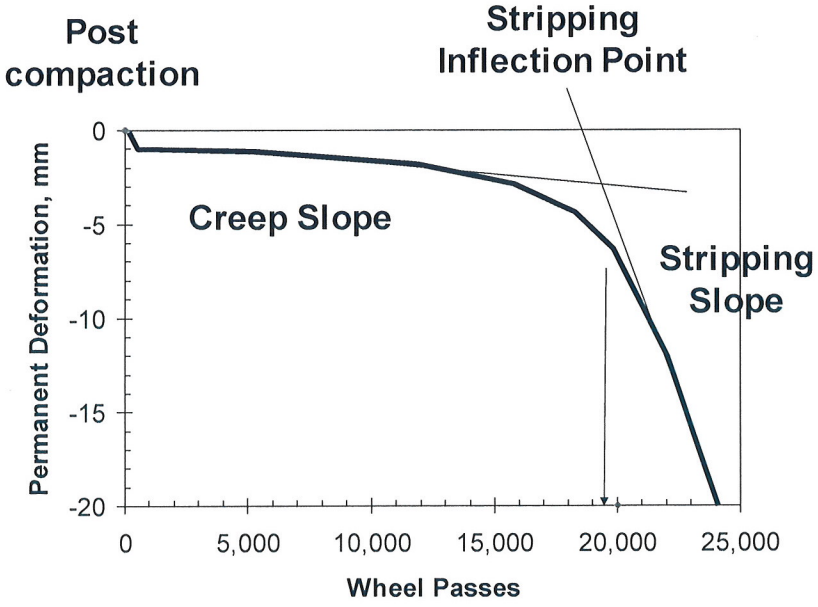


Figure 2.5. Typical HWTT Deformation Behavior with Load Cycles

Izzo and Tahmoressi (1999) evaluated the laboratory repeatability, testing configuration, test temperature, and capability to assess the effects of anti stripping additives. Their results showed that the test was capable of detecting the use of anti stripping additives, yielding improved performance in terms of moisture susceptibility. Several modifications were made to the original equipment, including the use of cylindrical specimens from the SGC. Recommended changes by Aschenbrener et al. (1995) to the rut depth threshold [from 0.16 to 0.40in (4 to 10mm)] under variable testing temperatures depending on the binder type resulted in improved correlation with field performance.

The Asphalt Pavement Analyzer (AASHTO TP63) utilizes a repeated loading test device on specimens in saturated conditions and compares them to unconditioned specimens tested dry. The test criteria is the ratio of conditioned rut depth to unconditioned rut depth, with values greater than 1 suggesting the mixture is moisture susceptible. Similar to the HWTT, the Asphalt

Pavement Analyzer (APA) allows for a maximum rut depth. Bausano et al. (2006) indicated that the APA testing of saturated mixtures is capable of identifying moisture susceptibility, simulating the repeated hydraulic loading that pavements undergo with desirable testing efficiency.

The Model Mobile Load Simulator 3 (MMLS3) can also be used to provide accelerated loading in the presence of water. This device applies traffic to mixture specimens in a hot-wet environment. A tire pressure of 100psi (690kPa), load of 607lbf (2.7 kN), and water temperature of 140°F (60°C) were used in previous research (Mallick et al., 2005). Specimens were put in a test bed that was then placed in a 140°F (60°C) water bath. An electronic profilometer recorded the rut depth of the specimens during the test process. Visual stripping was also considered for comparison of mixtures after loading. Results indicated that the MMLS3 is a promising moisture conditioning method and that after loading, cores extracted from trafficked specimens could be used to determine conditioned indirect tensile strength (ITS) (Mallick et al., 2005).

More recent development includes the Moisture Induced Stress Tester (MIST) that is designed to evaluate the resistance of an asphalt concrete mixture to stripping and moisture damage and to simulate more realistically the action of traffic on wet pavement (InstroTek, 2011a). This equipment replicates the condition of water being forced in and out of the pavement as tires roll over by cyclically applying and removing high pressure from an unsaturated compacted specimen fabricated to 7% AV. To further accelerate the potential damage from this action, the test is performed at an elevated temperature of 140°F (60°C). The change in bulk specific gravity of the sample measured using ASTM D2726 or AASHTO T166 is monitored to evaluate the loss of integrity due to moisture damage. MIST has a shorter test time as compared to other moisture sensitivity tests, and testing is automated. The TSR and visual inspection of the sample are also determined after conditioning to evaluate moisture susceptibility, and results are promising in providing good correlation with field performance (InstroTek, 2011b). Use of the MIST as a moisture-conditioning protocol prior to E* or Superpave IDT creep and strength tests has also shown promise in characterizing moisture susceptibility (Chen and Huang, 2008).

2.3 Previous Moisture Susceptibility Research on WMA

Despite the attractive economical, environmental, and safety advantages of WMA, a number of changes in the production process as compared to HMA have raised concerns regarding the

long-term performance of WMA pavements. Factors that can potentially increase the moisture susceptibility of WMA are described in this section. In addition, results from prior research studies regarding moisture susceptibility characterization of WMA, laboratory conditioning/curing, WMA time horizon, and comparison of specimen types are discussed.

2.3.1 Moisture Susceptibility Factors

Besides the moisture susceptibility mechanisms explained in the previous section, there are several factors related to the lower production temperature of the WMA and the use of certain foaming and additive technologies that can increase the moisture susceptibility of WMA. These factors include:

- introduction of additional moisture with the free water foaming WMA technologies
- use of wet/damp aggregates in the production process
- reduced binder absorption by the aggregates at lower production temperatures
- reduced binder-aggregate bond strength in the presence of certain WMA additives

While the first factor has not been addressed to date in previous research, the remaining factors have been investigated to some extent. Previous research on each of the last three factors is described in this section, and Table 2.3 provides a summary of these selected research studies.

Table 2.3. Studies on the Effect of WMA Technologies on Mixture Performance

WMA Technology	Test Method	Reference	Conclusions
Advera®	HWTT	Austerman et al., 2009	Based on SIP: - Mixtures are more moisture susceptible - More moisture susceptible vs. Sasobit
Aspha-min®	Adhesive Bond Strength	Wasiuddin et al., 2008	- No significant effect on SFE - No significant improvement in wettability - Increased adhesion for PG 70-28; no effect for PG 64-22
	HWTT	Hurley and Prowell, 2006	- Less rutting resistance vs. HMA control - Hydrated lime improves rutting resistance
	ITS/TSR	Hurley and Prowell, 2006	- Lower ITS at reduced aging and compaction temperatures - Improved ITS and TSR at higher short-term aging temperature - Lower TSR values vs. HMA control - TSR values < 80% - Hydrate lime reduced moisture susceptibility
Sasobit®	Adhesive Bond Strength	Wasiuddin et al., 2008 and Wasiuddin et al., 2011	- Increased wettability - Decrease in dry cohesive strength by ~25% - Decrease in binder-aggregate adhesive bond by ~30% - Reduced total SFE of the binder
	HWTT	Austerman et al., 2009	Based on SIP: - Mixtures are more moisture susceptible - Less moisture susceptible vs. Advera
		Hearon and Diefenderfer, 2008	Based on rut depth at 20,000 cycles: - No difference in rutting of WMA vs. HMA
		Hurley and Prowell, 2006	- Improved rutting resistance with limestone but not with granite aggregate -Anti stripping additive improves rutting resistance
		Mohammad et al., 2008	- No difference in rutting vs. control
	ITS/TSR	Hearon and Diefenderfer, 2008	- Improved TSR after long-term aging of the mixtures - TSR improved with higher mixing temp. - TSR > 80% in all cases where anti stripping additives were used
		Hurley and Prowell, 2006	- Lower ITS values vs. HMA control - TSR values < 80% - Improved TSR with use of anti stripping additive
		Mohammad et al., 2008	- Lower ITS values vs. HMA control - Statistical significant differences in ITS vs. HMA control for aged specimens - No significant differences for TSR

2.3.1.1 Use of Wet Aggregates

Aggregates used in WMA production may not dry completely due to the lower production temperatures, especially if absorptive aggregates are employed (Prowell et al., 2011). This

residual moisture in the mixture may disrupt the bond between the binder and the aggregate and increase the moisture susceptibility and stripping potential of the mixture. Even if the dwell time in the drum at lower temperatures is enough to dry the moisture on the aggregate surface, it may not be enough to dry the internal moisture, especially for the coarser aggregate fractions. Parker and West (1992) observed that the moisture content of coarse aggregates in stockpiles was always higher than the finer fractions and that the variation was highly dependent on the environmental conditions before and/or during production (i.e., rainy and cool days versus hot and dry days).

Researchers have studied the effect of residual aggregate moisture on the stripping potential of HMA. Their results show that the moisture susceptibility of HMA prepared with aggregates having varying levels of moisture content and measured in terms of the ITS and TSR depends on the aggregate type, with gravel and sandstone being more susceptible to the initial level of moisture content than limestone (Joslin et al., 1998; Parker and West, 1992). In addition, the initial moisture content of the aggregate during production had an effect on the correlation between observed field performance and test predictions in the laboratory (Parker and West, 1992).

In a study aimed at investigating the relationship between the tender behavior of HMA during compaction and initial moisture content in the aggregates, Huber et al. (2002) developed a method to introduce and measure moisture in laboratory specimens. They placed aggregates that were soaked overnight in a bucket mixer and mixed while heating using a propane tank with a nozzle attachment to simulate the burner flame at the production plant. The binder was added once the aggregate reached a temperature of 290°F (143°C) (measured with an infrared probe). The procedure was successful in trapping moisture, with residual moisture values in the mixture ranging from about 0.8 - 1.8%. The recommended method for measuring moisture content in the mixture was drying at constant mass using a forced draft oven at 230°F (110°C) (Huber et al., 2002). In a later study, Mallick et al. (2011) developed a laboratory method to simulate incomplete aggregate drying during HMA production. To prepare aggregate batches with various moisture contents, an oven-dried aggregate blend was vacuum-sealed in a bag, soaked with water introduced in the bag with a syringe, vacuum-sealed again, left soaking overnight, and oven-dried at 194°F (90°C) for different periods. After mixing at 194°F (90°C) using Sasobit® as a compaction aid, the mixture was short-term aged at the same temperature [i.e., 194°F (90°C)].

The compacted mixtures were placed in sealed bags to preserve the moisture content until testing.

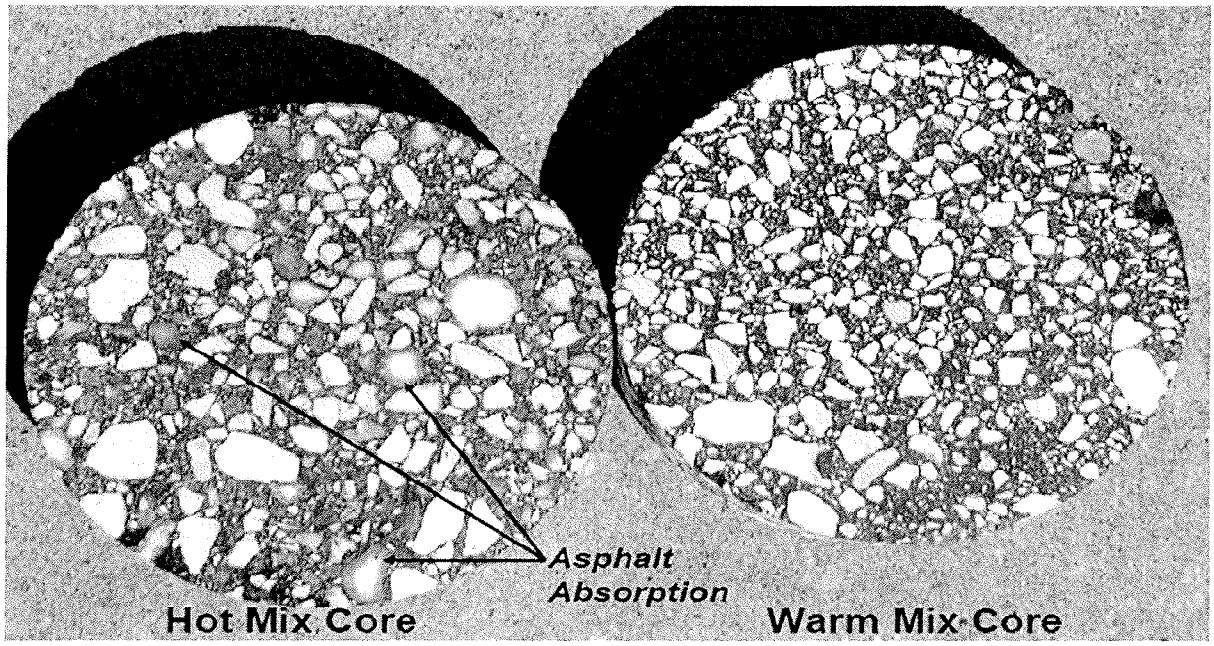
Similar work evaluating the effect of the initial aggregate moisture content in WMA performance has been done. Hurley and Prowell (2006) used the same bucket mixer and propane torch method developed by Huber et al. (2002) to prepare WMA mixtures with limestone and granite aggregates containing 3% moisture content beyond their absorption capacities and evaluated the effect on the moisture susceptibility of Aspha-min®, Sasobit®, and Evotherm™ technologies. The use of moist aggregates decreased the ITS in all cases versus the HMA control. The TSR values were dependent on the type of aggregate and technology; the combination of granite and Evotherm™ yielded acceptable values of TSR (i.e., 96%) as did limestone and Sasobit® with a TSR of 91%. All other combinations were below the recommended threshold, ranging from 51 - 71%.

In a separate study, Bennert et al. (2011) proposed a modified mixing procedure to simulate incomplete aggregate drying during WMA production and measure the moisture susceptibility of the mixture. The procedure consisted of pre-wetting the aggregate blend for 24 hours, mixing the aggregate while heating with a propane torch until the mixing temperature was reached (measured with an infrared probe), adding the pre-heated binder, mixing until the aggregates were fully coated, and conditioning for 2 hours at the compaction temperature. The aggregates were pre-wet at 3 and 6% moisture content and a blend of dry aggregates was also used. Highly absorptive (i.e., gravel) and low absorptive aggregates (i.e., trap rock) were used to prepare the aggregate blends. The moisture susceptibility was measured using ITS, TSR, and HWTT. The TSR results showed that all mixtures with the exception of the ones prepared at a higher mixing temperature [315°F (157°C)] with the dry aggregate blend failed to meet the recommended TSR threshold of 80%. The reported TSR values for the mixtures prepared with the moist aggregates ranged from 38.7 - 71.5%, while the TSR range for the mixtures prepared with the dry aggregate blends was 62.6 - 93.9%. Also, for both aggregate types, the lower mixing temperature [270°F (132°C)] and increased moisture content had a significant negative effect on both the unconditioned and conditioned ITS. For the HWTT, only the WMA mixtures prepared at the lower mixing temperature and 6% moisture content exceeded the rutting threshold of 0.5in (12.5mm) at 20,000 passes. The rutting values for all other mixtures at that load level varied between 0.27 – 0.46in (6.89 - 11.62mm).

Xiao et al. (2009) also investigated the influence of initial aggregate moisture content on the moisture susceptibility of WMA. They prepared mixtures using three aggregate sources, dry and moist aggregate blends (0.5% moisture content), two WMA additives, and various quantities of hydrated lime and evaluated the performance of the mixture with ITS, TSR, deformation, and toughness. The 0.5% aggregate moisture content was achieved by drying aggregate blends soaked with specific amounts of water at 349°F (176°C) for 140 min. The ITS values of all unconditioned mixtures prepared with the moist aggregates were lower compared to their dry counterparts and highly dependent on the aggregate source. The addition of hydrated lime in the unconditioned mixtures where moist aggregates were used provided no significant improvement in the ITS value. However, for the mixtures subjected to moisture conditioning (prepared both with dry and moist aggregates), the addition of hydrated lime was beneficial in increasing the ITS value. With regard to the TSR results, only the mixtures containing hydrated lime (prepared both with dry and moist aggregates) were above the 85% threshold established by the agency. A follow-up study confirmed that the moisture susceptibility of the WMA is dependent on the aggregate type and that the addition of hydrated lime improves the resistance to moisture damage (Xiao et al., 2011).

2.3.1.2 Reduced Binder Absorption

Another mechanism that can potentially increase the moisture susceptibility of WMA is the reduced absorption of binder by the aggregates that occurs at lower production temperatures. This lower binder absorption can weaken the bond between the two materials and ultimately render the mixture less resistant to moisture. Figure 2.6 shows the difference in binder absorption by the aggregate in HMA and WMA field cores obtained from Loop 368 in San Antonio, Texas, after 1 year in service. The binder absorption in the HMA field core is evident in Figure 2.6(a), which implies that the lighter and less polar fractions of the binder were absorbed by the aggregate, leaving behind the thicker more polar fraction of the binder to coat the aggregate. In contrast, binder absorption is not visible in the WMA core [Figure 2.6(b)], which means that a lower viscosity coating surrounds the aggregate, providing less protection from moisture.



(a) (b)

Figure 2.6. HMA and WMA Field Cores (Estakhri et al., 2010)

2.3.1.3 Reduced Binder-Aggregate Bond Strength

A third mechanism to take into consideration when evaluating the moisture susceptibility in WMA is the effect that certain additives used in WMA production may have on the binder-aggregate bond strength. The additives used in some of the WMA technologies have an impact on binder rheology, which in turn may weaken the adhesive bond between the binder and the aggregate, especially in the presence of water. The Surface Free Energy (SFE) and work of adhesion are commonly used to estimate the binder-aggregate bond strength. In a recent research study, the work of adhesion was calculated for combinations of two types of aggregates, two types of binders, two binder sources, and three WMA additives: Evotherm™, Sasobit®, and Rediset™ WMX (Estakhri et al., 2010). The results showed that in a dry condition, the work of adhesion between the aggregates and the binder plus additives decreased with respect to the control case (i.e., virgin binder-aggregate combination); that is, the binder-aggregate bond was weakened when the additives were included. In addition, in the presence of water, the values obtained for the work of adhesion were negative, which means that de-bonding between the two materials is likely to occur when water is present. Also, higher negative work of adhesion magnitudes were obtained for the cases where the binder was combined with the additives, which implies that debonding could be a more likely occurrence.

In another study, SFE was also used to evaluate the adhesive bond strength between two types of aggregates and two types of binders prepared with various levels of Sasobit® (Wasiuddin et al. 2008). The results showed that although the additive promoted better aggregate wettability, it had a negative effect on the binder-aggregate bond (regardless of the quantity of additive in the binder). This effect was more critical when PG 70-28 was used versus PG 64-22.

The adhesive bond strength has also been measured with the BBS test using PG 64-22 binder specimens prepared with Advera®, Sasobit®, Evotherm™ and applied on top of a limestone surface (Mogawer et al. 2010). The tests were performed at room temperature [i.e., 68°F (20°C)] on unconditioned specimens and moisture-conditioned specimens soaked for 24 hours at 104°F (40°C). The pull-off tensile strength results showed that the unconditioned values were equal to or higher for the WMA binders versus the HMA control. The conditioned tensile strengths for the WMA binder specimens, although lower than their dry counterparts, were also similar to the unconditioned HMA tensile strength values. The unconditioned to conditioned tensile strength percent drop was between 5 - 10% higher for WMA versus HMA.

Evaluations of various WMA technologies based on mixture performance usually indicate increased moisture susceptibility for WMA versus HMA, and mixed conclusions with regard to rutting. For example, the durability of WMA measured via the HWTT on mixtures prepared with two different levels of Sasobit® (1.5 and 3%) showed that the SIP of the mixtures containing the additive occurred earlier (at a lower number of passes) than the control mixture prepared with no additive (Austerman et al., 2009). Conversely, a separate study on Sasobit® using the HWTT indicated no significant differences in the rutting performance of WMA versus HMA with values [i.e., 0.08 - 0.10in (2.11 - 2.44mm)] well below the rutting limit required by the test method (Hearon and Diefenderfer, 2008). Table 2.3 provides a summary of selected research studies on the effect of different WMA technologies on mixture performance.

2.3.2 Laboratory Conditioning/Curing

To simulate the binder absorption and aging that occurs during construction, the standard practice for laboratory mixture design is to short-term oven age (STOA) or condition/cure the loose mixture prior to compaction. For HMA, the recommended time for performance testing (i.e., moisture susceptibility and rutting) is 4 hours at 275°F (135°C); for mixture design, when aggregate absorption is less than 4%, the conditioning/curing time can be reduced to 2 hours

(AASHTO R30). As part of recently completed NCHRP Project 9-43, the recommended conditioning/curing protocol for WMA was 2 hours at the compaction temperature (Bonaquist, 2010). This conclusion was obtained after testing WMA laboratory mixtures conditioned/cured at 2 and 4 hours and observing that the maximum specific gravity was practically the same for both conditions and that the tensile strength of laboratory mixtures after the 2-hour conditioning/curing period was equivalent to the tensile strength of PMLC specimens. However, when rutting and moisture susceptibility tests were conducted on the 2-hour conditioned/cured mixtures, the majority failed the established performance criteria, which were developed for HMA mixtures conditioned/cured for 4 hours at 275°F (135°C). Therefore, to apply the same performance testing criteria to WMA mixtures, a 2-step conditioning/curing process was recommended. The first step is the 2 hour conditioning/curing at the compaction temperature, and the second step is conditioning/curing at a high in-service pavement temperature [e.g., 140°F (60°C)] for a long enough period to reach a mixture stiffness equivalent to the stiffness of the HMA after 4 hours at 275°F (135°C) (Bonaquist, 2010). The time period for the second step was recommended not to exceed 16 hours based on results from NCHRP Project 9-13 which showed that the stiffness of HMA mixtures conditioned/cured at 16 hours at 140°F (60°C) exceeded the values of equivalent mixtures conditioned/cured for 4 hours at 275°F (135°C) (Epps et al., 2000).

In general, the majority of the studies that have been performed to understand the effect of conditioning/curing on the performance of WMA have concluded that an increase in conditioning/curing time and/or temperature reduces the difference in performance between HMA and WMA. Mogawer et al. (2011) measured the effect of conditioning/curing temperature and time on WMA prepared with Advera®, Sasobit®, SonneWarmix, and Evotherm™ at three conditioning/curing temperatures [i.e., 235°F (113°C), 265°F (129°C), and 295°F (146°C)] and three conditioning/curing periods (i.e., 2, 4 and 8 hours). The performance comparison was based on HWTT results, E* stiffness ratio (ESR), bond energy ratio, and fracture characteristics. The HWTT test results showed that conditioning/curing time and temperature had a significant effect on the moisture susceptibility of the mixtures. All WMA mixtures subjected to the highest and longest conditioning/curing temperature/time reached 20,000 passes without showing a Stripping Inflection Point (SIP), whereas all of the WMA mixtures conditioned at the lowest temperature of 235°F (113°C) reached the SIP before 20,000 passes. The fracture characteristics based on the DMA also showed that the difference in performance decreased as the conditioning/curing time

increased from 4 to 8 hours. With regard to stiffness, however, opposite trends were observed. The ratio of moisture conditioned to unconditioned E* measurements (ESR) showed that at the lower conditioning/curing temperature [i.e., 235°F (113°C)] for both 4 and 8 hours the WMA mixtures had higher ESR values than the HMA control and all the moisture conditioned E* values for WMA were higher than those for HMA. Conversely, at the higher conditioning/curing temperature of 265°F (129°C) the control HMA had higher values of ESR versus WMA. Thus, the stiffness of the WMA mixtures was more moisture susceptible when the mixtures were conditioned/cured at a higher temperature. At the higher conditioning/curing temperature, the moisture-conditioned E* values were lower for WMA than for HMA (except for Sasobit®) and the unconditioned E* values were higher for WMA versus HMA (except for Evotherm™). The authors recommended standardizing the conditioning/curing time of the WMA mixtures in the laboratory since it had a significant effect on the moisture damage performance of the mixtures (Mogawer et al., 2011).

A recent study by Estakhri et al. (2010) evaluated the effect of three conditioning/curing conditions [2 hours at 220°F (104°C)/250°F (121°C) for WMA/HMA, 2 hours at 275°F (135°C), and 4 hours at 275°F (135°C)] on HWTT results for WMA mixtures prepared with Evotherm™. In addition, WMA mixtures prepared with Advera®, Sasobit® and Evotherm™ conditioned/cured under two different conditions [2 hours at 220°F (104°C)/250°F (121°C) for WMA/HMA and 4 hours at 275°F (135°C)] were compared. The results for Evotherm™ showed that the number of passes to a 0.5in (12.5mm) rut depth rose with increasing conditioning/curing temperature and time, and that the mixture conditioned/cured for 4 hours at 275°F (135°C) showed equivalent performance to the control HMA conditioned/cured at 250°F (121°C). The HMA showed only a slight decrease in the number of passes to a 0.5in (12.5mm) rut depth when conditioned/cured at 250°F (121°C) versus 275°F (135°C). However, the change for the WMA mixtures prepared with the different technologies was significant for the two conditioning/curing temperatures. The number of passes for the Advera®, Sasobit®, and Evotherm™ mixtures was similar when conditioned/cured at 220°F (104°C), yet much higher when conditioned/cured at 275°F (135°C). The highest increase in the number of passes was for the Sasobit® technology. Based on these observations, a recommendation to condition/cure WMA for 4 hours at 275°F (135°C) was made (Estakhri et al., 2010).

Another study looked at the effect on compactability, HWTT rutting, and density of different conditioning/curing periods (i.e., 2, 4, 8, 20, and 32 hours) and temperatures [i.e., 250°F (121°C) and 275°F (135°C)] using plant mix prepared with Aquablack™ WMA technology (Estakhri, 2011). The number of gyrations to achieve the target density increased with conditioning/curing time and temperature, but it was always higher for HMA versus WMA (Figure 2.7). With regard to HWTT results, the rut depths decreased with conditioning/curing temperature and time up to about 8 hours. In addition, the rut depths for conditioning/curing times of 8, 20, and 32 hours showed no significant differences for both conditioning/curing temperatures (Figure 2.8). The density of the mixtures was higher with WMA versus HMA, but this difference reduced as the conditioning/curing time and temperature increased. The difference in densities may also be reduced by lower the conditioning/curing time and temperatures. A separate project that studied the effect of conditioning/curing on WMA prepared with Evotherm™ showed no significant differences in the density of WMA versus HMA that were unconditioned/uncured, conditioned/cured for 2 hours at 200°F (93°C), and 240°F (116°C) (Estakhri et al., 2009).

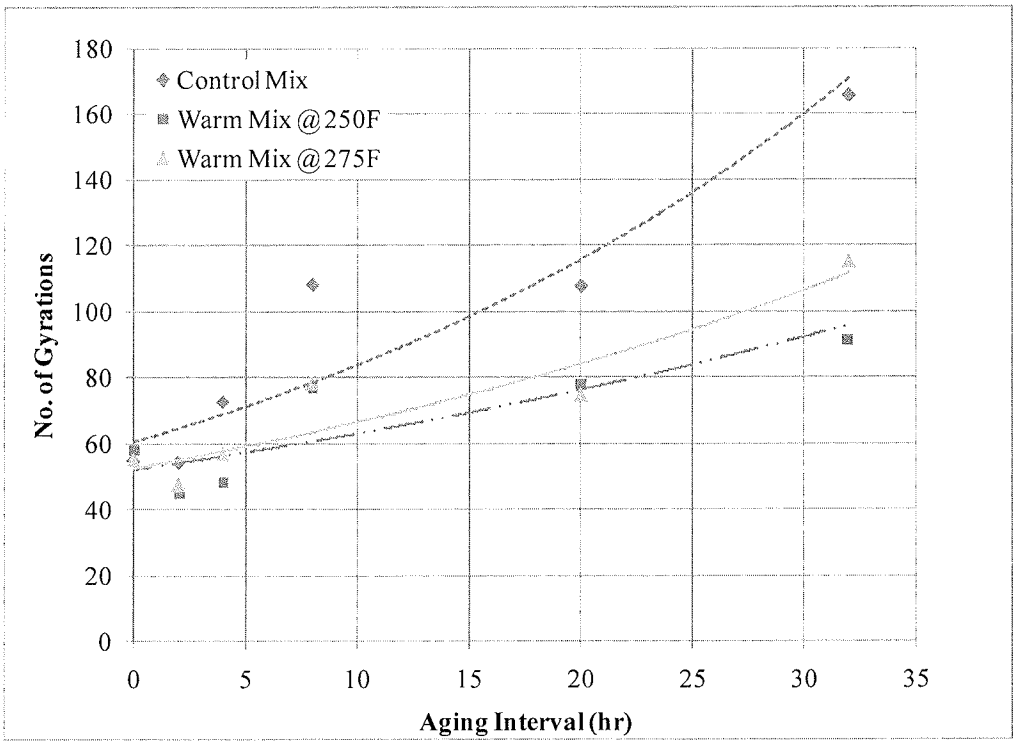
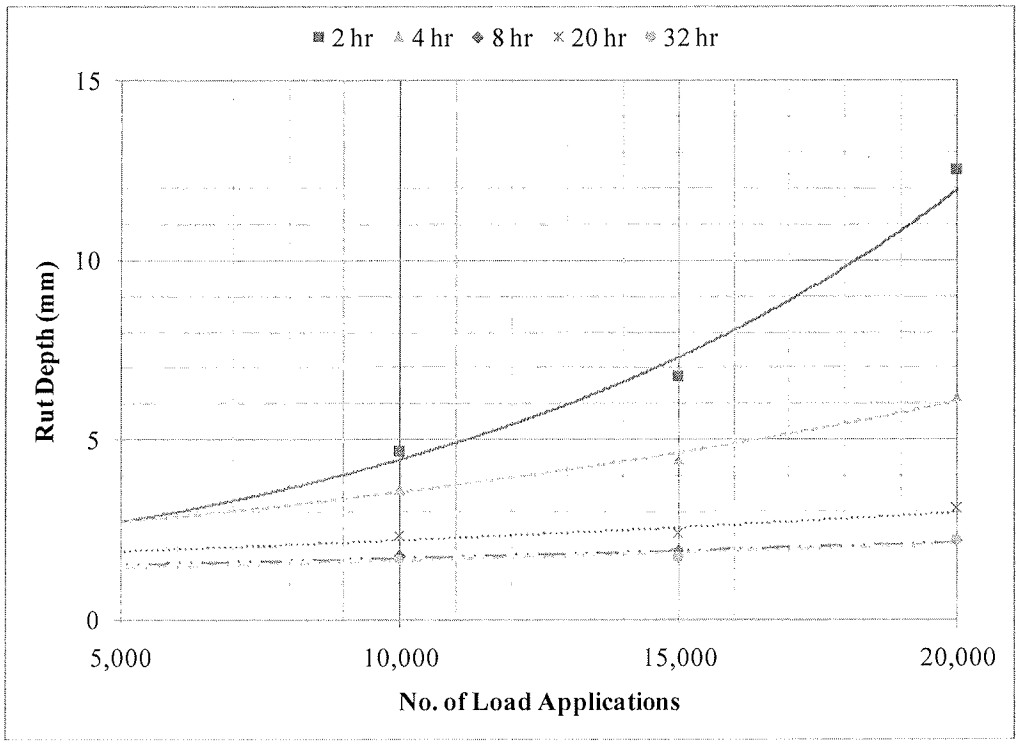
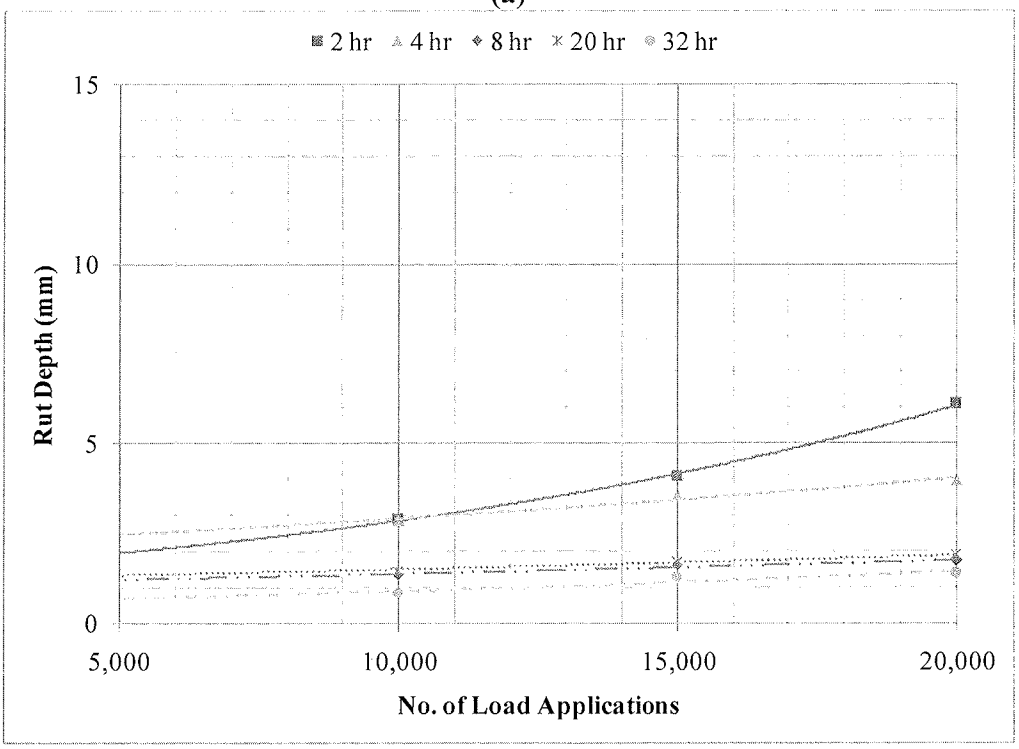


Figure 2.7. Number of Gyrations Required to Achieve Target Density (Estakhri, 2011)



(a)



(b)

Figure 2.8. Effect of Curing Time on Rut Depth for WMA Cured at (a) 250°F (121°C) and (b) 275°F (135°C) (Estakhri, 2011)

2.3.3 *Time Horizon*

From the conditioning/curing studies, it is apparent that the initial stiffness of the WMA is less than the stiffness of conventional HMA. This gap is reduced with increased laboratory conditioning/curing or elapsed time in the field. Therefore, a handful of researchers have tried to quantify this time horizon or long-term aging of the WMA in an effort to understand the difference between HMA and WMA and its impact on performance. More importantly, it is relevant to determine when the properties of the two types of mixtures converge. This is particularly significant when evaluating moisture susceptibility, which can occur early in the life of the pavement or after several years in-service, depending on environmental and loading conditions.

In a recent research study aimed at identifying moisture conditioning parameters that caused variability in the Modified Lottman (i.e., AASHTO T283) test results and lack of agreement between the laboratory test results and observed field performance, the time horizon effect was recognized as a crucial factor in understanding and describing these discrepancies (Kringos et al., 2009). Moisture infiltration in the field is a concentration-driven process and, as such, the time necessary for the moisture to activate in the components of the mixture and the weakening mechanism (e.g., pumping action) is most likely different from the saturation used in the laboratory procedure, which is based on pressure (i.e., vacuum saturation) and static loading. In addition, the time horizon was identified as the link between mixture physical properties (i.e., mastic film thickness) and environmental conditions (i.e., accessibility to moisture) that may indicate when moisture damage is more likely to occur in the field (Kringos et al., 2011).

For binders, the effect of aging time and temperature on performance was evaluated on short-term and long-term aged WMA binders at standard and low temperatures (Hanz et al., 2011). A PG 64-22 binder combined with Advera®, Rediset™, and a viscosity-reducing additive was evaluated. For the short-term aging, the Rolling Thin Film Oven (RTFO) procedure at the standard temperature of 325°F (163°C) and reduced temperatures of 220°F (104°C) and 250°F (121°C) was used. The performance evaluation was based on the continuous PG and non-recoverable creep compliance. The long-term aging was done using the Pressure Aging Vessel (PAV) at 220°F (104°C) and 325°F (163°C). The high temperature binder properties were significantly affected by the lower short-term aging temperature, causing a decrease equivalent

to one grade PG (i.e. 6°C) for the binders aged at 220°F (104°C). In contrast, the intermediate and low temperature binder properties were not significantly affected by the long-term aging in the PAV. From these observations, the study concluded that reduced binder aging had an adverse effect on the high temperature performance of the binder, especially during the early life of the pavement.

For mixtures, several researchers have also measured the effect of long-term aging (LTOA) on WMA. Mogawer et al. (2010) used WMA mixtures prepared using Advera® and SonneWarmix, conditioned/cured for 4 hours at 235°F (113°C), allowed to cool at room temperature for 6 hours, and then long-term aged for 14 hours at 140°F (60°C). The results of the HWTT were compared against WMA mixtures aged for 4 hours at 235°F (113°C), 265°F (129°C), and 295°F (146°C). For SonneWarmix, the number of passes to reach the SIP was lower (i.e., 4,200 passes) than for those with 4 hours conditioning/curing at 235°F (113°C) (i.e., 4,300 passes). For Advera®, the number of passes (i.e., 4,000 passes) was between the values obtained for those with conditioning/curing at 235°F (113°C) (i.e., 3,400 passes) and 265°F (129°C) (5,500 passes).

In another study, the effect of LTOA on WMA mixtures was assessed by oven aging the specimens in a forced-draft oven for 5 days at 185°F (85°C). Mixtures were prepared with Aspha-min®, Sasobit®, and Evotherm™ using two aggregate sources and various amounts of coal ash and shingles (Xiao et al., 2011). The ITS values of the aged and unaged specimens were very similar; however, the difference was significant for moisture-conditioned WMA specimens according to the South Carolina SC T70 procedure. The ITS values for the moisture-conditioned and aged WMA specimens were higher versus their unconditioned counterparts (except for Aspha-min®). The study concluded that the long-term aging process improved the moisture susceptibility of the WMA mixtures.

Another study also used LTOA in a forced-draft oven at 185°F (85°C) for 4 and 8 days to evaluate the TSR performance of WMA mixtures prepared with Sasobit® (Hearon and Diefenderfer, 2008). After the 4-day aging period, the TSR of the mixture produced at 302°F (150°C) did not improve, but the TSR values of the mixtures produced at lower temperatures of 266°F (130°C) and 230°F (110°C) did. The only mixture for which TSR did not improve from

the 4-day to the 8-day aging period was the one produced at 230°F (110°C). In general, the ITS values increased with longer aging time.

In the field, the effect of the time horizon has been evaluated using PMFC cores from WMA pavements after 1 month and 1 year in service (Estakhri et al., 2009). The HWTT rut depths were compared to those for PMLC specimens that were prepared at a compaction temperature of 240°F (116°C) and 300°F (149°C) [HMA was only compacted at 300°F (149°C)]. All the laboratory compacted WMA specimens failed the HWTT criteria of maximum rut depth of 0.5in (12.5mm) at 20,000 passes. The 1 month-old WMA cores showed no improvement in rut resistance, but the results of the 1 year-old WMA improved significantly, with rutting values similar to the ones obtained for the control HMA specimens. The ITS results showed a significant improvement of the 1 month-old cores [i.e., 160 psi (1,103 kPa)] versus the PMLC specimens [i.e., 60 psi (414 kPa)] with more than double the increase in strength. The ITS of the HMA control stayed constant after 1 year in service.

2.3.4 Laboratory Versus Field Specimens

As explained in the previous sections, conditioning/curing and long-term aging of WMA specimens have a significant effect on measured performance. This is particularly apparent when LMLC, PMLC, and PMFC specimens are compared, since they are often subjected to different conditions in the laboratory and the field. This phenomenon is not exclusive to WMA specimens; in a study by Parker and West (1992), PMLC HMA specimens prepared with limestone aggregates had higher strengths and TSR values than LMLC HMA specimens did. The authors noted that in the field, it was likely to have residual aggregate moisture; in the laboratory, the aggregates were dried completely. They hypothesized that the residual moisture could form a weaker but more moisture-resistant binder-aggregate adhesive bond versus an initially stronger but more moisture-susceptible bond formed between the binder and the dry aggregates.

In the WMA demonstration project built in Birmingham, Alabama, the performance of PMLC and LMLC specimens prepared with Evotherm™ DAT were compared using ITS and the HWTT (Kvasnak et al., 2009). The WMA specimens were also compared to the HMA control. LMLC specimens were prepared at 290°F (143°C) for HMA and 248°F (120°C) for WMA. Both HMA and WMA mixtures contained recycled asphalt pavement (RAP) and recycled asphalt shingles (RAS) and were compacted to 7% air voids and conditioned/cured for 2 hours after

mixing. The PMLC specimens were prepared from mixture obtained from the trucks and compacted right after sampling in the National Center for Asphalt Technology (NCAT) mobile laboratory (without reheating the mixture) with the compaction temperature and target air voids the same as that used for LMLC specimens. To evaluate moisture susceptibility, a subset of the specimens were conditioned using Alabama's moisture-conditioning protocol in ALDOT 361: partial vacuum saturation, 140°F (60°C) water bath for 24 hours, and 77°F (25°C) water bath for 1 hour before testing. The LMLC TSR results were lower for the WMA than for the control HMA, and all WMA specimens failed to meet the 80% TSR threshold. Conversely, all the PMLC TSR results for WMA (as well as the control HMA) were greater than 80%. The number of passes to the SIP for the PMLC specimens was higher for the control HMA versus the WMA, although in some cases the WMA exceeded 10,000 passes. Overall, with respect to moisture susceptibility, the PMLC WMA specimens performed better than the LMLC specimens did.

Ongoing NCHRP Project 9-48 is investigating the differences in volumetrics and mechanical properties of LMLC, PMLC, and PMFC specimens (Mohammad, 2010). Existing data sets published in the literature were used to investigate possible sources of variability between specimen types. These factors include compaction methods, silo storage time, baghouse fines, mixture reheating, aggregate absorption, plant type/settings, sampling location, gradation density, aggregate degradation, and aggregate moisture content. With respect to moisture sensitivity, preliminary conclusions based on the analysis of data collected during the original and rehabilitation construction cycles of the WesTrack study showed that the TSR of PMLC and PMFC specimens were lower than the LMLC specimens were. However, analysis of research data from the University of Nevada showed the opposite trend, with larger TSR values for PMLC versus LMLC specimens. When combining both datasets, the largest absolute average difference was for PMFC versus LMLC specimens followed by PMLC versus LMLC specimens.

2.4 WMA Field Experience

Many different contractors across the United States and Canada have used WMA in demonstration or trial projects. Selected examples of contractors' experiences as reported by NAPA and by the Canadian Technical Asphalt Association are listed in Table 2.4 (Aurilio and Michael, 2008; Forfylow and Middleton, 2008; Hughes et al., 2009; Johnston et al. 2008; Manolis et al., 2008, 2009; NAPA, 2008). The concerns and/or objectives of the trial pavements

and the lessons learned from the studies are summarized. These examples demonstrate that WMA is an effective and environmentally friendly material with tangible benefits and overall appropriate performance.

The majority of the agencies that have evaluated field performance of trial or routine WMA pavements indicate that they are in good condition and showing no signs of moisture damage (Brown, 2008; Diefenderfer and Hearon, 2010; Estakhri et al., 2010; Kim et al., 2010; Prowell et al. 2007, Prowell et al. 2011). Exceptions are one project located in Franklin, Tennesse (wet, no-freeze climate) and another located in Kimbolton, Ohio (wet, freeze climate), where the occurrence of raveling has been reported.

The project in Ohio is located on SR-541 and was built in 2006 using Evotherm™, Sasobit®, and Aspha-min®, along with a control HMA section. Each section was approximately 10 miles long. The materials used in the mixture included a PG 70-22 modified binder with 15% RAP and limestone aggregate. The compaction temperatures of the WMA ranged from 240 to 260°F (110 to 127°C). After periodic monitoring of the pavement, the Ohio DOT reported that the WMA sections were showing signs of raveling (i.e., loss of aggregate from the pavement surface). From the visual observations, the Evotherm™ section had the smallest degree of distress while the Sasobit showed the greatest. Factors contributing to the raveling were attributed to poor construction practices (i.e., mixture dragging under the screed and poor handwork) as well as the lower placement temperatures, especially for Sasobit® (Hurley et al., 2009).

The project in Tennessee is located on SR-46 and was built in 2007 using Advera® WMA, Astec DBG®, Evotherm™ DAT, and Sasobit® plus two control HMA sections. The WMA pavements were produced between 240 and 260°F (115 and 127°F) and placed at 230°F (110°C). Visual inspections were conducted after 1 year in service. During the assessment, it was noted that the control HMA pavements exhibited raveling as well as the Astec DBG® and the Evotherm™ DAT along the centerline of the pavement. The Sasobit was less affected by raveling. Raveling in the Advera® WMA pavement was more severe, especially in areas shaded by trees. Tests performed on extracted cores from the pavements indicate that the HMA control had the highest ITS values, while the Advera® WMA and the Sasobit® exhibited significantly lower ITS values (Kvasnak et al., 2010). Despite the occurrence of raveling in these two

pavements, the distress was neither severe nor extensive and the reports did not indicate that the functionality or ride quality of the pavement was impacted.

Table 2.4. Documented Trial WMA Pavements

Contractor	Location	Date	Technologies	Production Temperature	Compaction Temperature	Concerns/Objectives	Lessons Learned
Boggs Paving, Inc.	York Co., SC	October 2007	Astec DBG®	270°F (132°C)	190°F (88°C)	- Use high contents of RAP in WMA	- Up to 50% RAP was incorporated - Appropriate densification was achieved - Rutting performance adequate
Graniterock	Northern California: San Benito Co., Hwy 152, Santa Clara Co.	Spring 2006	Aspha-min®, Evotherm™, Sasobit®	260°F (127°C)	250°F (121°C)	- Effective production of WMA - Changes in the paving process - Service life of the WMA - Long-term benefits of WMA - Differences between WMA technologies - Use of rubberized asphalt in WMA	- WMA improved workability of rubberized asphalt - Technologies have their limits - Proper technology selection should be done depending on the characteristics of the job
Hubbard Construction Co.	Florida State Rd 417 and Orlando, FL	February 2006 and 2007	Aspha-min®, Astec DBG®	250-290°F (121-143°C)	225-260°F (107-127°C)	- Use of WMA with open-graded friction course - Drain-down potential	- Compaction window was extended - Workability was improved - Less measured drain-down with WMA - No shoving was observed soon after opening to traffic - Starting paving at a higher temperature and gradually reduce it to find optimum
Lehman-Roberts Co.	Memphis, TN	December 2007, April 2008	Terex®	260°F (127°C)	245°F (118°C)	- Volumetrics of the WMA Use of RAP in WMA	- Comparable volumetrics to HMA - Tender compaction zone was avoided
LoJac Enterprises, Inc.	State Rd 46, Williamson County, TN	2007	Astec DBG®	260°F (127°C)	230°F (110°C)	- Workability - Rutting - Internal moisture - Freeze/thaw damage - Moisture susceptibility and stripping problems - Applicability of HMA mix design to WMA	- Harder to achieve target compaction at very low temperatures - Starting construction at a higher temperature and gradually reducing it worked best

Table 2.4. Documented Trial WMA Pavements (Continued)

Contractor	Location	Date	Technologies	Production Temperature	Compaction Temperature	Concerns/Objectives	Lessons Learned
Shelley & Sands, Inc.	Ohio State Route 541	September 2006	Aspha-min®, Evotherm™, Sasobit®		230-260°F (110-127°C)	- Reduced asphalt fumes at the plant and paving site - Reduce energy consumption - Extend the paving season - Increase haul distance - Use of RAP in WMA	- Reduction in emissions as high as 77% - Promising technology with no apparent performance tradeoff
Suit-Kote Corp.	State Rt. 11, Cortland, NY	July 2006	Low-Energy Asphalt - LEA	200°F (93°C)	180-205°F (82-96°C)	- Resistance to heavy traffic soon after construction	- Good in-place and less variable density achieved - WMA exhibited less cracking vs. HMA - Energy consumption reduced by 47% - Good performance of WMA after 1-yr in-service
Bitumar, Inc.	Victoria St, Ottawa	October 2007	Sasobit®	265°F (129°C)	230°F (110°C)	- Evaluate the performance of WMA using Sasobit - Change in paving and compaction procedures - Costs	- No problems encountered during production, paving, or compaction of WMA - Fuel savings of up to 30% vs. HMA.
Coco Asphalt Engineering	Ramps on Hwy 401, Hamilton, Ontario	October 2008	HyperTherm™	255°F (124°C)	200°F (93°C)	- Compare performance properties of WMA vs. HMA - Evaluate use of polymer-modified binder in WMA.	- Both WMA and HMA had acceptable moisture susceptibility characteristics - WMA had marginally larger rutting levels - WMA had better fatigue characteristics vs. HMA.
LaFarge, Inc.	Taradale, Calgary, Alberta	August 2005	Warm-Foam	215F (102°C)	N/A	- Calibration and coordination of the soft binder, water for foaming, and hard binder. - Moisture susceptibility potential	- Similar volumetric characteristics of WMA vs. HMA - Fatigue performance of WMA better vs. HMA
	Katimavik Rd and Oxford Rd 4, Ottawa	October-December 2007	HyperTherm™	250F (121°C)	235F (113°C)	- Extend the paving season by placing WMA during cold weather	- WMA can be placed in extreme cold temperatures - WMA provides increased workability

Table 2.4. Documented Trial WMA Pavements (Continued)

Contractor	Location	Date	Technologies	Production Temperature	Compaction Temperature	Concerns/Objectives	Lessons Learned
LaFarge, Inc.	Vancouver, British Columbia	September 2007	Astec DBG®	265F (129°C)	N/A	- Evaluate economic, environmental, and performance factors to assess the sustainability of WMA - Evaluate introduction of RAP and RAS.	- At a mixing temperature of 250°F (121°C) with 50% RAP non-uniform coating occurred - Compaction of WMA with RAP and RAS was enhanced and workability improved - Moisture susceptibility is not affected negatively by the DBG® process - Energy savings of 24% and emissions reductions of 10% - Visual evaluation after two years showed good performance
McAsphalt Industries	Hwy 111, Saint John, New Brunswick	2007	Evotherm™	265F (129°C)	200F (93°C)	- Physical properties and performance comparison of WMA vs. HMA	- Significant decrease in fumes - Safer and more pleasant work environment - WMA more workable - Easier to achieve required density - Less segregation observed in the WMA - Tighter longitudinal joints achieved with WMA - Good performance after 1 year in service
	Rt. 106, Allison, New Brunswick	2008	Evotherm™ 3G				
	Rt. 135, Burnsville, New Brunswick	2008	Evotherm™ 3G	265F (129°C)	245F (118°C)		

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NCHRP PROJECT 9-49A

PERFORMANCE OF WMA TECHNOLOGIES: STAGE II—LONG-TERM FIELD PERFORMANCE

PHASE I INTERIM REPORT
EXCERPTED LITERATURE REVIEW

Prepared for
Transportation Research Board
Of
The National Academies

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September 2011

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2.2 STATE STUDIES OF WMA PAVEMENT FIELD PERFORMANCE

2.2.1 Texas

A number of WMA pavements with control HMA sections have been constructed and evaluated in the state of Texas.

San Antonio Loop 368 [Estakhri, et al, 2010; Button et al, 2007]

Texas Department of Transportation (TxDOT) placed their first warm-mix asphalt trial using the Evotherm ET technology on Loop 368 of the San Antonio District in 2006. The existing pavement (prior to placement of the warm mix and control) consisted of a cold-milled asphalt surface that had been seal coated with AC-15P and a Grade 4 precoated aggregate. The thickness of the WMA overlay is 2 inches. Plant-produced loose WMA and HMA mixtures were reheated in the laboratory to prepare gyratory samples. The compaction temperatures for WMA and HMA mixtures were 240°F and 300°F, respectively. These samples were used to conduct laboratory

tests including Hamburg Wheel Tracking test (HWTT), overlay test, and density test using X-ray CT images.

Both WMA and HMA pavements were evaluated one month, one year, and three years after pavement construction based on field performance evaluation and laboratory testing on field cores (HWTT, overlay test, resilient modulus test, and air void distribution using X-ray CT). As indicated from one-year cores, HMA samples had clear asphalt absorption while no asphalt absorption was found in the WMA samples (Figure 2.1). The WMA and HMA samples obtained one year after construction behaved similarly in the HWTT and overlay tests. After three years, some cracking started to develop in both the WMA and HMA sections, as shown in Figure 2.2. However, there is no evidence of any rutting in either WMA or HMA sections.

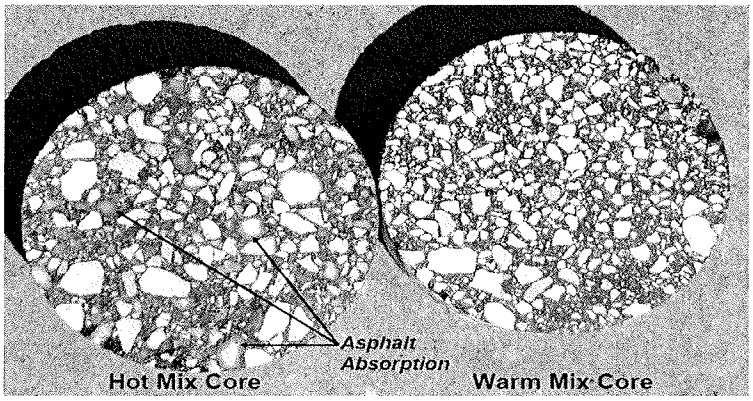

Figure 2.1 Photos of Loop 368 One-Year Cores [Estakhri, et al, 2010]

Figure 2.2 Evidence of Cracking in both WMA and HMA Sections [Estakhri, et al, 2010]

Austin SH 71[Estakhri et al, 2010]

The Austin SH 71 project was constructed in 2008 using Evotherm DAT technology along with an HMA control section. The production temperatures were 330°F for HMA and 240 °F for WMA. The design mix was a Type C dense-graded mix for both WMA and HMA. The thickness of the overlay is 2 inches.

Soon after construction, both WMA and HMA cores were taken and tested using HWTT rutting test and indirect tensile strength test. HWTT results indicated that the rutting resistance of HMA is better than that of Evotherm DAT technology. Indirect tensile strength test results showed that the HMA is stronger than WMA. Based on the results of ground penetration radar (GPR), the WMA pavement had similar in-place density and uniformity as the HMA control section.

After one year of service, field cores were taken from both WMA and HMA pavements to conduct laboratory tests. Tests on these cores included air voids, indirect tensile strength, overlay test, and HWT test. Visual survey indicated that the pavements performed well after one year's service and there were no signs of distress in either HMA or WMA pavement sections.

Lufkin FM 324 [Estakhri et al, 2010]

In 2008, the Lufkin district placed WMA pavement field trials with four different WMA technologies, Sasobit®, Evotherm DAT®, Akzo Nobel Rediset® and Advera®. HMA pavement was also constructed as a control. A Type D dense-graded mix was used for both HMA and WMA, with a production temperature of 330°F for HMA and 240°F for WMA. The thickness of WMA layer is 1.5 inches.

During construction, TTI's Pave-IR System was used to evaluate the thermal characteristics of the WMA and HMA materials. Pave-IR thermal images indicated that the temperature of HMA material is higher than that of WMA material. The WMA pavement also had a more uniform temperature distribution. Plant-mixed mixtures were cured for 2 hours at 250°F for the HMA and 220°F for all of the WMA, followed by on-site compaction. These samples were sent to laboratory for HWTT testing and overlay testing. HWTT results indicated that only the Sasobit and HMA mix passed the HWTT criteria. Overlay test results showed that compared with the HMA, the Advera and Evotherm improved dramatically in cracking resistance.

Visual inspection indicated that all four WMA pavement sections and the HMA control sections performed well in the first year of service. No evidence of rutting or cracking has been observed. Field cores were also taken for conducting air voids testing, indirect tensile strength testing, overlay testing, and HWT rut depth testing.

Fort Worth BU 287 Project [Estakhri et al, 2010]

In 2008, the Fort Worth project was constructed on BU 287 north of Saginaw using Evotherm DAT, but with no HMA control section. A similar pavement, SH114 in the Fort Worth district was used for comparison. The average daily traffic of the road was 24,100 vehicles per day. The

existing pavement structure varies throughout the project. Much of the cross-section consists of several inches asphalt concrete pavement over 8 inches of crushed stone flexible base. A portion of the project is CRCP with an existing 3.5-inch overlay which was milled and replaced; and another portion consists of jointed concrete pavement. The WMA project consists of two portions, 10 inches Type-B WMA shoulder rehabilitation and 3.5 inches of type-D WMA resurfacing on the entire project. The mixture production temperatures at load-out were 240 °F for Type-B WMA mix and 275 °F for Type-D WMA mix.

Cores were taken at the time of construction and then after one year's service. HWTT, indirect tensile strength test, and air void measurements were conducted for both Type-B and Type-D WMA mix.

The pavement was evaluated for the field performance. The GPR data revealed that the almost 14 inch thick WMA shoulder on BU 287 was uniformly compacted throughout the depth with no signs of any defects. Two months after construction, falling weight deflectometer (FWD) tests were performed on the WMA section and a similar HMA section. Based on testing data, there was no significant difference in structural strength characteristics of the two pavements. After one year service, the WMA section performed well with no evidence of rutting or cracking distress.

2.2.2 Alabama

SR-79 [Kvasnak et al, 2010]

This project was constructed in the northwest of Tarrant City, Alabama on SR-79 in 2007. The majority of the project was a two-lane highway, and the WMA and the associated HMA control mixes were placed in the southbound lane. The project used Evotherm DAT as the WMA additive. Reclaimed asphalt pavement (RAP) and recycled asphalt shingles (RAS) were included in both WMA and HMA mixes. The production temperature ranged from 225 °F to 255 °F for WMA mix and 295 to 355°F for HMA mix.

Loose mixtures were sampled to determine asphalt content, asphalt binder properties, and aggregate gradations. Plant-produced mixtures were compacted in the field and tested using HWTT, indirect tensile strength test, and asphalt pavement analyzer (APA) test for evaluating moisture susceptibility, rutting resistance and cracking potential of the mix. Additional specimens were reheated and compacted in the laboratory for conducting dynamic modulus and flow number testing. TSR test results showed that the HMA specimens exhibited higher tensile strengths and tensile strength ratios (TSR) than the WMA. The results from the APA, HWTT, flow number, and dynamic modulus test consistently indicate that the WMA mix was less stiff and slightly more rutting-susceptible than the HMA control mix. Indirect tensile strength tests showed that compared with other mixes placed, the WMA is less resistant to load-induced damage (e.g. fatigue cracking). The Hamburg wheel tracking results indicated that the WMA may be more susceptible to stripping damage than the HMA although all mixes passed the Hamburg stripping inflection criterion of a minimum of 5,000 cycles.

Three site revisits were conducted. The first and the second site revisits included visual inspection. On the third revisit, both visual inspection and evaluation on field cores were conducted. At the first site revisit, the pavements were found to have segregation issues regardless of mix type. No cracking and other distress were observed. At the second revisit, pavement distress was photographed. An example of HMA mix segregation was shown in Figure 2.3. Cracks in the WMA paving sections and patches in the HMA sections were also found (Figure 2.4). The usage of the WMA additives was found to have no influence on the bond between pavement lifts. During the third site revisit which was one year after the construction, field cores were taken from each section. Asphalt contents, aggregate gradations, air voids, indirect tensile strength, and bond strength were determined from field cores. The IDT strength of the WMA field cores was found to be similar to the HMA cored samples one year after construction.

Figure 2.3 Mix Segregation in HMA Pavement for AL SR-79 Project [Kvasnak et al, 2010]

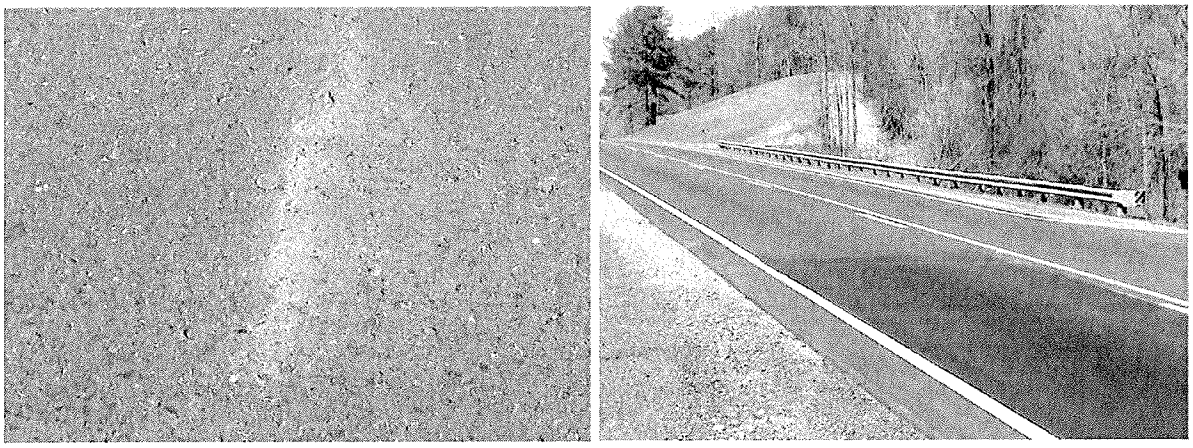

Figure 2.4 Crack in WMA Sections (left) and Patch in HMA Sections (right) [Kvasnak et al, 2010]

NCAT Test Sections [Prowell et al, 2007; Chowdhury and Button, 2008]

A WMA project (Evotherm ET) with a HMA control section was constructed at NCAT test track in early November 2005. Figure 2.5 shows a section view of the layers. Sections N1 and N2 were milled to a depth of 5 inches to complete the structural rehabilitation of those sections. Two lifts of 19.0-mm nominal maximum aggregate size (NMAS) WMA were placed in Sections N1 and N2. The top 1 inch of sections N1 and N2 was Evotherm with PG67-22 binder and HMA control mix. Section E9 consisted of the original track structure, 20 inches of HMA, 5 inches of asphalt treated drainable base, and 6 inches of aggregate, with a 1-inch Evotherm overlay at the surface. The production temperatures for WMA and HMA were 240°F and 327°F, respectively.

Plant-mixed HMA and WMA mixtures were compacted in the laboratory close to test sections. The mixtures were evaluated for volumetrics, rutting resistance (APA test) and moisture susceptibility (AASHTO T283). Manual distress survey was conducted and field cores were taken to conduct moisture susceptibility test. The two WMA sections and HMA section showed excellent field performance in terms of rutting, after an application of little over half a million ESALs within 43 days.

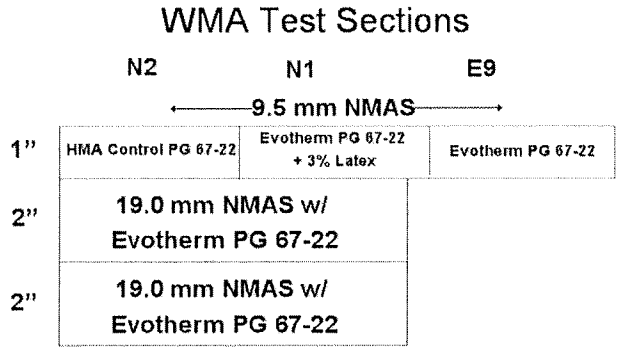

Figure 2.5 Section View of Evotherm Layers at NCAT Test Track [Prowell et al, 2007]

2.2.3 Florida

Orlando demonstration project, Florida [Hurley and Prowell, 2005; Chowdhury and Button, 2008]

In February 2004, a field demonstration project using Aspha-min® zeolite was constructed at Hubbard Construction's equipment yard in Orlando, Florida. The existing surface was milled and tacked prior to paving. The control mix was a fine-graded Superpave mix with 20% RAP, and was designed for a traffic level of 3-10 million ESALs. The production temperatures for Aspha-min® and HMA control mix were 300°F and 336°F, respectively.

Plant-mixed loose WMA and HMA mixtures were compacted in laboratory at compaction temperatures of 270°F and 300°F, respectively. APA tests were conducted to determine the rutting potential of the mixes and TSR tests were used to evaluate the moisture susceptibility. In the field, the WMA mix was reported to be more workable than the control

mix. Based on an analysis of field core samples, the WMA section was found to have comparable density as the control section.

The pavement was revisited in 2005. No signs of distress were evident in either the Aspha-min® warm mix lanes or the control mix lanes. Five six-inch diameter cores were taken from each section to determine ITS and density.

SR-417 project [Sholar and Nash, 2009]

This project consisted of a 0.758-mile test section of FC-5 open-graded friction course placed in the southbound passing lane of SR-417 utilizing the Aspha-min® WMA technology. Directly to the north of the WMA test section is a 1.024 mile control section, consisting of the same FC-5, without the Aspha-min® additive. Both mixtures contain a polymer modified PG 76-22 asphalt binder and were constructed in February 2006.

The mixing temperature was 320°F for the HMA control mixture and was 270°F for the WMA mixture. Samples of each mixture type were obtained and tested for their cracking properties based on the Energy Ratio concept developed at the University of Florida. Pavement condition surveys were performed in May 2006 and July 2009 to evaluate the rutting, cracking, and ride rating performance of each section. Results showed that there was no difference in field performance between the HMA and WMA sections.

US-92 (SR-600) project [Sholar and Nash, 2009]

This project consisted of a 1.164-mile test section of SP-12.5 structural mix placed in the eastbound travel and passing lanes of US-92 in Lakeland utilizing the Evotherm DAT WMA technology. Directly to the west of the WMA test section is a 0.634-mile control section, consisting of the same SP-12.5 mixture but without the Evotherm additive. Both mixtures contained a polymer modified PG 76-22 asphalt binder and were constructed in October 2007. Subsequently, a conventional HMA FC-5 open-graded friction course mixture was placed over the structural mix. The mixing temperature was 325°F for the HMA control mixture and was 250°F for the WMA mixture.

Samples of each mixture type were tested to determine the cracking properties of the mix utilizing the Energy Ratio concept, rutting performance utilizing the Asphalt Pavement Analyzer (APA), and moisture susceptibility utilizing the tensile strength ratio (TSR) approach.

Pavement condition surveys were performed in November 2007 and December 2008 to evaluate the rutting, cracking, and ride rating performance of each section. Results indicated that there were no practical differences between the WMA and HMA control sections.

SR-11, Flagler County [Sholar and Nash, 2009; Copeland et al, 2010]

The SR-11 WMA project was located at the south of Bunnell utilizing the Astec Double Barrel Green (DBG) technology with an HMA control section. Both WMA and HMA mixtures contained a RA-800 asphalt binder and 45% fractionated RAP. The test sections were constructed in December 2007 and January 2008. Subsequently, a conventional HMA FC-12.5

dense-graded friction course mixture was placed over the structural mixtures. The mixing temperatures for the WMA and HMA control mixture were 270°F and 310°F, respectively.

During construction, PG grading was conducted in accordance with AASHTO M 320 and the binder samples were taken from five sources: virgin binder, coarse Fractionated RAP (FRAP), fine FRAP, plant-produced control mix, and plant-produced WMA mix. Both plant mixes were reheated to 300°F before conducting binder extraction and recovery. Additional plant-produced loose mixtures were collected to test dynamic modulus and flow number and these samples were also reheated to 300°F before compaction. PG grading test showed that the high RAP WMA mix had a lower PG grade than that of the high RAP HMA mix.

Samples of each mixture type were tested for the cracking properties using the Energy Ratio concept, the rutting performance using the APA and flow number, the moisture susceptibility using the tensile strength ratio (TSR), and the AMPT dynamic modulus. Mixture tests results indicated that both $|E^*|$ values and the Fn values of RAP WMA mixture are lower than that of the RAP control mix.

Pavement condition surveys were performed in June 2008 and July 2009 to evaluate the rutting, cracking, and ride rating performance of each section. Results showed that there were no practical differences between the HMA and WMA sections.

2.2.4 Tennessee

SR-46, Franklin, Tennessee [Kvasnak et al, 2010]

The project in Tennessee is located in Franklin on SR-46 and was built in 2007 using Advera® WMA, Astec DBG®, Evotherm DAT, and Sasobit® plus two control HMA sections. It was also included in the NCHRP 9-47A study. SR-46 is a two-lane road with mostly automobile traffic and the average daily traffic is 10,492. The thickness of the overlay is 1.25 inches. Prior to overlay, TDOT surveyed the condition of the existing pavements in terms of roughness index (PSI), IRI, rut depth, distress index and pavement quality index (PQI). The existing pavement surface was cracked and crack sealant had been applied in several locations.

The WMA mixtures were produced between 240 and 260°F (115 and 127°C) and placed at 230°F (110°C). Plant -mixed samples were compacted on site to evaluate compactability, moisture susceptibility (TSR), rutting susceptibility (APA and HWTT), dynamic modulus, and low temperature cracking resistance (indirect tensile creep compliance). Field cores were also taken to measure in-place density and indirect tensile strength. Asphalt binders were extracted and recovered from plant-produced mix to evaluate the aging that occurred at the different mix production temperatures.

Visual inspections were conducted after one year in service. During the assessment, it was noted that the control HMA, the DBG WMA pavement, and the Evotherm DAT WMA pavement exhibited raveling along the centerline of the pavement. The Sasobit® was less affected by raveling. Raveling in the Advera® WMA pavement was more severe, especially in areas shaded by trees. Tests performed on extracted cores from the pavements indicate that the HMA control had the highest indirect tensile strength, while the Advera® WMA and the Sasobit®

exhibited significantly lower indirect tensile strength values [Kvasnak et al., 2010]. Despite the occurrence of raveling in these pavements, the distress was neither severe nor extensive and the reports did not indicate that the functionality or ride quality of the pavement was impacted.

2.2.5 Washington

I-90 West of George Paving project [Russell et al, 2009]

This project is located on I-90 west of George, Washington using Sasobit WMA technology. The average daily traffic (ADT) ranged between 6448 and 7327 with 27 percent trucks according to traffic data from the 2008 Washington State Pavement Management System (WSPMS). The pavement structure consisted of a 0.25-feet existing HMA pavement with a 3" overlay (WMA or HMA control). Both HMA and WMA mixtures included 20% RAP. The average production temperatures for WMA and HMA control sections were 286°F and 328°F, respectively.

FHWA mobile asphalt testing laboratory (MATL) assisted on the laboratory experiments for this project. Tests included dynamic modulus test using an Asphalt Mixture Performance Tester (AMPT), flow number test, and Hamburg Wheel-Tracking (HWT) test. The Sasobit® mixture appeared to be stiffer and more rutting-resistant than the HMA control mix, based on the results of dynamic modulus tests and flow number tests. HWT test results suggested the resistance of both Sasobit WMA mix and the HMA control mix to permanent deformation is very good.

2.2.6 Wyoming

Yellowstone National Park [Neitzke and Wasill, 2009]

The project reconstructed a portion of the East Entrance Road of Yellowstone National Park. The mixtures were placed in August/September 2007 using two different WMA technologies, Advera® and Sasobit®. Conventional hot mix was also paved as a control. The traffic loadings for a 20-year design life were estimated to be 1,000,000 ESALs. The pavement structure was designed using the DARWin pavement structure design program. The pavement structure consisted of three layers. The subbase was constructed using full-depth reclaimed pavement. This material was placed and compacted to a depth of 6 inches (150mm). The base layer was a crushed aggregate base treated with 1% emulsified asphalt. This material was compacted to a finished depth of 8 inches (200mm) and provided an excellent paving platform for the asphalt concrete pavement. The surface layer was 4 inches (100mm) of asphalt concrete pavement placed in two equal lifts. The production temperature ranged from 300°F to 325°F for HMA, between 250 °F to 275°F for Sasobit® WMA, and was 250°F for Advera® WMA.

During the production of the three mixtures, loose mix samples were taken periodically from the windrow. The gradations for the three different mixtures were fairly consistent. In-place density values, tensile strength ratio, and rutting resistance using Hamburg and APA were evaluated based on plant-mixed lab-compacted samples. Asphalt binder was extracted and recovered from the mixtures obtained during the production for evaluation of binder properties.

2.2.7 Maryland

MD Route 925 [Burke, 2006]

In 2005, the Maryland State Highway Administration (SHA) completed a WMA project in Charles County using Sasobit® technology with 15% RAP. A control mix was also paved with 15% RAP. The mix was designed at the 0.3 to 3 million ESALs level using a PG 64-22 binder. During construction, the temperature of the control mix behind the paver was approximately 310°F to 350°F, and the temperature of the WMA mix was 270°F. A thermal camera was also used to track the temperatures during paving, and the results indicated a fairly consistent mat temperature distribution. The WMA section with Sasobit® was reported to be easy to handle and have better compactability even at a lower temperature.

Visual inspection after one year indicated that the pavements performed well in both HMA and WMA sections with no visible signs of deterioration or distress, as shown in Figure 2.6. It exhibited a smooth texture and a minimal amount of segregation. However, after more than five-years of service life, some areas of segregation and centerline cracking have been observed on both control and WMA sections.

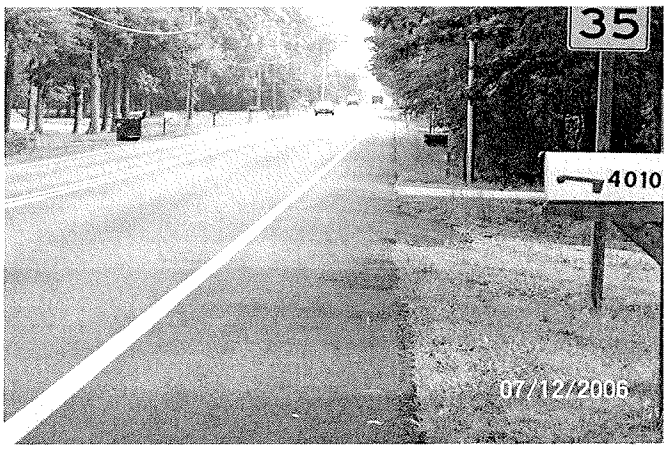

Figure 2.6 Visual Inspection of MD Route 925 in 2006 [Burke, 2006]

2.2.8 Nebraska

Antelope County Nebraska Trial Sections [Kim et al, 2010]

In 2008, Nebraska Department of Roads (NDOR) paved four trial sections, including two WMA sections (Evotherm WMA and Advera® zeolite WMA) and their control HMA section, in Antelope County, Nebraska. The trial sections started from Elgin and ended at US Highway 20.

During construction, compaction temperature was around 255°F (124°C) for WMA and around 275°F (135°C) for HMA. Field-mixed loose mixtures were collected and transported to the laboratories for comprehensive evaluations. Tests included PG grading for extracted and recovered asphalt binder, dynamic modulus test, creep compliance test, uniaxial static creep test, APA test, TSR test, and semi-circular bending (SCB) fracture test with moisture conditioning. In general, the WMA additives were found to have no significant impact on the viscoelastic

stiffness characteristics of the asphalt mixtures. All WMA mixtures showed better rutting resistance than the HMA control mix, with the Sasobit® WMA had the highest rut resistance. Based on the results from AASHTO T283 test and the SCB fracture test with moisture conditioning, WMA mixtures showed greater potential to moisture damage than other mixes.

The pavement sites were visited in 2009 (one year after placement) and in 2010 (two years after placement). Visual evaluations of each section indicated that both the WMA and HMA sections performed well without any major distresses. In addition to the visual evaluation, the performance of WMA pavements was also assessed using pavement performance data (roughness, rut depth, and surface texture) collected by PathRunner. The rut depth and the roughness of WMA and HMA sections were found to be similar.

2.2.9 Louisiana

LA 1 Lafourche Parish Project [Cooper 2009]

The Lafourche Parish Project was paved in 2005 using Sasobit® WMA technology and a control HMA. Both HMA and WMA mixture used a PG76-22 polymer modified binder, siliceous limestone aggregate, and 19% RAP. The same temperature was used for HMA and WMA mix production (310 to 330°F) except that the temperature for one part of Sasobit® section was reduced to approximately 290°F.

Asphalt binders were PG graded based on the Superpave specifications. Laboratory-mixed gyratory specimens were prepared to evaluate durability, permanent deformation (loaded wheel tracking test, Hamburg type) and moisture susceptibility (Modified Lottman test) of the mixes. The testing results indicated that Sasobit® additive had no significant effects in terms of permanent deformation resistance and moisture susceptibility of the mix. The Sasobit® section was found to have better compactability than the HMA control pavement based on the in-place density results from nuclear gauge and field cores. The addition of Sasobit® may adversely affect the low temperature properties of the original asphalt cement binder being utilized as observed through asphalt cement binder rheology testing.

2.2.10 Michigan

Iron Mountain M95 Project [Hurley et al, 2009]

The Michigan Iron Mountain M95 project was constructed in 2006 using Sasobit® WMA technology along with an HMA control section. The WMA was used as an overlay for the top 1.5 inches of the surface course in the passing lane. The control test section was placed in the newly constructed adjacent travel lane of M95. For the control HMA mix, the mixing temperature was approximately 325°F (163°C) and the compaction temperature was 300°F (149°C). For the WMA mix, the mixing and compaction temperatures were 260°F (127°C) and 250°F (121°C), respectively.

This project was monitored by the NCHRP project 9-47A team. During construction, loose mixtures were sampled at the plant and compacted on-site. Performance tests were conducted, including Asphalt Pavement Analyzer (APA) test (AASHTO TP 63), tensile strength

ratio (TSR) test (AASHTO T 283), Hamburg Wheel Tracking test (AASHTO T 324), and dynamic modulus test (AASHTO TP 62). Additional mix was also sampled such that comparisons could be made between on-site compacted samples and samples that were reheated prior to compaction.

The site was revisited two years after construction to compare the field performance of the WMA to that of the HMA based on visual observations, field rut depth measurements, and field core analysis. The visual inspections were conducted to identify and classify any distresses in accordance with the LTPP guidelines. Rut depth measurements were taken in both the WMA and HMA sections using a string line. Field cores were obtained to evaluate the pavement densification due to traffic loading and the indirect tensile strength. Based on field test results, minor pop-outs of coarse aggregate particles were observed in the Sasobit® section. Rut depths of 1/16 inch were recorded in the right wheel path of the control section and 1/8 inch in the left wheel path of the control section. After two years, the in-place density of Sasobt® section was similar to the control HMA section, although the HMA section had lower density than the Sasobit® section right after construction. Hurley et al. [2009] explained it could be due to the Sasobit® being in the passing lane and thus not receiving the same amount of traffic as the control section. The indirect tensile strengths of both the Sasobit®WMA and the control HMA increased as expected after two years of in-place aging. No visual stripping was observed in the field cores from either section. A photo of the test sections were shown in Figure 2.12.

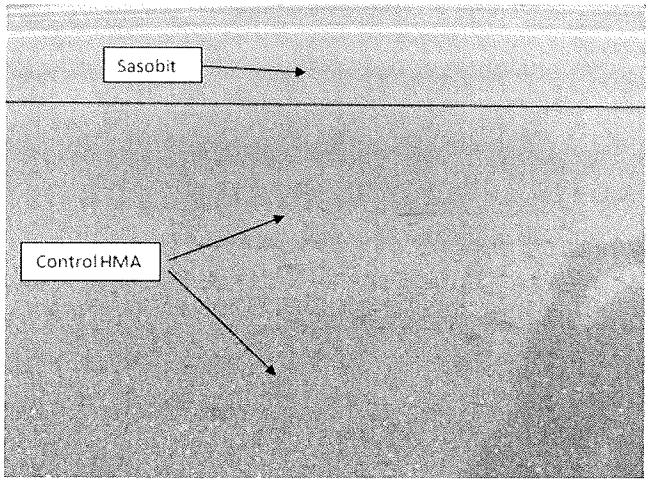

FIGURE 2.7 Sasobit® and Control Test Sections after Two Years of Traffic [Hurley et al, 2009]

2.2.11 Missouri

Hall Street [Hurley et al, 2010]

The Hall Street WMA project was constructed in 2006 using three WMA technologies, Asphamin®, Sasobit®, and Evotherm ET, and one HMA control mix. Both of the WMA and the HMA mixtures contained 10% reclaimed asphalt pavement (RAP). The average annual daily traffic (AADT) for Hall Street is approximately 21,000 vehicles per day, with 7% trucks. The existing pavement structure consisted of a concrete pavement that had been previously overlaid with

HMA. The production temperatures for the HMA control section, Sasobit® section, Evotherm ET section, and Aspha-min® section were 320°F, 240°F, 225 °F, and 275°F respectively.

This project is monitored by the NCHRP 9-47A team. During construction, loose mix samples of each asphalt mixture were obtained from the end-dump trucks as they were leaving the asphalt plant and were used to produce test specimens for performance testing. Laboratory tests including mixture volumetric tests, APA rutting test (AASHTO TP 63), TSR (AASHTO T 283) and Hamburg Wheel Tracking moisture test(AASHTO T324), and Dynamic modulus. For the Evotherm ™ ET and Sasobit®, all specimens were prepared on site with no reheating except for dynamic modulus (E*) specimens. For the Aspha-min® section, all specimens were prepared back in the laboratory with reheating.

Site revisits were conducted to assess the performance of the WMA and HMA pavements. Visual inspections were conducted to identify distresses such as cracking, raveling, flushing, and polished aggregate. The field performance data as of 2008 indicted that limited number of cracks was identified in each section. These cracks seemed to be reflective in nature and appeared to be tight. Visual observation also indicated that there seemed to be a surface-texture difference between the center turn lane placed in 2005 and the WMA and HMA placed in 2006, with the 2006 pavement having rough macrotexture. In general, the pavements performed well after two years of service life with minimal rutting and cracking in all WMA and HMA sections.

2.2.12 Ohio

SR 541 [Hurley et al, 2009; Sargand et al, 2009; Nazzal et al, 2011]

The project in Kimbolton, Ohio is located on SR-541 and was built in 2006 using Evotherm, Sasobit®, and Aspha-min®, along with an HMA control section. Each section was approximately 10 miles long. The road was originally constructed in the early 1960's using 1.25 inches (31.8 mm) asphalt surface, 1.75 inches (44.5 mm) asphalt intermediate layer, 5 inches (127 mm) of granular base, and 4 inches (102 mm) of granular material. An overlay of 1.5 inches (38 mm) of asphalt was applied to the west end in 1985 and the east end in 1987. Another overlay of 1.5 inches (38 mm) asphalt was added to the entire segment in 1994. The 2006 2" overlay layer included two lifts, a standard 0.75" HMA leveling course plus a 1.25" wearing course placed in four sections (one HMA control section and three WMA test sections).

The materials used in the mixture included a PG 70-22 modified binder with 15% RAP and limestone aggregate. The compaction temperatures of the WMA ranged from 240 to 260°F (110 to 127°C). During construction of the test sections, gyratory samples were compacted on site. A number of laboratory tests were conducted including mixture volumetric testing, Asphalt Pavement Analyzer (APA) rut testing, AASHTO T 283 testing, Hamburg testing, and dynamic modulus testing. Additional mix was also sampled such that comparisons could be determined between on-site compacted samples and samples that were reheated prior to compaction. No testing was conducted to evaluate the effects of WMA additives on asphalt binder properties.

Field cores were taken 3 months, 12 months, 20 months, 22 months, and 46 months after construction to evaluate the air voids distribution and indirect tensile strength (ITS). The WMA mixtures were found to have higher indirect tensile strength than the HMA mixture after three months of service; while the HMA ITS values became the highest after 46 months. After 46 months of service, moisture susceptibility testing (AASHTO T283 test) indicated the WMA and HMA had acceptable resistance to moisture damage.

After periodic monitoring of the pavement based on field cores evaluation, visual distress survey, and roughness measurement, the WMA sections were reported to exhibit low levels of raveling (defined by ODOT as "disintegration of the pavement from the surface downward due to the loss of aggregate particles"). Based on the visual observations, the Evotherm section had the smallest degree of distress while the Sasobit® showed the greatest. Factors contributing to the raveling were attributed to poor construction practices (i.e., mixture dragging under the screed and poor handwork) as well as the lower placement temperatures, especially for Sasobit® [Hurley et al., 2009].

IRI values were taken after 3, 27 and 46 months of construction. ANOVA analysis was conducted to analyze IRI data and the results indicated that IRI values of the WMA sections are similar to that of the HMA sections. Mechanistic Empirical Pavement Design (MEPDG) program was used to predict the pavement life, and the results indicated longer service life for the Aspha-min section than the HMA control section. Based on MEPDG results, the Evotherm and Sasobit sections will require rehabilitation sooner than the HMA control section. After 46 months of service, no measureable rutting was observed. Visual inspections showed no performance problems in those sections.

Accelerated Pavement Loading Facility (APLF) in Lancaster Ohio

The same three WMA technologies, Aspha-min®, Evotherm ET, and Astec DBG as used in the SR541 project, were also evaluated at the Accelerated Paving Load Facility (APLF) in Lancaster Ohio under controlled traffic and pavement structures. The indoor APLF facility can apply dual or wide-based single wheel loads of up to 30,000 lbs (133kN) and control air temperatures between +10°F (-12.2°C) and +130°F (54.4°C). The pavements are fully instrumented to measure dynamic responses under the rolling wheels and environmental conditions in the pavement structures. The rutting performance can also be measured based on surface profile. The northern pavement sections were constructed as perpetual pavement with 16 inches total thickness of asphalt layer, of which the top 1.25 inch surface layer was paved using either WMA or HMA control mixtures. The APLF section used the same WMA mixtures as used in the SR541 project.

Indirect tensile creep and tensile strength tests were conducted on lab-fabricated specimens at three different temperatures: 0°C (32°F), -10°C (14°F), and -20°C (-4°F). The results were used to determine the creep compliance curve, unconfined tensile strength, and Poisson's ratio. Cores were taken at the time of construction to evaluate air voids and indirect tensile strength. Strain under fatigue resistant layer, pressure under based layer, deflection of

subgrade, and FWD results obtained in the APLF facility were collected which provided comprehensive information for pavement analysis.

2.2.13 Virginia

Highland County, Rappahannock County, and York County test sections [Diefenderfer, 2010]

Three trial sections using two WMA technologies were constructed in Virginia in 2006. Two WMA sections were built using Sasobit® technology and the third section was based on Evotherm ET. For the Sasobit® sections (1 and 2), the mixtures contained 20% and 10% reclaimed asphalt pavement (RAP). For the Evotherm ET section (3), the mixture contained 20% RAP. The experiences with these trial sections were used in the development of the Virginia Department of Transportation's special provision to allow the use of WMA.

The sections were evaluated over a 2-year period to assess the initial performance of the WMA pavements and compare it with that of the HMA control sections. Coring and visual inspections were performed during the initial construction and at intervals of 3 months, 6 months, 1 year, and 2 years. The cores were tested to determine air-void contents and permeability, followed by asphalt binder extraction and recovery to determine the binder's PG grading. In addition, for the two Sasobit® trial sites, historic data, cores, and ground-penetrating radar (GPR) results were collected to compare the results on pavement structure.

Visual surveys indicated no significant distresses in the WMA or the HMA sections during the first 2 years in service. Evaluations of the air voids for field cores indicated that generally the air voids for the WMA and HMA were not significantly different from each other. The air-void contents at different ages were significantly different in a few instances; however, no trends concerning air voids were observed. Permeability measurements did not indicate any trends concerning permeability over time. The PG grading of the recovered binder suggested that the WMA produced using Sasobit® aged at a slightly reduced rate than the HMA, as indicated by decreased stiffening effect. No difference in performance grade was measured between the HMA and WMA sections produced using the Evotherm emulsion technology.

2.2.14 Wisconsin

Milwaukee, WI WMA Project [Hurley et al, 2010]

The State Highway 100 (Ryan Road) project (Figure 2.14) in Wisconsin was constructed in 2006 using Sasobit® and Evotherm technologies and an HMA control section. The pavement structure includes 6.25 inches asphalt mixtures (design ESALs 3million) plus 4 inches open-graded base course on 8.5 inches dense-graded aggregate base course (DGBC). The WMA was used in the top 1.75 inches surface course in the westbound passing lane. Both the control and the WMA mixtures contained 14% reclaimed asphalt pavement (RAP). During construction, the compaction temperature was 300°F (149°C) for the HMA mixture and 250°F (121°C) for the WMA mixture.

Experimental tests were conducted based on plant mixed laboratory compacted samples with and without reheating. These tests included APA test, TSR test (AASHTO T283), Hamburg

Wheel Tracking test, and dynamic modulus test. Evotherm™ emulsion mixture exhibited higher rut depths, lower tensile strengths, and lower moduli than the HMA, which may be a result of fuel contamination. The laboratory APA test indicated that the Evotherm was more rut susceptible than the control HMA while the Sasobit is less rut susceptible than the control. The fuel contamination of the Evotherm was believed to contribute to the high rut depth in the APA testing. The field rut measurement indicated that the rut depth of WMA sections was comparable to the HMA section after four months of traffic. All mixtures except the reheated Evotherm samples had similar TSR values, indicating similar performance in resistance to moisture damage. This finding was confirmed by the results of the Hamburg wheel-tracking tests. Field performance of the three pavement sections were evaluated four months after construction and the pavements had comparable performance.

2.2.15 California

California HVS Study on Warm-Mix Asphalt [Jones et al, 2008]

The University of California Pavement Research Center (UCPRC) constructed an 262'×262' test track containing three warm-mix additives (Advera®, Evotherm DAT, and Sasobit®) as well as one HMA control mix in 2007. The location of the test track was at one of Granite Rock's plants near Aromas, California (Figure 2.15). Using a standard mix design with no changes made for the insertion of the warm mix additives, the asphalt was placed in two 2.4-in lifts on top of 12-in imported aggregate base.

Plant-mixed, laboratory-compacted (FMLC) specimens were prepared on site to conduct laboratory testing including shear test, wet and dry fatigue test, Hamburg Wheel-Track test, and AASHTO T283 moisture susceptibility test. Laboratory moisture sensitivity testing indicated that all the mixes tested (both WMA mixes and HMA control mix) were potentially susceptible to moisture damage. There was, however, no difference in the level of moisture sensitivity between the control mix and the WMA mixes evaluated in this study.

Heavy Vehicle Simulator (HVS) was used to assess the pavement performance under controlled traffic and determine whether the WMA additive could adversely affect the performance of asphalt pavements. The results indicated that these three WMA additives did not influence the rutting performance of the mixture. Based on the laboratory fatigue testing results and the strong pavement structure on the test track, the occurrence of fatigue cracking under HVS testing seems to be unlikely. In addition, HVS moisture sensitivity testing is recommended by Jones et al. [2008] to confirm the laboratory findings that all mixes are susceptible to moisture damage.

Santa Clara Project [Cheng et al, 2010]

In 2006 Santa Clara Rte. 152 received a 0.15' (1.8") overlay on the shoulder using 200 tons of RHMA-G mix with Sasobit® additive. The Sasobit® was added to the rubber binder and mixed for 45 minutes. There was a recorded drop in the production temperature of 40°F (from 320°F to

280°F). The paving took place at night with a 30 minute haul time. After two years, the shoulder was reported as still "looked good".

I-5 District 10 [Cheng et al., 2010]

Astec Double Barrel Green (DBG) and Evotherm WMA technologies were used in the southbound shoulder of I-5 in District 10 near Santa Nella, CA in 2008. The top lift of WMA layer was 0.12' (1.44"). The pavement condition was evaluated in 2010 and there were no signs of distress after two years of service in the field.

2.2.16 Pennsylvania

Pennsylvania SR2012 project (Corrigan et al., 2010)

SR 2012 was built from May to June 2009 using four WMA technologies, Low Emission Asphalt (LEA), Sasobit®, Advera® zeolite, and Gencor foaming. Mixture tests were conducted on plant produced laboratory compacted samples with and without reheating. Details of the experimental testing were shown in Table 2.6. Binder samples were also tested by the FHWA Mobile Asphalt Testing Laboratory (MATL), and the detailed information regarding the binder's testing is presented in Table 2.7. Based on the experimental findings, the HMA and the Sasobit® WMA mixtures demonstrated high resistance to permanent deformation.

Table 2.6 Summary of Mixture Testing for PA SR2012 Project

Mixture Performance	Test Equipment	Test Methods	Test on PMLC samples with reheating	MATL Test on PMLC samples without reheating (by MATL)
Volumetric testing		AASHTO T 30, AASHTO T 209, AASHTO T 269, AASHTO T 308, AASHTO T 312	√	
Rutting	Asphalt Mixture Performance Tester (AMPT)		√	√
	Hamburg Wheel-Track Device (HWTD)	AASHTO T324	√	√
Dynamic Modulus Test	AMPT		√	√
Flow Number Test	AMPT		√	√

Table 2.7 Summary of Binder Testing for PA SR2012 Project

Binder	Test Equipment	Test Method	MATL
Performance Grade (PG), viscoelastic rheological behavior	DSR	AASHTO M320-09 T1, M320-09 T2, M320-09 T3	√

2.2.17 Colorado

I-70 in Silverthorne, Colorado [Tim et al, 2011]

The Colorado Department of Transportation (CDOT) conducted a WMA study in 2007 to compare the production, constructability, laboratory performance, and field performance of three WMA technologies (Advera®, Sasobit®, and Evotherm DAT) with a control HMA. The project was located on I-70 about 70 miles (110 km) west of Denver (Figure 2.23) This project was also included in the NCHRP 9-47A study for short-term field performance monitoring.

The existing pavement is an asphalt pavement with a thickness between 10 to 13 inches. After milling 2.5 inches, a 2.5 inch layer of overlay was constructed. The 10-year design ESALs of the pavement is 4.85 million based on an annual average daily traffic of 30,000 with 10% trucks. The WMA test sections were compacted about 30 to 50°F cooler than the HMA control.

During construction, the in-place densities were measured with a nuclear -density gauge. Cores were also taken from the pavement in the second and third years. The top lift of pavement was removed from the rest of the core by sawing. The top lift was tested for indirect tensile strength.

Experimental testing was conducted on plant-produced laboratory-compacted gyratory samples without reheating. The engineering properties evaluated included the volumetric properties of the mix, the moisture susceptibility (TSR and HWTT), asphalt binder PG grading, dynamic modulus, and flow number. The laboratory performance test results indicated the WMA mixes were slightly more susceptible to rutting and slightly more susceptible to moisture damage than the HMA control mix was.

Field-performance evaluations (manual distress survey) were conducted annually for three years after construction. Data regarding rutting, in-place void, cracking, raveling and weathering were gathered to document the performance. After three years of field evaluations, the performance of the WMA test sections was found to be comparable to the HMA control sections in terms of rutting, cracking, and raveling. The field performance was excellent even though the location typically has a very harsh winter climate.

2.3 SUMMARY

In summary, there are three NCHRP projects that have been conducted or are under going to evaluate the engineering properties and field performance of warm mix asphalt mixtures and pavements. Field WMA studies in 17 states were summarized in this chapter based on their documented project information and field evaluation results, if any. In addition, the team is

continuing to collect WMA field projects that has been studied and documented in the US. This information will provide important historical background for identifying candidate WMA projects to be included into the NCHRP project 9-49A study.

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NCHRP PROJECT 9-53

PROPERTIES OF FOAMED ASPHALTS FOR WARM MIX ASPHALT APPLICATIONS

INTERIM REPORT
PART I
LITERATURE REVIEW

Prepared for
Transportation Research Board
Of
The National Academies

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October 2012

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1.0 INTRODUCTION

1.1 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 aggregate and increased compaction requirements have resulted in increased mixing temperatures up to a limit of 350°F (177°C) where polymer breakdown in the binder can occur. The use of 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 which may be employed by the contractor.

Economic, environmental, and possible performance benefits motivate the reduction of HMA mixing and compaction temperatures. Past efforts that date back to the late 1950s include binder foaming processes (using either steam or water), asphalt emulsification, and incomplete aggregate drying (Kristjansdottir 2006; Zettler 2006). The latest technology is warm mix asphalt (WMA) where temperatures range from 175°F (79°C) to 295°F (146°C) and the following benefits have been cited (Newcomb 2005a; Koenders et al. 2002; Jones 2004; NCAT 2005, Newcomb 2005b; McKenzie 2006; Button et al. 2007):

- Decreased energy consumption of up to 30 to 40% (Jenkins et al. 2002; Kuennen 2004)
- Reduced emissions and odors at the plant (30% reduction in CO_2) (Kuennen 2004)
- Reduced fumes and improved working conditions at the construction site (fumes below detection limits and significant dust reduction) (Newcomb 2005a)
- Decreased plant wear and costs
- Extended haul distances, a longer pavement construction season, and a longer construction day if produced at typical HMA temperatures (NCAT 2005; Kristjansdottir 2006)
- Reduced construction time for pavements with multiple lifts (Kuennen 2004)
- Improved workability and compactability
- Reduced initial costs (in some cases)
- Reduced aging and subsequent susceptibility to cracking and raveling
- Decreased life cycle costs (in some cases)

There have been a number of products and processes introduced to the market place to produce WMA over the last seven years. These include waxes, surfactants, mineral additives, and mechanical foaming processes. Waxes, such as Fischer-Tropsch, montan, and polyolefin waxes, are high melt-point materials which become fluid at mixing and placement temperatures, and then harden at service temperatures. Surfactants reduce the surface tension of the liquid asphalt allowing it to better coat the aggregate at low temperatures and to remain workable at reduced placement temperatures.

The most popular method of producing WMA is by foaming the asphalt binder (Hansen and Newcomb, 2011). Zeolite, a mineral additive, has a small amount of water contained in its interstices which is released in the form of steam when the material comes in contact with the hot asphalt mix. The steam then foams the liquid asphalt, increasing its volume and allowing it to coat the aggregate at a lower temperature. The WAM foam process used in Norway precoats the

aggregate with a low viscosity asphalt and then adds a foamed hard asphalt. In the Low Emissions Asphalt (LEA) process used by McConnaughy in New York, coarse aggregate is dried and coated with asphalt, after which wet sand and an additive are introduced to create foam with the already heated asphalt. In current mechanical plant foaming processes in the U.S., cold water is injected into a hot asphalt stream that may be anywhere from 285 to 340°F (140 to 171°C). The amount of water used in producing the foam varies between one and three percent by weight of the binder. Foaming works in two ways to promote mixing at lower temperatures: 1) it increases the volume of the binder which makes it easier to coat particles, and 2) it reduces the overall viscosity of the binder through shear-thinning which makes the mix more workable (Fort et al, 2011). All of these processes and products work to allow asphalt mixing and placement temperatures to be reduced, but they introduce a new set of conditions that are not readily accounted for in the selection of materials and mixture design.

As discussed, the changes brought about by WMA in mixture components, mix 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 mixes and the physical characteristics required to meet performance expectations.

WMA production in the U.S. has increased exponentially in recent years from 19.2 million tons in 2009 to 47.6 million tons in 2010, a 148 percent increase in only one year (Hansen and Newcomb 2011). For the work in NCHRP 9-53, the focus will be on central plant produced foamed WMA. Foaming has become an increasingly important segment of the WMA market. According to a survey done by the National Asphalt Pavement Association (NAPA) (Hansen and Newcomb 2011), mechanical foaming units were responsible for about 83 percent of all WMA produced in 2009 and 92 percent of all WMA produced in 2010.

1.2 Scope

This report will document the history of asphalt foaming technology, its application, and traditional methods of ascertaining its effectiveness. Next, current used mechanisms for foaming asphalt will be presented in the context of plant produced asphalt mixtures. Asphalt foam characteristics, binder properties which affect foaming, and the effect of foaming on mixture characteristics will all be reviewed. Methods used in other industries to measure foam characteristics and their applicability to this study will be presented. Finally, based upon the review of the state-of-the-art and state-of-the-practice, a series of conclusions and recommendations will emerge to guide the remainder of this research.

2.0 TECHNIQUES FOR FOAMING ASPHALT

Most of the available literature dealing with foamed asphalt concerns its application in the stabilization of soils or base materials which began with Csanyi (1957). Although Csanyi used steam injected into the hot liquid asphalt, Mobil Oil Australia acquired the patent and modified it so that cold water was introduced into a stream of hot asphalt and then the foamed asphalt was mixed with cold, wet aggregate or soil (Muthen 1998). The desired outcome for stabilization was the coating of the fine particles by asphalt and the "spot welding" of the coarse aggregate to achieve some measure of cohesion. This type of stabilization is usually done in place but can also be accomplished through the use of a mixing plant (Muthen 1998). In recent years, this has been increasingly applied to in situ recycling of pulverized asphalt pavements (Fu 2011).

Fortunately, the process for WMA foaming, regardless of whether it is done mechanically through a nozzle, by introducing wet sand, or by using zeolite, it all amounts to adding a small

quantity of water to the hot asphalt binder (1 to 3 percent by weight) (Fort et al 2011) and allowing the generation of steam to expand the binder through the formation of voids. Thus, lessons learned in base and soil stabilization should apply to WMA production using foam.

2.1 Types of Mechanical Foaming Processes

There are a variety of methods to disperse a foaming agent such as water into a medium such as liquid asphalt. In all cases to date, water in a liquid state is introduced to the hot asphalt stream wherein it turns to steam. Mechanical systems which could be used for foaming and applied in the field have been identified by the authors as mechanical mixing, venturi mixing, expansion chamber, shear/colloid mixer, air atomized water, and high-pressure atomized water. In some instances, more than one mechanism may be employed or the system actually uses a hybrid of methods. The commercially available laboratory units use either air atomized water or pressurized water in an expansion chamber.

Mechanical Mixing (Figure 1)

Systems using mechanical mixing have an inlet port for introducing water into the asphalt stream. Downstream of the water introduction, the system may have paddles, baffles or other means of mixing the water/asphalt mixture. In this approach, some dispersion of the water into the asphalt occurs as the cold water turns to steam and creates bubbles followed by additional agitation which may serve to more finely divide the bubbles and enhance the foaming action.

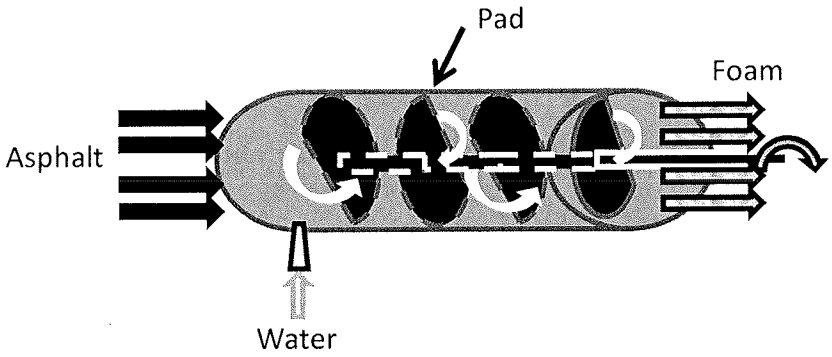

Figure 1. Mechanical Mixing

Venturi Mixing (Figure 2)

Venturi mixing is accomplished by introducing the water into the asphalt line ahead of a constriction in the pipe. This reduced cross-section increases the pressure in the line which is released as the cross-section opens. This creates turbulence in the fluid which acts to mix the steam and liquid asphalt.

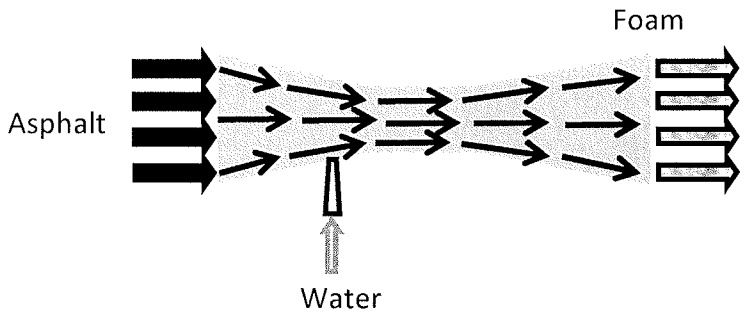

Figure 2. Venturi Mixing.

Expansion Chamber (Figure 3)

In an expansion chamber, asphalt and water are introduced simultaneously. The cold water comes into contact with the hot asphalt, converts to steam and expansion of the asphalt occurs. The foam is then forced out of a nozzle and into the mix. Expansion chambers can be configured in a manifold system where several are placed in parallel.

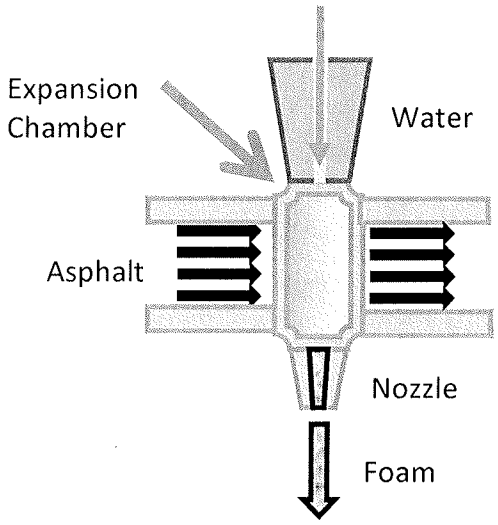

Figure 3. Expansion Chamber.

Shear/Colloid Mill (Figure 4)

Shear/colloid mills are used to mix substances such as asphalt and water. However, it is normally used to suspend asphalt in water for asphalt emulsions. In this case, cold water is introduced into a chamber with hot asphalt. The water turns to steam and is forced with the asphalt through a very small opening between a rotor and a stator which shears the water into very small particles. As the mixture exits the colloid mill, it expands and is introduced into the mix.

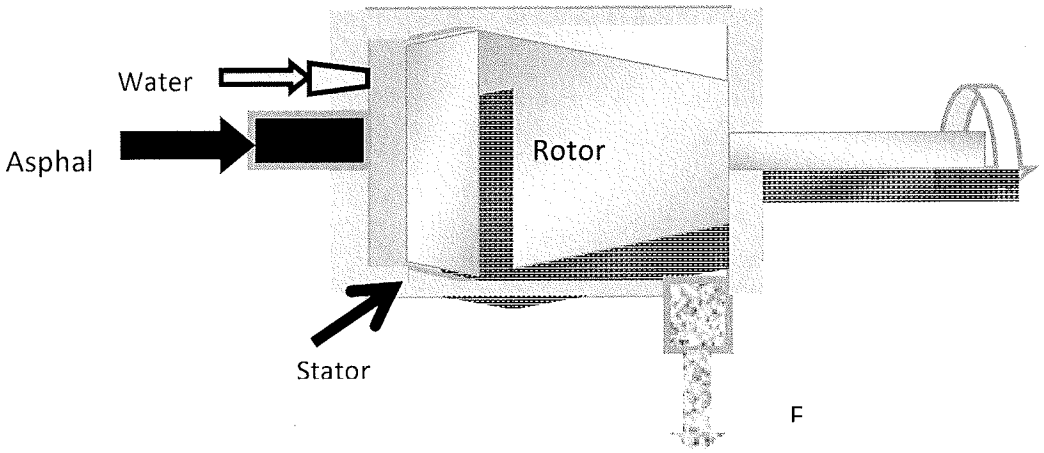

Figure 4. Shear/Colloid Mill.

Air Atomized Water(Figure 5)

The use of air atomized water is a variation on the expansion chamber discussed above. In this case, a stream of air is forced into the stream of water to break it into finer droplets. This should disperse the moisture throughout the asphalt stream and as the moisture comes into contact with the hot asphalt, it expands. The chamber allows for the expansion of the foamed asphalt which is then forced out through a nozzle into the mixture.

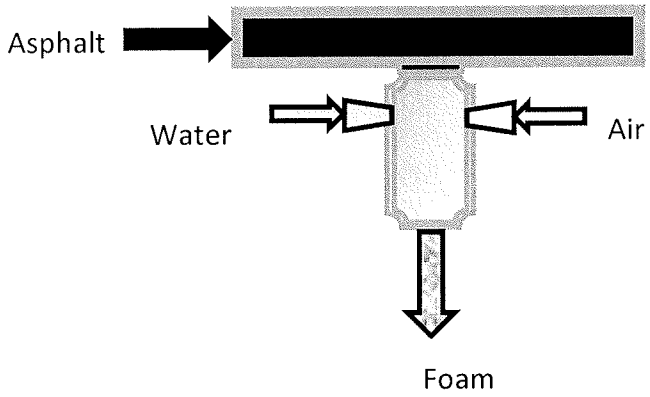

Figure 5. Air Atomized Water.

High Pressure Atomized Water (Figure 6)

In this type of system, water is forced through a very small orifice under very high pressure into the asphalt line. At this point the asphalt line may be enlarged to accommodate the increased volume or its cross-sectional area may remain the same in which case the pressure is considerably increased.

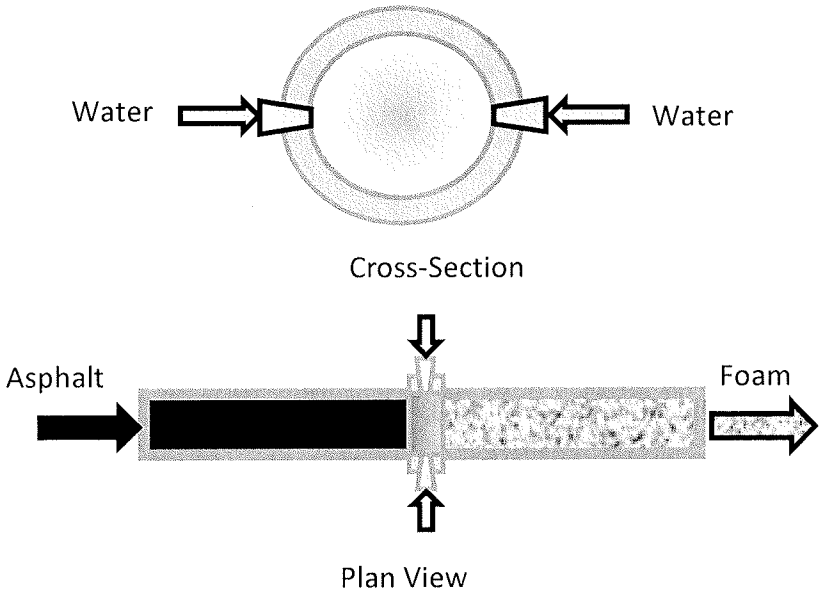

Figure 6. High Pressure Atomized Water.

A survey has been sent to manufacturers of asphalt plant foaming equipment to gain an understanding of what types of systems exist and their operating characteristics.

3.0 ASPHALT FOAMING CHARACTERISTICS

The asphalt literature reviewed to date shows that most investigators have relied on the expansion ratio (ER) and half-life (HL) as meaningful foam properties (Muthen 1998; Fu 2011; Englebrecht 1999; Bowering and Martin 1976; Brennen et al 1983; Abel 1978). The expansion ratio (ER) is defined as:

$$ER = V_{max}/V_{orig.} \quad (1)$$

Where:

V_{max} = Volume of the binder after foaming

$V_{orig.}$ = Original volume of binder

The half-life (HL) is computed as:

$$HL = t_2 - t_1 \quad (2)$$

Where: t_2 = time to a point where the foam is one-half of its original volume

t_1 = time when foam is at its maximum volume

Figure 7 shows these properties relative to the generation of foam. Fu (2011) points out that the traditional means of measuring half-life is from the point in time of maximum foam volume to half foam volume, and that anticipating the point of maximum foaming becomes problematic. He suggests that an alternative definition would be to start the timing at the point of water cut-off (t_0). This would likely result in a more repeatable half-life measurement, although the parameter should be renamed. Table 1 shows a comparison of ER and HL values published or proposed by various authors. From this list, it seems that a maximum for observed ER is 20:1, and that a ratio on the order of 10:1 is generally recommended. The HL values are quite variable, but a requirement of 60 sec. may be excessive. For WMA applications, it is important for the ER to be large enough to ensure the aggregate coating desired, and for the life of the foam to be long enough to ensure that the workability of the mix remains through the placement.

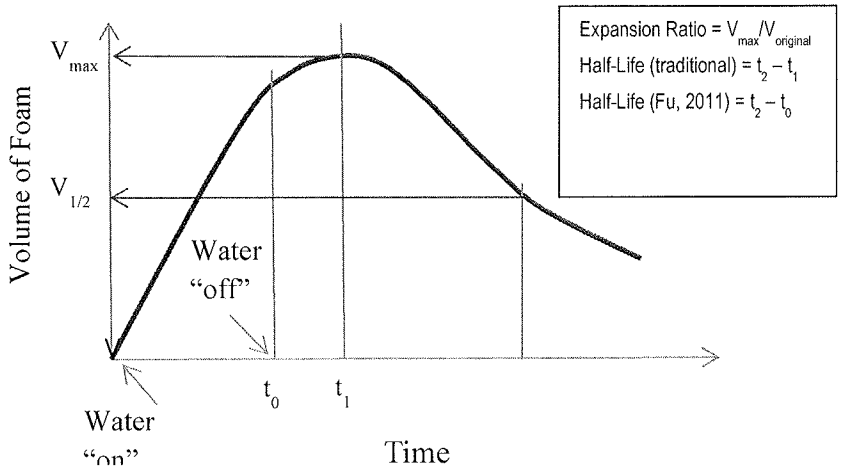

Figure 7. Graphical Depiction of Expansion Ratio and Half-Life (after Fu 2011).

Table 1. Expansion Ratio and Half-Life Values Reported in the Literature.

Reference	Expansion Ratio	Half-Life, sec.
Bowering and Martin 1976	15:1	---
Engelbrecht 1999	---	60
Muthen 1998	10:1	12
Ruckle et al. 1983	8:1 to 10:1	20
Fu et al. 2011	Range: 6:1 to 20:1	Range: 6 to 30

4.0 EFFECTS OF ASPHALT CHARACTERISTICS ON FOAMING

Factors that affect the ability of an asphalt binder to foam include:

1) The presence of anti-foaming agents (Abel 1978; Fu et al 2011),
2) The presence of anti-stripping agents (Abel 1978),
3) The high-temperature viscosity of the binder (Abel 1978),
4) The quantity of water (Brennen et al 1983; Ruckle et al 1983; Fu et al 2011) and
5) The temperature of the binder (Abel 1978; Ruckle et al 1983; Fu et al 2011).

Notwithstanding these factors, Castedo and Wood (1983) maintain that with the proper combination of nozzle type, water, air, and liquid asphalt pressure, any asphalt binder can be foamed.

Silicone anti-foaming agents are sometimes found in asphalts, and may be there as a result of the crude refining or asphalt production process. These additives do not show up under the standard physical testing done to grade asphalts, and their impact on the potential for intentionally foaming asphalt for warm mix may be quite significant (Fu et al 2011). While these will inhibit the ability of asphalt to foam, they are easily detected through quick chemical tests such as Fourier transform infrared (FTIR) spectroscopy wherein silicones show strong absorption in the 1100-1000 and 700 cm^{-1} range (Kekevi et al. 2011).

Many of the anti-stripping agents used in the production of asphalt mixtures are surfactants. Abel (1978) found that the presence of anti-stripping agents may improve the foaming capability of an asphalt. In fact Engelbrecht (1999) indicated they should be considered

part of the foaming system when he described foamed asphalt as "98 percent asphalt binder, one percent water, and one percent anti-stripping agent." He further suggested that lime be incorporated with the aggregate to facilitate mixing.

Lower viscosity asphalt binders have greater expansion ratios and half-lives than higher viscosity binders, but higher viscosity binders produce better coating (Abel 1978). Engelbrecht (1999) called for a 150/200 pen grade asphalt to be used in foaming. Although it would be expected to have a lower high-temperature viscosity, the pen grading is actually performed at 77°F (25°C) and may not be indicative of high temperature behavior. In seemingly contradictory findings, Fu et al (2011) and Lee (1981) stated that asphalts of the same grade may have markedly different potentials for foaming, i.e., the grade of asphalt is not indicative of how well an asphalt will foam, and asphalt foaming conditions need to be optimized for a given asphalt (Fu 2011). These apparently contradictory findings are resolved by a better understanding of the relationship between material properties and foaming process. These relationships are briefly discussed below.

The temperature of the binder at the time of foaming will have an impact on the ability to foam it. If the temperature is too high, then the viscosity may be too low and there will be an insufficient amount of surface tension available to form the bubble structure. Whereas, if the temperature is too low, the viscosity will be too high and the steam will not be able to distribute itself throughout the binder. Able (1978) reported that foaming was at an acceptable level when the temperature of the binder was greater than 149°F (65°C) while Engelbrecht (1999) called for a much higher temperature of 356°F (180°C). Ruckle et al (1983) reported that higher temperatures led to a greater ER but lower HL.

There are a number of factors that influence the foamability of an asphalt binder that include but are not limited to:

- surface tension of the asphalt binder (which is governed by the chemical composition of the crude oil and processing of the crude oil to produce the asphalt binder including chemical and polymer modifications),
- temperature of the asphalt binder,
- viscosity of the asphalt binder (which is governed by the same factors as surface tension)
- water content used to produce foam,
- size and dispersion of the water droplets introduced in the asphalt binder (which is governed by the characteristics of the nozzle delivering the water),
- quality of the water and the presence of any additives in the water that influence its surface tension,
- presence of anti-foaming agents in the asphalt binder, and
- use of foam promoters or other additives in the asphalt binder and the concentration of such additives.

5.0 EFFECTS OF ASPHALT FOAMING ON MIXTURE CHARACTERISTICS

In WMA asphalt foaming, water is used as a physical blowing agent. As the water comes into contact with the hot asphalt, it turns into steam and expands creating a void in the liquid and turning it into foam. In an open environment, steam expands water to 1673 times its liquid volume. In liquid asphalt, this expansion of water could translate to an overall expansion of the binder between 5 and 20 percent, generally. In current mechanical plant foaming processes, one to three percent cold water is injected into a hot asphalt stream that may be anywhere from 285 to 340°F (140 to 171°C), just ahead of mixing the binder with aggregate. Water may also be

introduced through zeolite by adding it to the mix just ahead of the liquid asphalt or through the addition of wet sand to hot, asphalt-coated coarse aggregate. The quantity of water used could be an important component of achieving an optimum foam. As stated by Ruckle et al (1983), as the quantity of water increases, the ER increases and the HL decreases.

There are numerous views on the importance of foam properties on the final mixture properties and characteristics. Bowering and Martin (1976) maintained that the cohesion and compressive strength of foam-produced mixes were greater when the ER was on the order of 15:1. Fu et al (2011) reported that small changes in asphalt temperature and foaming moisture can significantly change the characteristics of foamed asphalt while only having minimal effects on the final mixture properties. However, there was a correlation between the foaming characteristics and the dispersion of asphalt in mixtures.

The improved workability of foamed asphalt in WMA applications has been attributed to its shear thinning characteristics by Fort et al (2011). They claim this helps to maintain a greater film thickness without draindown during storage and transport, yet helps the workability during paving and compaction. However, Clark and Rorrer (2011) noted that challenges with foamed asphalt included difficulty with handwork and temperature segregation, but that lower temperatures allowed for compaction of mixes sooner and for improved density.

Clark and Rorrer (2011) found that in Virginia, typical mixture temperature reductions achieved with foaming systems was between 25 and 50°F (14 and 28°C), but this depended upon a number factors including weather conditions, type of paving job, haul distance, and moisture content of the aggregate. It was common for the temperature of the mix to be increased if the paving required a significant amount of handwork, and for temperature segregation to be addressed by the use of a material transfer device. In the construction of I-55 and I-57 in Missouri, Fort et al (2011) found that the use of a polymer modified binder (PG76-22) required that the average temperature of HMA production needed to be at 350°F (175°C) while the WMA average temperature was 293°F (145°C). The compaction temperature range was 320 to 230°F (160 to 110°C) and 275 to 212°F (135 to 100°C) for HMA and WMA, respectively, and the density of the final mat was 94 percent for both (Fort et al 2011).

The formation of steam bubbles within the liquid asphalt increases the volume of binder and foam reduces mass viscosity from that of the liquid at the same temperature, but how long does it last? External work in the form of shearing may destroy some or all of the bubbles. External work includes mixing, dropping into the slat conveyor, the silo, and the truck, dumping from truck to into the paver, movement through the paver to spreader auger, spreader auger to screed, and finally the compactor. At the same time, the mix is cooling, and as this progresses, the volume of steam contracts to the point of liquid, below boiling point. Ideally, the mix would remain at a relatively constant level of workability up through its compaction at which point the asphalt mat would assume its final density and stiffness.

Hajj et al (2011) stated that Marshall mixture and volumetric properties for an asphalt mix that contained an unmodified binder were met if the lab compaction occurred within four hours of plant production. They also observed that the effect of foaming in a WMA project is lost somewhere between four and 15 hours after short term oven aging at 250°F (121° C). They reported problems in meeting Marshall mix design requirements when polymer-modified foamed WMA was lab compacted at the project placement temperature of 255°F (124°C), and that these could only be obtained when the compaction temperature was raised to 305°F (152°C). The temperature observations agree with Fort et al (2011) who suggested that polymer modified binders may require an elevated temperature. However, the time that the foaming characteristics

are reported to last in Hajj et al (2011) and also noted by Prowell et al (2011) is far greater than the matter of minutes reported by Muthen (1998).

For WMA applications, Allosta et al (2011) found that the characteristics of asphalt mixtures produced by foaming were not very distinguishable from HMA. Nazzal et al (2011) reported similar results for a foamed asphalt project in Ohio, except that the foamed mix seemed to have a greater tendency for rutting in the Asphalt Pavement Analyzer. They speculated that the reason for the increased rutting may have been due to the presence of natural sand in the mix. This agrees with observations by Sakr and Manke (1985). Fu et al (2011) stated that asphalt temperature and foaming moisture can significantly change the foaming characteristics of the asphalt while only having minimal effects on the final mixture properties. However, they did find a correlation between the foaming characteristics and the dispersion of asphalt in mixtures.

Heating zeolite is a means of moisture release for foaming asphalt, but researchers at TTI have identified and are investigating other additives that can perform a similar function. When an additive such as zeolite is used to provide moisture release, one must consider the potential residual impact of the additive once the moisture release has taken place. The release rates, moisture release quantities and temperatures over which the moisture release takes place will be important to this project. It should be determined if a gradual release of moisture that extends beyond the period required for improved workability during construction exists and, if so, whether this extended release compromises the "early" performance of the mixture. TTI has developed moisture diffusion models on several length scales (mixture, mastic and binder) that can be used as a virtual analysis of how residual moisture might impact performance. Coupled with these diffusion models, TTI researchers are able to quantify the degradation in bond strength between asphalt mastic and aggregate that can result in adhesive failure or stripping within the mixture. The research team also has developed methods to track moisture release over temperature changes using the differential scanning calorimeter for a number of additives and have identified significant variation in the moisture release spectra among these. The impact of the "dehydrated" particles on the asphalt binder after the release has taken place, and how other physic-chemical interactions between particles and the binder impact performance are being addressed at TTI. Although one would not generally expect a significant impact between the additive and the binder due to the very small quantities involved, it is possible that this effect might be significant if the additive degrades under the influence of the binder and releases constituents from the matrix structure that might interfere with performance. An example would be the release of monovalent cations that could react with acids in the binder to produce water soluble organic salts that could impact durability of the mixture. TTI researchers have experience in investigating these phenomena as well as remedies to address these.

6.0 FOAMING IN OTHER INDUSTRIES

This section reviews the literature relevant to foaming in other industries. This review was conducted with the following objectives.

- Review the metrics that are commonly used to characterize foam and foamability of different liquids in other industries (Section 6.2).
- Review the experimental methods and techniques that are used to obtain the aforementioned metrics (Section 6.3).
- Develop a better understanding of the foaming mechanisms and the material properties that influence foaming from the work that has already been accomplished in other disciplines (Section 6.4).

6.1. Definitions

It is important to identify a few common terms used in the industry and literature to characterize foams. The term foamability is often used to describe the extent to which a liquid can be foamed, both in terms of volume and stability. Foamability is a property of the liquid, although it is governed by external factors such as gas concentration and temperature. There are types of foams prepared by processes not involving the direct dispersion of a gas in a liquid phase (Klempner et al. 2004). These may be prepared by the leaching of a fugitive phase such as a water-soluble salt, sintering small particles dispersed in a heat-stable matrix, fusing initially discrete polymer particles that initially entrap air or other gases, and forming a polymer matrix around hollow spheres. These processes do not follow the same steps of gas dispersion, bubble growth, and stabilization, as found in the asphalt foaming process, and thus are not considered in this review.

The terms "foam" and "froth" are used interchangeably. However, the term "foam" is used to describe a two component system comprising of a gas and a liquid (typically) which when broken down leaves a homogenous liquid phase (Pugh 2005). In some cases, foam is also regarded as an emulsion of a liquid and a gas. The term froth is used to described a three component system, typically a gas, fluid and a solid which when broken down results in a two component composite.

Not all foams are the same. Foams are typically classified and studied in two different categories (Adamson et al. 1997; Pugh 2005):

- Polyederschaum (polyhedral foam): In this type of foam the volume of gas is much larger as compared to the volume of the fluid. The fluid exists in the form of very thin films separating the gas also referred to as lamella. The name is derived from the fact that the gas cells are polyhedral in shape (Adamson et al. 1997).
- Kugelschaum (spheroidal foam): In this type of foam the gas volume is relatively lower compared to the Polyederschaum. A relatively thicker film of the fluid separates the gas bubbles.

Figure 8 is a schematic that illustrates the two different types of foam. These two types of foam could coexist in the same material, given the highly dynamic process of foaming. Bubbles could start isolated and then impinge on each other and form polyhedral foam as they grow. In the context of foamed asphalt mixtures, the latter (Kugelschaum) type of foam appears to be more relevant as the bubbles would emerge to the surface and collapse before a thin wall between bubbles could be formed

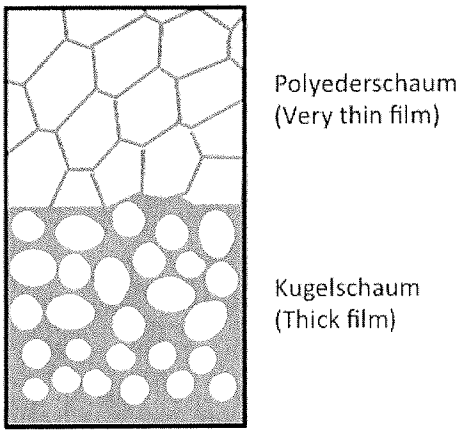

Figure 8. Schematic to illustrate the two different types of foam (Adapted from Pugh 2005).

6.2. Foaming and Metrics to Characterize Foam

Foaming in other industries

There are a number of industries that require the design and use of foamed materials. The applications range from food, pharmaceutical and health care products, polymers and in some cases even metals (Koehler et al. 1998). Several different methods are used to produce foam in the food industry including whipping, injection, sparging (bubbling a gas through a liquid) and shaking (German et al. 1985). The characteristics of the foam produced using each one of these methods are very different. There are several different variables associated with each of these four methods that can result in the production of foams with very different characteristics.

Arzhavitina and Steckel (2010) present a detailed review of the different types of foams typically used in the pharmaceutical sector. One of the types of foam that is commonly used is the pressurized aerosol foam. Aerosol foam is produced by using a propellant vapor to drive the liquid through a nozzle from a storage can.

In the packaging industry, foaming is typically achieved using a blowing agent, which upon controlled energy input (e.g. thermal) causes the desired level of foaming of the polymer. In a typical foaming process, the blowing agents gasify through chemical reaction or thermal decomposition under the foaming process conditions. The blowing agent is mixed with pelletized polymer and loaded into an extruder. It decomposes in the extruder barrel at an elevated temperature, resulting in gas formation in the polymer melt. The gas formed in the polymer melt diffuses out, leading to bubble nucleation and growth. Upon exiting the extruder the polymer profile expands in volume because of the bubble growth. In addition to chemical reaction and thermal decomposition, gas can be delivered to the extruder through either pre-saturation of the polymer pellets or high pressure injection right into the polymer melt inside the extruder. Foams produced this way have a smaller bubble size and a more uniform foam structures.

Metrics to characterize foam in other industries

The two most common attributes that are used to characterize foams across different industries are the volume and stability of the foam. In fact, as it will be evident in the summary that follows, these two attributes and concomitant metrics are similar to what is currently being used to characterize asphalt foaming: expansion ratio and half-life.

Foamability of wines is an important characteristic that is measured to select base wines for the production of sparkling wines. Over the past few years, researchers in this area have developed two different sets of parameters to characterize the foamability of different wines. The first set has three different parameters: (i) HM or *maximum height* reached by the foam after carbon dioxide injection for a specific interval of time, (ii) HS or *foam stability height* during carbon dioxide injection, and (iii) TS or the *foam stability time* identified as the time it takes for all the bubbles to collapse after carbon dioxide injection has stopped. The second set has two different parameters: (i) E or *foam expansion* which is very similar to the expansion ratio used to characterize expansion of asphalt binders and (ii) Lf or *foam stability*, which is broadly defined as the area under the foam height versus time curve following the peak expansion divided by the maximum height of the foam. These two parameters are sometimes also accompanied with a characteristic bubble size. Andres-Lacueva et al. (1996) demonstrated that in the first set of parameters, HM and HS were strongly correlated and therefore only HM and TS were adequate to describe the foamability of different wines. Gallart et al. (1997) compared the relative benefits between the two sets of parameters and reported that the precision of each set of parameters was dependent on the extent to which the wine foamed; the second set being more appropriate for low foaming wines.

Food products are routinely characterized to determine the influence of factors such as production method and ingredients (proteins in particular) on its foaming characteristics. The two parameters that are often used in the food industry are (i) *foam overrun* and (ii) *foam stability*. Foam overrun is roughly a measure of the volume of foam produced. For example, Phillips et al. (1987) defined it as the difference between the weights of 100 mL of protein and 100 mL of foam divided by the weight of 100 mL of the foam. The measurements were made at a specific time after foaming the subject material. Foam stability was defined as the ability of the foam to resist rupture or collapse under the influence of gravity. In more recent studies, Raymundo et al. (1998) tried to modify these two metrics to come up with a single parameter referred to as the *foaming index*. In this case, overrun was measured not at a pre-specified point in time but continually as a function of time using automated equipment. The integral of the overrun with respect to time was used as single parameter referred to as the foaming index. Raymundo et al. (1998) tried to demonstrate the sensitivity and advantages of using this single parameter to characterize the foam.

In pharmaceutical applications, two metrics that are used to describe the quality of the foam are *foam density* and *breakability* (Arzhavitina et al. 2010). Foam density is simply the relative density of the foam with respect to water. Stability of the foam is characterized in terms of three different types of breaking behavior: (i) quick breaking foams or foams that are thermally unstable and collapse on contact with skin, (ii) lathers or stable foams that demonstrate a tendency to increase in volume when subjected to shearing action and (iii) breakable foams or foams that are stable at skin temperatures but collapse and spread easily upon application of shear forces. Other metrics and methods, similar to those used in the food sector are also used to describe the foam stability.

In addition to foam density and stability, *rheology of the foam* is also considered of importance in pharmaceutical applications. However, there are several challenges associated with measuring the rheological properties of a foam, even when the foam is relatively stable compared to foamed asphalt. Kealy et al. (2008) conducted a detailed study on the methods that can be used to measure the rheological properties of several different types of pharmaceutical foams. They cited several difficulties associated with the use of standardized methods, such as

the Dynamic Shear Rheometer (DSR), to measure the rheological properties of the foam. For example, they report that the small gaps in typical test geometries (1 to 2 mm with parallel plate or a few micrometers with the cone and plate geometries) are inappropriate because of the presence of large bubbles in the foam. Even if the bubble size were small, there are problems associated with slippage between the foam and the end plates. In their study they tried to overcome these limitations by using larger gaps and serrated plates. They also used vane geometry to measure the rheological properties. They were able to measure the yield stress for the different foams by applying a monotonically increasing shear stress until the specimen failed or started to flow (Figure 9). They also measured the complex shear modulus of different foams in the frequency domain and demonstrated that at low frequencies the rheological properties measured using the vane geometry was similar to the properties measured using the serrated parallel plate. In the context of foamed asphalt, this is an important finding because it may be possible to use geometry similar to the vane (e.g. a portable paddle viscometer) to obtain real time viscosity and rheological measurements of the foam as it decays.

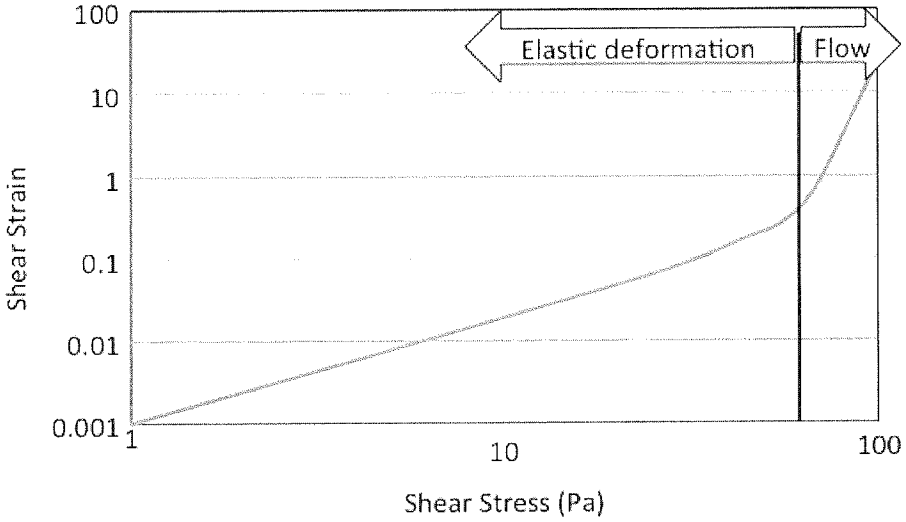

Figure 9. Typical results for the yield strength of a shaving foam measured (Adapted from Kealy 2008).

One other characterization procedure that is worth mentioning is foam drainage. Drainage tests are frequently conducted to evaluate the structure and stability of different kinds of foam (Pugh 2005). There are several different methods that are used to drain foams to determine its characteristics. Koehler et al. (1998) identify four different methods of draining foams: (i) free drainage under gravity, (ii) wetting of the foam by placing the foam in contact with a liquid bath, (iii) forced drainage where a constant liquid supply is pumped onto the top of the foam, and (iv) response of the foam to an external pulse. Parameters that are measured in a foam drainage experiment may include the rate at which the liquid flows out of the foam and changes in pressure across the boundaries of the foam. Measurements are usually carried out using optical or electrical based instrumentation attached to a column designed to create and drain foams (discussed in the following section). Koehler et al. (1998) present the analytical solutions that describe the rate at which foams drain as a function of the properties of the foam. However, based on the literature it appears that the drainage method is more appropriate to characterize the polyederschaum category of foams (foams with very thin films of the fluid separating the gas).

A more detailed study of the drainage tests may be useful for future studies related to the foaming technology.

In summary, the current approach to characterize asphalt foams based on expansion ratio and half-life are generally consistent with the research and practice related to foaming in other industries. However, based on this review, the following factors will be considered in this research project.

- We will attempt to obtain time history measurements of the foamed asphalt sample as it expands (after being dispensed in the container) and collapses. Section 6.3. presents a review of the different methods that are currently being used to obtain such measurements in other industries. Appendix A presents a more detailed discussion of the potential methods that can be used to characterize asphalt foams.
- We will evaluate the use of metrics other than expansion ratio and half-life to characterize the foamability of different asphalt binders. For example, a foaming index that is based on the area under the foam volume versus time curve.
- We will attempt to directly or indirectly measure the rheological properties or viscosity of the foamed asphalt binder as the foam decays. For example, it may be possible to use a portable or a variation of a paddle viscometer to measure the change in the shear resistance of the foamed binder at low shear rates as the foam decays. Considerations for the shear rate, which may interfere with the foam to promote or retard the foam decay, and time sensitive nature of the foam will have to be made in such measurements.

6.3. Methods to Measure Foam Formation and Decay

In the context of foamed asphalt binders, it is important to determine the optimum combination of water content, temperature and foam promoting or other additives that will result in desired levels of foaming. In order to accomplish this, it is important to have a reliable and repeatable method by which to measure the foamability of different asphalt binders under different conditions in a laboratory and/or field environment. Several different methods have been used to characterize foaming in other industries. A review of these methods along with the potential for being used to determine the foamability of asphalt binders is presented below.

Volumetric methods

The basic principle of using a volumetric method is to measure the change in the volume of a given mass of the material over time as it foams and as the foam decays. Examples of direct volumetric measurements method are the Expansion Ratio and Half-Life approaches currently used in asphalt industry. In this case, a graduated dipstick is used along with a stopwatch. The foamed binder is dispensed into a container and the maximum height of the foam along with the time it takes for the foamed binder to decay to different heights is recorded (Jennings et al. 1993). Although these are direct measurements of the foam volume and longevity they are susceptible to human errors and have low precision on account of the dynamic surface of the foamed asphalt binder. A more accurate variation of this method involves measurement of pressure difference across two cells (one of which carries the foamed material) to characterize the foam (Nishioka et al. 1981).

An indirect method to measure volume change is to measure the density of a representative volume for a given mass of the foam. Typical liquid density meters use very small volumes of liquid that are in the order of a few milliliters. In order to use a density meter to characterize

foam, it is important to have a density gauge or meter that is suitable for measurement of larger sample volumes. This is discussed further in Appendix A.

Ultrasonic method

The ultrasonic method is also an indirect method to measure the density of the foamed material. This technique relies on the use of ultrasonic waves passing through a cross section of the sample to characterize the foam in real time. Figure 10 shows an example of the experimental results from a study conducted by Piché et al. (1999) on extrusion foaming of polystyrene with a blowing agent HCFC-142b. The pressure of molten polystyrene was measured in the extrusion die. Prior to bubble nucleation, the sound velocity and attenuation are identical to the pre-foamed material. However, at the onset of bubble nucleation, the sound velocity begins to decrease while the attenuation begins to increase. The decrease in velocity is related to the change in the material properties. The increase in attenuation is due to acoustic scattering from nucleated bubbles. Therefore, by continually measuring the attenuation of an ultrasonic signal being transmitted through a volume of a foamed material, it is possible to determine the rate of decay of foam within the material.

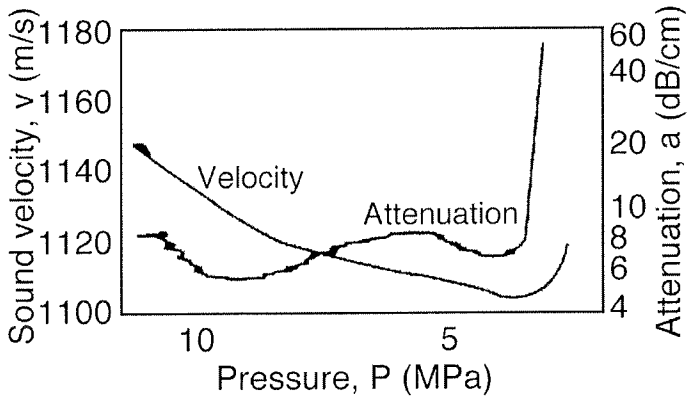

Figure 10. The sound velocity and attenuation of polystyrene during the foaming process. (Piche et al. 1999).

St. John invented an ultrasonic bubble detection apparatus to detect bubbles in flow tubing situated in a channel (Peter 1983). Ultrasonic transmitter and receiver were positioned on opposite sides of the channel. When air bubbles passed through the ultrasound transmission path, a major change in received signal was detected due to the strong coupling effect between the ultrasound and air bubbles. The application of this apparatus can be in many fields under circumstances where visual or optical detection techniques are either undesired or infeasible. In particular, this invention is targeted on automated processing of blood, blood components, or parenteral solutions, e.g., the pumped administration of parenteral solutions to a patient. A bubble detector may be needed to automatically shut off the device when a bubble is detected.

Optical methods

The optical methods typically involve the use of a microscope or other optical methods to examine the foam in real time (Wilde 1996). This method is widely used in the food and beverage industry to characterize foamability of different products. For example, one of the devices / techniques that is commonly used to characterize foaming in beverages is the Mosalux

procedure (Andrés-Lacueva et al. 1996). This procedure uses a photoelectric cell to measure the height of the foam in a standard cylinder. Various automated versions of this procedure exist and various metrics pertaining to foamability derived using this procedure are very repeatable with coefficients of variation between 4 to 9 percent (Andrés-Lacueva et al. 1996).

Optical methods have also been used to determine the spatial and size distribution of bubbles in foamed liquids. For example, Du et al. (2001) used a capillary tube with a number of photoelectric cells placed along the length of the tube at fixed distances to determine the size distribution of bubbles in a foam. As the capillary tube is inserted into the foam, the bubbles are sucked into the capillary tube and take the shape of a slug that moves at a constant velocity. The photoelectric sensors (each sensor is an emitter-receiver pair) on the tube measure the speed of the slug as suction is applied on the other end of the capillary tube. According to Du et al. (2001) the speed and dimensions of several hundred bubbles can be measured in a few seconds. However, corrections need to be applied to transform the measured "slug" dimensions to the dimensions of the bubble through which the tube passes. These corrections are obtained either theoretically based on the pressure gradient inside the tube or experimentally via calibration. By changing the vertical location of the submerged end of the capillary tube within the liquid and foam, the spatial distribution of the bubble size can also be measured. They quantified the change in the bubble size distribution as a function of the depth in a given volume of the foam (the foam was being produced by injecting gas from the bottom of the column).

Although optical methods (via sensors or direct image capture and analysis) offer one of the more direct ways to examine foam evolution and decay, these methods cannot be used directly within foamed asphalt on account of its opacity. However, the research team will attempt to use digital images of the foamed surface, processed through image recognition software to capture bubble size and distribution.

Electrical method

Kato et al. (1983) highlighted that despite all the studies that have tried to relate the influence of proteins and other factors on the foamability of different food materials, the number of factors (e.g. pH, temperature, foaming method, protein type and concentration) and their combinations that affect foamability are far too many to allow reasonable theoretical predictions. Therefore, they emphasized the importance of having a standardized method to measure the foaming characteristics of food materials.

Kato et al. (1983) developed and proposed the use of an electrical conductivity cell to measure the formation and decay of foam. The cell measures the electrical conductivity across two electrodes with the collapsing foam in between. The electrical pathway between the electrodes changes with the structure of the foam and bubbles and is used to characterize the foam. They demonstrated that their technique could be used to accurately measure the characteristics of different foams and foamability of different food products.

Sensor rods based on this principle are commercially available for use with products related to the food, pharmaceutical and cosmetics industry. The efficacy of this method depends on the electrical properties of the foaming agent and the material being foamed as well as the amount of the material that is being foamed that deposits on the sensor rods. Typically, combinations of optical and electrical methods are commonly used to measure foaming characteristics in the food and pharmaceutical industries (Hall 1996).

A variation of the above method that measures dielectric constant instead of electrical conductivity is more appropriate to characterize foaming in non-conducting media such as asphalt binders. In this technique, the dielectric constant of the foam or liquid is measured using

a probe. The dielectric constant is a function of the relative concentration of the materials surrounding the probe, i.e. the relative concentration of the gas and the fluid in the case of a foam. By measuring the changes in the dielectric constant, it is possible to estimate the extent of foaming in the material. A commercially available version of this probe is further described in Appendix A.

6.4. Mechanics of bubble formation

Over the past few years, research team member Dr. Li's group has conducted extensive research on polymer foaming with carbon dioxide (CO_2), where they have explicitly modeled and demonstrated the significance of the relationship between material properties and the foams (Wang, 2008; Kim and Li, 2010). In CO_2 polymer foaming, a few weight percent of CO_2 is injected into the polymer matrix. The bubble growth process can be modeled as a quasi-static diffusion-driven pressure and momentum balanced process in a viscoelastic fluid. Figure 11 shows a single bubble model for a polymer-gas system.

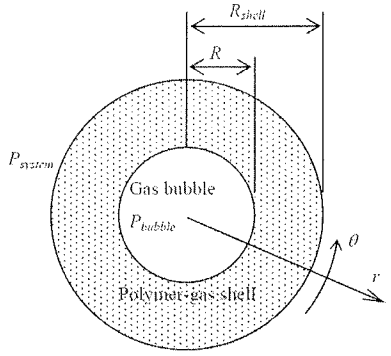

Figure 11. A schematic of the single bubble model for polymer-gas systems (Wang, 2008).

The momentum equation of the polymer shell surrounding the gas bubble can be expressed as

$$P_{bubble} - P_{system} - \frac{2\gamma}{R} + 2\int_{R}^{R_{shell}} \frac{\tau_{rr} - \tau_{\theta\theta}}{r} dr = 0 \tag{3}$$

where, P_{bubble} is the gas pressure inside the bubble, P_{system} is the system pressure around the bubble, R is the bubble radius, R_{shell} is the bubble shell radius representing the polymer around the bubble, γ is the surface tension at the bubble surface, and τ_{rr} and $\tau_{\theta\theta}$ are the stress components in the shell along the r and θ directions, respectively.

The viscoelastic rheology of the polymer-CO_2 system can be modeled as

$$\frac{d\tau_{rr}}{dt} = -\left(\frac{1}{\lambda} + \frac{4R^2\dot{R}}{y+R^3}\right)\tau_{rr} - \frac{4\eta_o}{\lambda}\left(\frac{R^2\dot{R}}{y+R^3}\right)$$

$$\frac{d\tau_{\theta\theta}}{dt} = -\left(\frac{1}{\lambda} - \frac{2R^2\dot{R}}{y+R^3}\right)\tau_{\theta\theta} + \frac{2\eta_o}{\lambda}\left(\frac{R^2\dot{R}}{y+R^3}\right) \tag{4}$$

where, λ is the relaxation time of the polymer/gas solution, η_o is the zero-shear-rate viscosity of the polymer-gas system, and y is the transformed Lagrangian coordinate defined as

$$y = r^3 - R^3 \tag{5}$$

From the above equations the bubble pressure P_{bubble} can be calculated by assuming a known bubble size at a given time step. On the other hand, it is clear that the bubble growth process is driven by gas diffusion into the bubble. The mass transfer equation can be modeled as

$$\frac{\partial C}{\partial t} = \frac{D_G}{r^2}\frac{\partial}{\partial r}\left(r^2\frac{\partial C}{\partial r}\right) - \frac{\dot{R}R^2}{r^2}\frac{\partial C}{\partial r} \quad (6)$$

where C is the local gas concentration in the polymer matrix and D_G is the gas diffusivity.

The law of conservation of mass requires that the rate of change of the mass in the gas bubble be equal to the mass of gas diffusing into the bubble through the bubble surface. Thus, the bubble pressure can be related to the concentration gradient at the bubble surface by

$$\frac{d}{dt}\left(\frac{4\pi}{3}\frac{P_{bubble}R^3}{KT}\right) = 4\pi R^2 D\frac{\partial C}{\partial r}\bigg|_{r=R} \quad (7)$$

where K is the universal gas constant, T is the temperature that is estimated from the heat transfer model for the current time step. Using the above two equations the pressure inside the bubble P_{bubble} can be related to the bubble size r as well.

By comparing the bubble pressures P_{bubble} calculated from both the momentum and diffusion equations, the bubble size r at each time step can be determined using a recursive procedure. Such a procedure relates the bubble growth process with the material properties such as surface tension, relaxation time, viscosity, and gas diffusivity, as well as process parameters such as temperature, gas concentration, and the system pressure.

The knowledge obtained from CO_2 gas foaming of polymers can be readily extended to better understand the factors that affect the characteristics of water foaming of asphalt binders. The *amount of water carried by the binder* depends on the pressure of water and binder within the nozzle as well as the diffusivity of water or steam in the binder. While the pressure within the nozzle varies with the type of nozzle, the ability of the water to diffuse within the binder depends on the type of asphalt binder. The importance of these two variables (type of nozzle and binder) has been recognized in previous studies (Castedo et al. 1983). It is also expected that the *size and life of the bubbles (related to expansion ratio and half life)* within the binder coming out of the nozzle will depend on the type of the binder; more specifically surface tension of the binder surrounding the bubble and its viscosity.

This preliminary understanding of the relationship between material properties and foaming helps explain the findings from other previous studies. For example, (Fu et al. 2011) demonstrated that binder grade is not related to its ability to foam. This is expected because the binder grade does not reflect the material properties such as viscosity and surface tension that are related to foaming. Another example is that the addition of liquid anti-strip agents improves the ability of the binder to foam (Abel et al. 1979; Engelbrecht 1999; Fu et al. 2011). This is also expected because Bhasin et al. (2007) have demonstrated that the addition of liquid anti-strip agents reduces the surface tension of the binder.

7.0 SUMMARY AND CONCLUSIONS

Asphalt foaming for the purpose of coating aggregate with asphalt binder has been in use for about 45 years at the time of this writing. There are a number of means of generating asphalt foam including mechanical mixing, venture mixing, expansion chamber, shear/colloid mill, air atomization, and high-pressure water injection. These techniques may be used alone or in combination to produce foamed asphalt.

Factors that have been shown to influence the ability of asphalt to foam include the following:

- surface tension of the asphalt binder (governed by the chemical composition and processing of the crude oil to produce the asphalt binder),
- temperature of the asphalt binder,
- viscosity of the asphalt binder (which is governed by the same factors as surface tension)
- water content used to produce foam,
- size and dispersion of the water droplets introduced in the asphalt binder (which is governed by the characteristics of the nozzle delivering the water),
- quality of the water and the presence of any additives in the water that influence its surface tension,
- presence of anti-foaming agents in the asphalt binder, and
- use of foam promoters or other additives in the asphalt binder and the concentration of such additives.

Up to this point, methods for measuring asphalt foaming characteristics have been restricted to manual measurements for expansion ratio and half-life. Although general guidelines are given by some authors on the desirable ranges for these parameters, they are based upon the application of foaming to the stabilization of soils or in-situ aggregate bases. There is no indication that these guidelines are appropriate for plant mixed asphalt. Additionally, it is the authors' experience that manual measurements may not be very accurate or precise. At typical production temperatures, asphalt tends to expand and collapse very rapidly. Thus, new automated procedures for measuring foam properties are needed.

The ability to measure and model the foaming process in asphalt binders will allow users to optimize the foaming characteristics for any given asphalt binder (e.g. moisture content, temperature, foaming additive). In this context, optimal foaming characteristics refer to the expansion ratio and half-life (or other appropriate metric) that allow the binder to be coated onto the aggregate particles without adversely affecting the performance of the resulting asphalt mixture. A combination of binder and mixture tests are planned for execution in this project to achieve this objective.

Based upon the literature review presented here, it is clear that there are theoretical models capable of explaining the phenomenon of foaming within asphalt binders using steam as a blowing agent.

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