NCAT Report 20-01

LABORATORY AND FIELD CHARACTERIZATION OF WARM ASPHALT MIXTURES WITH HIGH RECLAIMED ASPHALT PAVEMENT CONTENTS IN ALABAMA

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1 INTRODUCTION

The Alabama Department of Transportation (ALDOT) and other highway agencies are interested in utilization of higher percentages of reclaimed asphalt pavement (RAP) in asphalt mixtures. There are a number of research studies at both state and national levels examining mix design and production issues related to high RAP content mixes. One of the mix design issues deals with the grade of virgin binder that should be used with RAP mixtures. The traditional approach has been to use a softer grade of virgin asphalt to minimize the stiffening of the aged binder in the RAP, specifically for RAP contents above 15 to 20 percent. A number of recent studies (shown in the following section) have indicated that softer binders may not be necessary, whereas other pavement engineers have suggested combining warm mix asphalt (WMA) technologies with higher RAP content mixes as a means of achieving more durable asphalt binder.

Most highway agencies have decades of experience with hot mix asphalt (HMA) containing low percentages of RAP (i.e., below 25% by weight of aggregate). Although there have been studies to directly compare the performance of virgin mixtures with mixtures containing RAP, there is a general perception that RAP mixtures may be more susceptible to various modes of cracking (i.e. fatigue, thermal, reflective). This is related to the fact that the aged RAP binder is stiffer and less strain tolerant in comparison to a virgin binder; as the RAP proportion increases, there is the potential for an increase in mixture stiffness and a decrease in resistance to cracking. Therefore, numerous recent research efforts have strived to increase the RAP percentage without sacrificing performance.

The principal concern of highway agencies is that a high percentage of RAP may significantly reduce the performance of the pavements, resulting in increased pavement rehabilitation costs. Therefore, before specifying high RAP percentages, agencies want assurance that high RAP mixes will provide satisfactory field performance.

1.1 Objective

This research study consisted of two primary objectives. The first objective was to conduct performance testing from plant produced high RAP-WMA mixtures. The second objective was to monitor the field performance of selected high RAP-WMA mixtures produced under the specification for a period of five years in order to determine if these pavements were durable.

1.2 Scope

It was anticipated that the study would involve at least six ALDOT high RAP-WMA construction projects constructed in 2009, and possibly as late as 2010. Projects were selected throughout the state based on availability. During the first lot and every 10,000 tons of high RAP-WMA mixture produced thereafter, contractors were required to supply recovered binder test results (viscosity and $|G^*|/\sin(\delta)$) and unconditioned indirect tensile strengths. NCAT assisted ALDOT in the analysis of this data and provided recommendations for adjustments to the high RAP-WMA mixtures as needed.

NCAT staff visited the projects during production of the high RAP-WMA mixtures on the selected projects to document production information and collect samples of the mixtures for testing at the main NCAT laboratory. NCAT performed indirect tensile strengths on each lot using contractor supplied gyratory specimens compacted to N_{design}. NCAT also collected loose mix samples for compaction and laboratory performance testing. In addition to running the QC/QA tests for the high RAP mixtures, simplified visco-elastic continuum damage (SVECD) testing, dynamic modulus testing, and semi-circular bending testing (LTRC and I-FIT) were performed on the mixtures sampled from the field.

At the end of the five-year monitoring period, cores were taken for material characterization and laboratory performance evaluation. Since ALDOT specifications only allow high RAP-WMA mixtures to be placed in the binder or base layers, surface measurements would not have allowed ALDOT to truly assess how the mixture was performing over time in the field.

2 BACKGROUND

RAP is an HMA mixture containing aggregates and asphalt cement binder that has been removed and reclaimed from an existing roadway. Once processed, RAP is incorporated as a recycled component in new HMA mixtures. In the 1970s, states and paving contractors began making extensive use of RAP in HMA pavements. The use of RAP results in cost savings and an environmentally positive method of recycling. Properly designed HMA containing RAP can perform as well as HMA prepared with 100 percent virgin materials (*1*).

Over the years, contractors generally stuck with having one RAP stockpile and feeding anywhere from 10% to 25% RAP into the mix. This produced significant cost savings, especially with respect to the asphalt binder, which is an expensive component in the mix. Higher amounts of RAP and the implementation of fractionated RAP stockpiles are now being considered, particularly as a response of the increasing oil prices.

Reports from the Federal Highway Administration (FHWA) and the United States Environmental Protection Agency state that more than 80 million tons of reclaimed asphalt pavements are recycled each year, and approximately 80 percent of removed asphalt pavements are reused as part of new roads, roadbeds, shoulders, and embankments *(2)*. In addition, more than 99 percent of asphalt pavement reclaimed from roads and parking lots was reclaimed for use in new pavements instead of going into landfills.

2.1 Challenging Aspects When Using High RAP Contents

One of the key issues with regard to RAP mix designs is how much blending occurs between the RAP binder and the virgin binder. One view of RAP blending has been that RAP simply acts as a black rock and does not blend with the virgin binder, therefore not contributing to bonding the aggregates together. The opposite view is that RAP binder completely blends with the virgin binder and that the composite binder has properties that can be estimated by proportionally combining the properties of the RAP binder and the virgin binder.

NCHRP 9-12 evaluated the RAP-virgin binder blending issue with an experiment that considered three scenarios of blending (*3*). In the first scenario, the black rock scenario was simulated by recovering RAP aggregate and blending it with virgin asphalt and aggregates. In the second scenario, RAP was mixed with virgin asphalt and aggregate (actual practice). In the third scenario, reclaimed asphalt was blended with the virgin binder (total blending). The specimens made for all three scenarios used the same gradation and total asphalt content. The laboratory experiment included three RAP materials with different recovered PG grades, two RAP percentages per RAP stiffness, and two virgin binders. Five mixture tests were used to evaluate the mixes for each scenario: frequency sweep at constant height, simple shear at constant height, repeated shear at constant height, indirect tensile creep, and indirect tensile strength. The test results revealed that the actual practice and the total blending scenarios were the most similar, thus indicating that there is blending of the reclaimed and virgin binder.

Huang et al. also evaluated the extent to which RAP binder is active in a new mix (*4*). In this study, fine RAP material (passing No. 4 sieve) was blended at 10%, 20%, and 30% with coarse virgin aggregate (retained on No. 4 sieve) to determine the extent of RAP binder transferred to the coarse aggregate. The virgin aggregate was heated to 190°C and the RAP was added at ambient temperature. The results indicated that approximately 11% of the RAP binder transferred to virgin aggregate during the mixing process. The researchers conceded that in real mixes that include virgin binder, some diffusion has been shown to occur between the RAP binder and virgin binder; this suggests that the percentage of RAP binder that will transfer will increase from 11% with time.

Bonaquist evaluated blending of virgin and recycled binders in mixtures containing RAP and recycled asphalt shingles (RAS) by comparing laboratory-measured dynamic shear moduli of recovered binders to predicted shear moduli using the Hirsch model (*5*). Plant-produced mixtures containing RAP and RAS were sampled, specimens were fabricated, and the dynamic moduli over a range of temperatures and frequencies for all mixtures were obtained. Next, the binders were extracted and recovered from the specimens. The recovered binders were tested in a DSR using a frequency sweep to determine the binder shear moduli, $|G^*|$. The measured shear moduli of the recovered binders (fully blended) were plotted with the predicted moduli from the Hirsch model. When predicted and measured master curves overlapped, it was inferred that the recycled and virgin binders in the plant mix were completely blended.

Mogawer et al. used Bonaquist's technique to evaluate 18 plant-produced mixtures from several northeastern states (*6*). This approach indicated that adequate blending occurred between the RAP and virgin binders in most cases. The authors argued that plant production parameters affected the degree of blending and the mix properties. McDaniel et al. also used Bonaquist's technique to assess the degree of blending for 25 plant mixes containing 15 to 40% RAP from four Indiana contractors and one Michigan contractor (*7*). They also found evidence of significant blending for the majority of the mixtures containing RAP.

As part of a study funded by the Alabama Department of Transportation and conducted by NCAT, four mix tests were evaluated for estimating effective binder properties using the Hirsch model (*8, 9*). The four mix tests investigated were dynamic modulus, dynamic shear rheometer with torsion bars, bending beam rheometer with mix beams, and the indirect tension relaxation modulus test. Testing included specimens fabricated with 100% virgin aggregates and binders and specimens fabricated with 100% RAP materials from several locations in Alabama. Only the results for backcalculating binder high and intermediate grade properties from dynamic moduli of 100% unmodified virgin mixes or 100% RAP specimens were promising. A sensitivity analysis of dynamic modulus was performed using laboratory-produced mixtures. Experimental factors included asphalt binder grade, RAP source, and RAP content (20%, 35%, and 50%). The results of this analysis indicated that the dynamic modulus and backcalculated binder properties were insensitive to both binder grade and RAP percentage.

Another key issue with regard to RAP mix designs is how to select the virgin binder grade for high RAP content mixtures. The current binder selection guidelines for RAP mixtures according to AASHTO M 323-13, *Standard Specification for Superpave Volumetric Mix Design*, were formulated based on the assumption that complete mixing occurs between the virgin binder and RAP binder. AASHTO M 323-13 recommends that for up to 15% RAP (of the mixture total weight), it is not necessary to change the grade of the virgin binder. When using between 16 to 25% RAP in a mixture, the virgin binder grade should be lowered by one grade level for both the high and low critical temperatures. For high RAP content mixtures (25+%), blending charts have been developed for performance grading (PG) properties to select the grade of the new binder based on properties of the RAP binder and proportion of the RAP binder in the total binder.

Blending charts assume that a composite asphalt binder (CAB) is created from the blending of the RAP binder, the new virgin asphalt, and/or the addition of a recycling agent. The blending of these different binders is assumed to occur during mixing and short-term mix storage at the asphalt plant. Recycling agents are also known as softening agents, rejuvenators, reclaiming agents, modifiers, fluxing oils, extender oils, and aromatic oils. Instead of using blending charts for high RAP mixture design, which require expensive, time-consuming binder extraction and recovery tests that use hazardous solvents, “bumping” down the binder grade has also been used effectively. Bumping means lowering both the low and high temperature grade of the virgin asphalt binder of HMA containing RAP. The objective is to compensate for the additional stiffness or brittleness of the mixture through blending the stiff RAP binder with a soft virgin binder.

AASHTO M320 includes the Performance-Graded Asphalt Binder specification, which is the basis for most specifications used by agencies in the United States. It is a performance related specification based on the rheological properties of asphalt binder. Binder grade selection considers anticipated high and low pavement temperatures, which may be adjusted to account for traffic loading and location within the pavement structure. Thus, when incorporating RAP in mixtures, one must obtain a suitable CAB for the climate where the asphalt mixture will be placed.

Scholz investigated how various proportions of RAP added to HMA mixtures affect the Superpave PG of the blended binder (*10*). Binders recovered from the mixtures with both RAP and RAS indicated an increase in both the high temperature and low temperature performance grades of the blended binder with increasing RAP contents up to about 30%. RAP contents above 30% did not result in any further increases in the low temperature performance grade and only slightly impacted the high temperature performance grade of the blended binders.

Wu et al. evaluated the effect of temperature on the viscosity of blends of RAP and virgin materials *(11)*. RAP binder was recovered and mechanically blended with an AH-70 virgin binder (penetration grading system). The RAP binder percentages evaluated were 0%, 25%, 50%, 75%, and 100%. Results of rotational viscosity testing were compared to the varying RAP percentages and temperatures. The test temperatures ranged between 125°C to 185°C. As expected, increasing the amount of RAP binder increased the viscosity at the same test temperature. Results were used to develop an equation that could be used to determine the mixing and compaction temperatures for any RAP mixture.

The results of the NCHRP 9-46 study indicated that the current standards for Superpave mix design are applicable to high RAP content mixes with a few minor but important changes to AASHTO R 35 and M 323 (*12*). Essentially, it was proposed that selection of the grade of virgin binder for high RAP content mixes should be based on knowledge of the true grade of the RAP binder, the high and low critical temperatures for the project location and pavement layer, and one of the following: (a) the approximate ratio of RAP binder divided by the total binder content, or (b) the high and low critical temperatures for the available virgin binder(s).

2.2 Laboratory Performance of Mixes with High RAP Content

Several recent studies have evaluated lab-produced and plant-produced RAP mixtures with a variety of mechanical tests. The three RAP sources and the two virgin binders used in NCHRP 9-12 were utilized to investigate the effects of RAP on the resulting mixture properties (*3*). Beam fatigue testing, shear tests, and indirect tensile tests were conducted to assess the effects of RAP on mixture stiffness at high, intermediate, and low temperatures. The results indicated a stiffening effect from the RAP binder at higher RAP contents. At low RAP contents, the mixture properties were not significantly different from those of mixtures with no RAP. The shear tests indicated an increase in stiffness and decrease in shear deformation as the RAP content increased, indicating that higher RAP content mixtures would exhibit higher resistance to rutting. The indirect tensile testing also showed increased stiffness for the higher RAP content mixtures. This is an indication of an increase in low temperature cracking. In addition, beam fatigue testing supported this conclusion since fatigue life decreased for higher RAP contents.

Li et al. investigated the effect of various types and percentages of RAP (up to 40%) on asphalt binder and asphalt mixture properties (*13*). The results of this study showed that the addition of RAP to a mixture generally increased the complex modulus and mixture stiffness only at the high end of the temperature range, making these mixtures more resistant to permanent deformation due to the addition of aged binder contained in the RAP. The IDT creep test was performed at temperatures of -18°C and -24°C. Results indicated that stiffness generally increased as the percentage of RAP increased, which is an indication of higher susceptibility to thermal cracking. Moisture susceptibility test data indicated that as the percentage of RAP increased, the strength also increased, while the tensile strength ratio decreased. This is another indication of an increase in moisture susceptibility as the content of RAP in the mix increases. Binder tests showed that the addition of RAP improved the binder grade in terms of high temperature performance, while the low temperature performance did not change significantly except for the case when 40% RAP was added, meaning that the resulting binder blends would be more resistant to rutting and equally resistant to thermal cracking compared to virgin binders.

A similar study developed by Daniel and Lachance indicated that at 15% RAP, the stiffness of the mixture increased and the compliance decreased, which indicates that the mixture would be more resistant to permanent deformation and less resistant to fatigue and thermal cracking due to the addition of aged binder contained in the RAP (*14*). However, mixtures containing 25 and 40% RAP did not follow the expected trends and behaved similar to the control mixture.

Beam fatigue results from mixtures at the NCAT Test Track suggest that cycles until failure were not statistically different when the virgin binder grade was bumped up or down (i.e., 45% RAP with PG 52-28, 45% RAP with PG 67-22, 45% RAP with PG 76-22 and 45% RAP with PG 76-22+Sasobit) (*15*). However, the fatigue lives of the RAP mixes were all much lower than for the virgin PG 67-22 mixture. This indicates that the use of softer binders with no other additive to compensate for the RAP binder's additional stiffness might not produce satisfactory results.

Huang et al. evaluated the laboratory fatigue characteristics of asphalt mixtures containing RAP (*16*). In this case 0, 10%, 20%, and 30% of fine RAP (passing No. 4 sieve) were used. Only one type of virgin aggregate was used and two types of asphalt binders (PG 64-22 and PG 76-22) were considered in this study. Fatigue characteristics of mixtures were evaluated through indirect tensile strength (IDT), beam fatigue, and semi-circular fatigue tests (SCB), and half of the specimens were long term aged. Results from the IDT test indicated that the increase of RAP had more tensile strength gains, suggesting that the recycled mixes would have an increased fatigue life. The results from the SCB fatigue test demonstrated that the inclusion of RAP generally increased the fatigue life of the mixtures. Results from beam fatigue tests indicated that the inclusion of RAP generally increased the flexural stiffness and fatigue life at low RAP content (10 to 20%). On the other hand, mixes subjected to long-term aging presented higher slopes of fatigue curves at 30% RAP content, which indicated potential lower fatigue life for these mixes at lower strain levels. In general, the results from this study showed that the inclusion of RAP increased the stiffness, indirect tensile strength, and laboratory fatigue resistance for the mixtures studied. However, mixture properties changed significantly at 30% RAP content as compared to those with 10 and 20% RAP.

Puttagunta et al. evaluated the fatigue and moisture damage potential of virgin and recycled mixes (25% and 50% RAP) through the use of indirect tensile strength and resilient modulus tests (*17*). Results for the indirect tensile strength test indicated that the tensile strength of all mixes decreased as temperature increased. No significant difference in the tensile strengths of the 25% and 50% recycled materials was found. From the results of the resilient modulus test, it was concluded that the virgin mix had a higher resilient modulus than the recycled mixes at all temperatures. On the other hand, at all temperatures, the difference between the results of the 25% and 50% recycled mixes was not significant. The fatigue analysis showed that the virgin mix generally had higher resistance to fatigue cracking than the recycled mixes. The fatigue performances of the 25% and 50% recycled mixes were relatively similar at all temperatures. In terms of moisture susceptibility, the results indicated that as the RAP content increased, the potential for moisture damage on the studied mixes decreased. The decrease in moisture susceptibility for recycled mixes was attributed to the fact that recycled aggregates allow a better coating with new asphalt as compared to virgin aggregates.

Sargious and Mushule studied the behavior of RAP containing mixes at low temperatures (*18*). This study included experimental analyses based on physical properties of asphalt mixtures and a theoretical analysis based on finite element methodology. In addition to the virgin control mix, 45% RAP and 55% RAP mixes were used. The results indicated that the performance of recycled asphalt pavements with respect to low-temperature cracking is superior to that of virgin asphalt pavements of comparable initial properties. Recycled mixtures had lower crack temperatures (-27°C for the virgin and -31.5°C for the recycled materials), which may be due to factors such as the use of a soft asphalt in the recycled mix as a modifier. Recycled mixtures also had higher coefficient of thermal conductivity (0.37 to 0.50 W/(m°C) higher), higher tensile strength (360 to 1260 kPa higher), and lower coefficient of thermal contraction (0.12×10^5/°C to 0.19×10^5/°C lower) than those of virgin mixtures. The theoretical work showed that pavement thickness and subgrade type play an important role in low-temperature cracking for both virgin and recycled asphalt pavements.

A research study developed by Vargas-Nordcbeck evaluated the effect of RAP on the combined overall performance of stone matrix asphalt (SMA) mixtures in Georgia (*19*). Four sources of RAP were combined at four levels (0%, 10%, 20%, and 30%) with four aggregate sources. Testing was performed to evaluate the binder effect on resistance to moisture susceptibility, rutting potential, thermal cracking potential, and fatigue life of the recycled mixtures. The results showed that increasing RAP content resulted in higher tensile strengths (conditioned and unconditioned) for moisture susceptibility testing. On the other hand, no significant difference was observed on the specimens tested for rutting potential when the amount of RAP was increased. Only fatigue life (at high strain levels) decreased significantly with the addition of 30% RAP. Adding up to 30% RAP had little effect on low-temperature performance grade properties, which may indicate that the grade of virgin binder does not have to be adjusted to provide the desired low-temperature binder properties.

Hajj et al. assessed the impact of high RAP content on moisture damage and thermal cracking (*20*). The mixes were designed using three RAP contents (0, 15, and 50%). A PG 58-28 binder was used for all mixes. An additional 50% RAP mix was made using a PG 52-34 virgin binder. All of the mixes were laboratory and plant produced mixtures designed to have similar gradations and binder contents. Laboratory test specimens were aged for four hours at 275°F prior to compaction while the plant-produced specimens were compacted without additional aging. Compacted mix specimens were subjected to either zero, one, or three freeze thaw cycles and then tested to determine their resistance to moisture damage using the tensile strength ratio (TSR) method (AASHTO T 283). Conditioned samples were also tested according to AASHTO TP 62 to assess changes in mixture dynamic modulus, |E*|, due to moisture conditioning. Finally, conditioned test specimens were tested using the Thermal Stress Restrained Specimen Test (TSRST) described in AASHTO TP 10. The researchers found that at 0 and 15% RAP, the recovered binders met the project binder grade requirement of PG 58-28. The 50% RAP mixture met the high-temperature grade requirement but did not meet the low-temperature requirement, even with the softer virgin binder. Plant-produced test specimens were found to be stiffer than the laboratory-produced specimens in most cases, although overall moisture damage trends and ranking were similar for all the tests performed. Dynamic modulus values decreased with increasing number of freeze-thaw cycles. The TSRST results showed no further reduction in fracture stress for the conditioned specimens with increasing RAP content. The TSRST fracture temperatures for the 0 and 15% RAP content specimens were very similar to the virgin binder low critical temperature. The 50% RAP content specimens had TSRST fracture temperatures several degrees warmer than the virgin binder, indicating decreased thermal cracking resistance. In general, moisture damage resistance and thermal cracking resistance improved with the use of the softer virgin binder.

West et al. performed a laboratory and field study on moderate and high RAP content surface mixes constructed on the NCAT Test Track in 2009 (*21*). Laboratory tests included APA rutting tests, dynamic modulus, bending beam fatigue, and energy ratio. The APA results corresponded to the effective stiffness of the binder in the mixes. Master curves of dynamic moduli showed the expected effects of the virgin binder grade on the stiffness of the mixtures. Beam fatigue tests indicated that the 45% RAP mixes had lower fatigue lives compared to the 20% RAP mixes, but the authors attributed this to lower effective volumes of asphalt in these mixes.

Willis et al. evaluated two means to improve durability of high RAP content mixes (*22*). One approach was to increase the asphalt content of the mixes by 0.25% and 0.5%; the other approach was to use a softer virgin binder grade. The study included 9.5 mm NMAS Superpave mixes designed with 0, 25, and 50% RAP with a PG 67-22 virgin binder and a softer PG 58-28 binder. The energy ratio test was used to evaluate the resistance to top-down cracking. The overlay tester was used to assess resistance to reflection cracking, and rutting potential was evaluated with the APA. Blended binders (recovered and virgin) were evaluated for fatigue resistance using the Linear Amplitude Sweep (LAS test). Results showed that the energy ratio decreased for the RAP mixes for both approaches (added virgin binder and softer virgin binder). However, fracture energy improved for the 25% and 50% RAP mixes when a PG 58-28 binder was used. Overlay tester results for the 25% RAP mixes significantly improved when the softer virgin binder was used. The average overlay tester results for the 50% RAP mixes with the PG 58-28 virgin binder also improved by three times compared to those with the PG 67-22 binder, but the results were not statistically significant due to the high variability of this test. The APA results for the 25% RAP mix containing PG 58-28 were just above the criterion established for high traffic mixes based on NCAT Test Track results. All other mixes met NCAT's recommended APA criterion. LAS testing also indicated that the softer virgin binder improved the fatigue resistance of the composite binder.

Another NCAT study evaluated ways to improve durability using a rejuvenating agent, Cyclogen L, to restore the performance grade properties of recycled binders (*23*). The study evaluated the effect of the rejuvenator on mixes containing 0% and 50% RAP, and another containing 20% RAP and 5% recycled asphalt shingles. The use of 12% of the rejuvenator was needed to restore the properties of the recycled binder to those of the PG 67-22 binder used as the virgin binder for the mix designs. The mix designs with and without the rejuvenator were tested for resistance to moisture damage using AASHTO T 283, rutting with the APA, dynamic modulus after short-term and long-term aging, resistance to top-down cracking using the energy ratio procedure, resistance to reflection cracking using the modified overlay tester procedure, and resistance to thermal cracking using the IDT creep compliance and strength tests. The tests showed that the rejuvenator reduced the mix stiffness, improved all four fracture properties included in the energy ratio computation, and improved the low-temperature critical cracking temperature. Overlay tester results also improved for the mixes with the rejuvenator, but the improvement was not statistically significant due to the poor repeatability of the test. All mixes passed the APA criterion for high traffic pavements.

2.3 Field Performance of Mixes with High RAP Content

Several studies have documented and analyzed the field performance of asphalt pavements containing RAP. Five projects were evaluated in Louisiana by Paul to compare functional performance (roughness, surface conditions, and rutting) and structural performance (structural number (SN) using the Dynaflect device) (*24*). These projects used 20 to 50% RAP and four conventional HMA mixtures. Conventional and RAP projects had the same contractor, similar mix designs, similar design traffic, and the same geological region. Some of the major distresses observed during five years of evaluation were longitudinal and transverse cracking and rutting. No significant difference was observed between RAP sections and the control section. Overall, pavements containing 20-50% RAP performed similarly to the conventional pavements for a period of six to nine years after construction.

Peters et al. recently reported the results of field performance of 16 projects with RAP contents ranging from 8 to 79% (half ≥ 70%) (*25*). These projects ranged from 1.5 to 10 years old. Two of the initial projects are still performing very well, and early data indicates equally promising results for the other 14 projects. Because of the impressive pavement performance exhibited by the recycled pavements, benefits such as conservation of natural resources and its cost advantage, hot-mix recycling became an attractive addition to the WSDOT paving program.

West et al. evaluated construction and performance of two sections with 20% RAP, four sections with 45% RAP, and a control section with no RAP at the NCAT Test Track (*26*). Different binders were used in the 45% RAP mixes including PG 52-28, PG 67-22, PG 76-22, and PG 76-22 plus 1.5% Sasobit. Analysis of compactability showed that the mixes with 20% RAP compacted easier than the mixes with 45% RAP. Of the four sections with 45% RAP, the two sections with softer binder required less compactive effort than the mixes with polymer modified binder. All sections performed well with regard to rutting (1 to 4 mm of rutting). The section with 20% RAP and a PG 67-22 virgin binder had the most rutting with 8.6 mm of rutting after 9.4 million equivalent single axle loads (ESALs). In terms of fatigue cracking, there was no significant evidence to conclude that one section performed better than any other section. In fact, some of the observed cracking was attributed to reflective cracking. With regards to functional performance, all of the 45% RAP sections had remarkably stable roughness data during the entire load cycle. The section with 20% RAP and PG 67-22 had a slight increase in roughness, which was primarily due to the increase in rutting.

Rorrer, Appea, and Clark studied different projects located in Virginia that used RAP contents from 10% to 30% (*27*). Field operations showed that the high RAP mixes were placed with minimal problems (98% to 102% of target value) and had satisfactory laboratory quality control results. In terms of rideability and International Roughness Index, the smooth paving operations resulted in the contractor earning an incentive cash bonus for one of the projects that used 30% RAP.

The US Army Research and Development Center evaluated in-service performance of pavements containing RAP in air force airfields that were 8 to 12 years old (*28*). Three airports from the U.S. and one from Terceira Island in Portugal were assessed. The mixes utilized in these airfields contained 35% to 60% RAP with rejuvenators or recycling agents. No pre-overlay structural deficiencies were observed and the extracted asphalt and aggregates were tested for physical properties. The Pavement Condition Index (PCI) was used to quantify the functional performance. PCI values ranged from 37 (poor) to 80 (very good). Low severity block cracking was the most dominant distress at all airports with high severity block cracking at the Portugal airport, which was the only mixture containing RAP with recycling agent. One airport also had low to medium severity patching and raveling distresses. In general, it was found that under the same environment conditions, pavements containing RAP performed similarly to virgin pavements. One of the recommendations of this report was that the design of mixes with RAP should be adjusted to resist the environment rather than to resist load.

The National Institute for Land and Infrastructure Management in Japan also evaluated the possibility of using RAP in airport surfaces (*29*). The main objective of this research project was to evaluate the effect of rejuvenators on performance of mixes containing RAP. Intensive lab tests were performed with various contents of RAP (up to 100 %) and rejuvenators. Some of the main findings are cited as follows: properties of mixes with RAP were similar with various rejuvenators; 100% RAP mixtures performed nearly as virgin pavements; 70% RAP pavements satisfied specifications for field performance; re-recycled pavement performed equal to recycled pavements; 70% RAP pavement is suitable for airport surfaces.

The Naval Civil Engineering Laboratory in Port Hueneme, California compared performance of pavements containing RAP in airfields with virgin airfield pavements and highway pavements containing RAP (*30*). Two five-year old airport pavements from California and North Dakota with 50% and 70% RAP were evaluated. Laboratory work included field cores tested for Marshal stability, resilient modulus, and moisture sensitivity. The extracted binder was tested for viscosity and penetration. It was found that RAP mixes from the California airport were stiffer and exhibited a TSR of 87%, and those from the North Dakota airport exhibited TSR values of 25-35%. In terms of performance, both pavements were rated as very good condition according to the FAA definition of PCI (greater than 75). Longitudinal and transverse cracking and raveling of low severity were the major distresses observed at the runway with the climatic effect on material durability as the primary distress mechanism, as expected at least for North Dakota.

2.4 WMA and RAP

The use of WMA may be beneficial for the performance of RAP mixtures when compared to HMA. Since excessive blue smoke is produced when RAP comes in contact with the burner flame, RAP material cannot be processed in drum mix plants (*31*). The proper plants for RAP mixture production include a separate RAP entry (i.e., drum mixer with center inlet, separate from the virgin aggregates entry) seeking to avoid exposure of the RAP to the burner flame. In the RAP mix production process, the virgin aggregates are super-heated such that they heat the RAP material by conduction. The RAP is heated only to a level where the RAP binder softens so that it blends with the virgin binder. Therefore, besides helping to mitigate the blue smoke problem and excess binder aging, the warm mix process may also help provide more suitable aggregate coating. The incorporation of WMA technology can produce a useful product without exposing RAP to relatively high temperatures in the plant.

Mallick et al. evaluated the use of WMA technology to produce high RAP asphalt mixtures (*32*). The results of this study showed that mixes containing 75% RAP had similar air voids as virgin mixes at lower temperatures than at conventional temperatures using 1.5% Sasobit WMA additive. One source of RAP was used in this study in addition to two sources of aggregate and three asphalt binders (PG 64-28, PG 52-28, and PG 46-40).

The lower temperatures used in WMA result in reductions of fuel consumption and emissions. According to Prowell et al., asphalt binder stiffness is reduced during WMA production, allowing the binder to sufficiently coat aggregates at lower temperatures (*31*). The different WMA technologies result in production temperature reductions of 35°F to 100°F. Thus, the reduction in fuel consumption and emissions is directly related to the temperature reduction.

The technique to produce asphalt mixes at lower temperatures includes the use of waxes, chemical additives, or water (through a foaming process). Wielinski et al. reported that the foaming process is accomplished by adding a small amount of water to the binder (*33*). The water then turns to steam and expands, resulting in a reduction of viscosity as a result of the expansion of the liquid asphalt binder.

3 LABORATORY TESTING PLAN AND METHODOLOGY

3.1 Project Information and Material Characterization

Six high RAP content projects were evaluated in Alabama in six different counties. Detailed descriptions of production, construction, and properties of the asphalt mixtures are included in the Appendix section. A brief description of each project is presented in the following sections.

Lafayette, Chambers County (AL-50)

A field project with a mix containing 35% RAP was constructed on AL-50 near Lafayette, Alabama in March 2011. The asphalt mixture consisted of a fine-graded 12.5-mm NMAS Superpave mix design with a compactive effort of 60 gyrations and used a PG 67-22 asphalt binder. The mixture was used as binder course and placed in a two-lane portion of AL-50 by East Alabama Paving. The mixture was produced as WMA using an Astec Double Barrel drum mix plant with water injection. The plant is located in Opelika, Alabama. The average production temperature for this project was 276°F, and the average measured temperature behind the paver screed was 243°F. The design aggregate gradation, optimum asphalt content, design volumetrics, specifications, and material percentages used for mix design and production are shown in Table 1.

Table 1 Mix Design Properties in Lafayette

Sieve Size, mm (in.)	Mix Design	Specifications	Aggregate Type	Mix Design (%)
	% Passing		#78 Limestone	28
25.0 (1”)	100.0			
19.0 (3/4")	100.0	100	Limestone SCRN’S	6
12.5 (1/2")	96.0	90 - 100		
9.5 (3/8")	85.0	0 - 90	M-10 Granite	8
4.75 (#4)	62.0			
2.36 (#8)	49.0	28 - 58	Baghouse Fines	1
1.18 (#16)	40.0			
0.6 (#30)	28.0		Sand	22
0.3 (#50)	15.0			
0.15 (#100)	9.0			
0.075 (#200)	5.6	2 - 10		
AC, %	5.1	--		
Air Voids, %	4.0	--		
VMA, %	15.2	> 14.5		
D/A Ratio	1.16	0.6 - 1.2		

Calera, Shelby County (I-65)

A second field project with a mix containing 35% RAP was constructed on I-65 near Calera, Alabama in March 2011. The asphalt mixture consisted of a fine-graded 19.0-mm NMAS Superpave mix design with a compactive effort of 60 gyrations and used a PG 67-22 asphalt binder. The mixture was used as binder course and placed by Wiregrass Construction Company, Inc. The mixture was produced as WMA using a drum mix plant with a Gencor Green Machine GX (water injection). The plant is located in Calera, Alabama. The average production temperature was 276°F, and the average measured temperature behind the screed was 238°F. The design aggregate gradation, optimum asphalt content, design volumetrics, specifications, and material percentages used for mix design and production are shown in Table 2.

Table 2 Mix Design Properties in Calera

Sieve Size, mm (in.)	Mix Design	Specifications	Aggregate Type	Mix Design (%)
	% Passing		#87 Limestone	25
25.0 (1”)	100.0	100		
19.0 (3/4")	99.0	90 - 100	#78 Limestone	9
12.5 (1/2")	88.0	0 - 90		
9.5 (3/8")	76.0		¼” Limestone	7
4.75 (#4)	55.0			
2.36 (#8)	41.0	23 - 49	# 69 Gravel	5
1.18 (#16)	33.0			
0.6 (#30)	23.0		Baghouse Fines	1
0.3 (#50)	12.0			
0.15 (#100)	7.0		Sand	18
0.075 (#200)	4.9	2 - 8		
AC, %	4.4	--		
Air Voids, %	3.9	--		
VMA, %	14.2	> 13.5		
D/A Ratio	1.12	0.6 - 1.2		

Wing, Covington County (AL-137)

A third field project with a mix containing 35% RAP was constructed on AL-137 near Wing, Alabama in April 2011. The asphalt mixture consisted of a fine-graded 12.5-mm NMAS Superpave mix design with a compactive effort of 60 gyrations and used a PG 67-22 asphalt binder. The mixture was used as binder course and placed by Wiregrass Construction Company, Inc. The mixture was produced as WMA using a drum mix plant with Tyrex water injection technology. The plant is located in Brantley, Alabama. The average production temperature was 290 °F, and the average measured temperature behind the screed was 263°F. The design aggregate gradation, optimum asphalt content, design volumetrics, specifications, and material percentages used for mix design and production are shown in Table 3.

Table 3 Mix Design Properties in Wing

Sieve Size, mm (in.)	Mix Design	Specifications	Aggregate Type	Mix Design (%)
	% Passing			
25.0 (1")	100.0		#67 Limestone	15
19.0 (3/4")	100.0	100	#8910 Limestone	11
12.5 (1/2")	92.0	90 - 100		
9.5 (3/8")	83.0	0 - 90	Shot Gravel	17
4.75 (#4)	61.0			
2.36 (#8)	47.0	28 - 58	Coarse Sand	21
1.18 (#16)	40.0			
0.6 (#30)	31.0		Baghouse Fines	1
0.3 (#50)	15.0			
0.15 (#100)	8.0			
0.075 (#200)	4.4	2 - 10		
AC, %	5.1	--		
Air Voids, %	3.5	--		
VMA, %	14.9	> 14.5		
D/A Ratio	0.9	0.6 - 1.2		

Troy, Pike County (US-29)

A fourth field project with a mix containing 32% RAP/3% RAS was constructed on US-29 near Troy, Alabama in July 2011. The asphalt mixture consisted of a fine-graded 12.5-mm NMAS Superpave mix design with a compactive effort of 60 gyrations and used a PG 67-22 asphalt binder. The mixture was used as binder course and placed by Wiregrass Construction Company, Inc. The mixture was produced as WMA using a drum mix plant with Tyrex water injection technology. The plant is located in Brantley, Alabama. The average production temperature was 285°F, and the average measured temperature behind the screed was 253°F. The design aggregate gradation, optimum asphalt content, design volumetrics, specifications, and material percentages used for mix design and production are shown in Table 4.

Table 4 Mix Design Properties in Troy

Sieve Size, mm (in.)	Mix Design	Specifications	Aggregate Type	Mix Design (%)
	% Passing		#67 Limestone	10
25.0 (1”)	100.0			
19.0 (3/4")	100.0	100	#8910 Limestone	9
12.5 (1/2")	94.0	90 - 100		
9.5 (3/8")	87.0	0 - 90	½” Gravel	23
4.75 (#4)	65.0			
2.36 (#8)	44.0	28 - 58	Shot Gravel	11
1.18 (#16)	31.0			
0.6 (#30)	21.0		Coarse Sand	11
0.3 (#50)	12.0			
0.15 (#100)	8.0		Baghouse Fines	1
0.075 (#200)	6.6	2 - 10		
AC, %	5.1	--		
Air Voids, %	4.0	--		
VMA, %	15.3	> 14.5		
D/A Ratio	1.11	0.6 - 1.2		

Fort Payne, Cherokee County (AL-35)

A fifth field project with a mix containing 35% RAP was constructed on AL-35 near Fort Payne, Alabama in September 2011. The asphalt mixture consisted of a fine-graded 19.0-mm NMAS Superpave mix design with a compactive effort of 60 gyrations and used a PG 67-22 asphalt binder. The mixture was used as binder course and placed by Goodhope Contracting. The mixture was produced as WMA using a drum mix plant with Evotherm 3G (terminal blend @ 0.5%) technology. The plant is located in Collinsville, Alabama. The average production temperature was 257°F, and the average measured temperature behind the screed was 233°F. The design aggregate gradation, optimum asphalt content, design volumetrics, specifications, and material percentages used for mix design and production are shown in Table 5.

Table 5 Mix Design Properties in Fort Payne

Sieve Size, mm (in.)	Mix Design	Specifications	Aggregate Type	Mix Design (%)
	% Passing		#67 Limestone	20
25.0 (1”)	100.0			
19.0 (3/4")	100.0	100	#78 Limestone	20
12.5 (1/2")	92.0	90 - 100		
9.5 (3/8")	83.0	0 - 90	Limestone SCRN’s	10
4.75 (#4)	61.0			
2.36 (#8)	47.0	28 - 58	Sand	15
1.18 (#16)	40.0			
0.6 (#30)	31.0			
0.3 (#50)	15.0			
0.15 (#100)	8.0			
0.075 (#200)	4.4	2 - 10		
AC, %	4.4	--		
Air Voids, %	4.1	--		
VMA, %	14.2	> 13.5		
D/A Ratio	0.92	0.6 - 1.2		

Lowndesboro, Lowndes County (US-80)

The final field project was a mix containing 40% RAP constructed on US-80 near Lowndesboro, Alabama in February 2012. The asphalt mixture consisted of a fine-graded 12.5-mm NMAS Superpave mix design with a compactive effort of 60 gyrations and used a PG 67-22 asphalt binder. The mixture was used as binder course and placed by Wiregrass Construction Company, Inc. The mixture was produced as WMA using a drum mix plant with Evotherm 3G (terminal blend @ 0.7%) technology. The plant is located in Montgomery, AL. The average production temperature was 282°F, and the average measured temperature behind the screed was 238°F. The design aggregate gradation, optimum asphalt content, design volumetrics, specifications, and material percentages used for mix design and production are shown in Table 6.

A summary of asphalt mixture properties for all projects is shown in Table 7. The mixture from AL-50 had the highest overall asphalt content while I-65 and AL-35 mixtures had the lowest. The remaining mixtures contained a similar amount of total binder. All mixtures met their respective VMA criteria, however, the effective binder content for mixture AL-35 was significantly lower than the other mixes. On the other hand, mixture AL-137 had significantly lower D/A ratio compared to the remaining mixtures. These properties were used later on to explain some of the performance behavior of these mixtures.

Table 6 Mix Design Properties in Lowndes

<table>
<tr><th rowspan="2">Sieve Size, mm (in.)</th><th>Mix Design</th><th>Specifications</th><th>Aggregate Type</th><th>Mix Design (%)</th></tr>
<tr><th colspan="2">% Passing</th><td rowspan="2">#78 Limestone</td><td rowspan="2">15</td></tr>
<tr><td>25.0 (1”)</td><td>100.0</td><td></td></tr>
<tr><td>19.0 (3/4")</td><td>100.0</td><td>100</td><td rowspan="2">Limestone SCRN’s</td><td rowspan="2">12</td></tr>
<tr><td>12.5 (1/2")</td><td>97.0</td><td>90 - 100</td></tr>
<tr><td>9.5 (3/8")</td><td>89.0</td><td>0 - 90</td><td rowspan="2">Shot Gravel</td><td rowspan="2">21</td></tr>
<tr><td>4.75 (#4)</td><td>68.0</td><td></td></tr>
<tr><td>2.36 (#8)</td><td>45.0</td><td>28 - 58</td><td rowspan="2">Sand</td><td rowspan="2">11</td></tr>
<tr><td>1.18 (#16)</td><td>32.0</td><td></td></tr>
<tr><td>0.6 (#30)</td><td>23.0</td><td></td><td rowspan="2">Baghouse Fines</td><td rowspan="2">1</td></tr>
<tr><td>0.3 (#50)</td><td>12.0</td><td></td></tr>
<tr><td>0.15 (#100)</td><td>8.0</td><td></td><td colspan="2" rowspan="2"></td></tr>
<tr><td>0.075 (#200)</td><td>5.4</td><td>2 - 10</td></tr>
<tr><td>AC, %</td><td>5.1</td><td>--</td><td colspan="2" rowspan="4"></td></tr>
<tr><td>Air Voids, %</td><td>3.4</td><td>--</td></tr>
<tr><td>VMA, %</td><td>15.0</td><td>> 14.5</td></tr>
<tr><td>D/A Ratio</td><td>1.08</td><td>0.6 - 1.2</td></tr>
</table>

Table 7 Summary of Quality Control Properties

Property	AL-50	I-65	AL-137	US-29	AL-35	US-80
AC, %	5.25	4.35	4.90	5.15	4.38	5.00
Air Voids, %	3.44	4.30	3.84	3.94	4.14	4.17
VMA, %	14.9	15.4	15.0	14.8	14.2	16.8
Vbe, %	11.46	11.1	11.16	10.86	10.06	12.63
D/A Ratio	1.17	0.99	0.80	1.23	1.19	0.90

3.2 Laboratory Characterization of Extracted Binder

When the mixtures were returned to NCAT, the first component characterized was the binder. The blended RAP-virgin binder from the asphalt mixture was extracted and recovered using AASHTO T164 Method A and ASTM D5404. Once the binder was extracted and recovered from the asphalt mixture, it underwent three sets of tests: performance grade characterization according to AASHTO M320 and R29, multiple stress creep and recovery performance grading according to AASHTO MP19, and linear amplitude sweep testing according to AASHTO TP101. Performance grading and MSCR grading are methods of placing binders into various grades based on performance. The linear amplitude sweep test has recently been developed to assess the fatigue properties of asphalt binders; however, further correlation to field performance is needed before the test becomes more widely accepted. Using the LAS test as a part of this research project allows the research team to determine if the LAS can be used to assess possible fatigue performance in the field based solely on a binder test.

The parameter ΔTc was also calculated for the recovered asphalt binders and the virgin binders. This parameter has received a great deal of interest in recent years as a potential indicator of susceptibility to age-related block cracking based on the work by Anderson (*34*). ΔTc was calculated as the numerical difference between the low continuous grade temperatures determined from the BBR stiffness criterion and the m-value criterion. Throughout the course of this project, most discussions regarding the ΔTc parameter among asphalt researchers were based on tests conducted after the standard 20-hour PAV aging for PG testing.

3.3 Laboratory Performance Tests

Dynamic Modulus

Dynamic modulus testing was performed for all mixtures. The samples were prepared in accordance with AASHTO PP60-09 from re-heated plant-produced mix and compacted to a height of 175 mm and a diameter of 150 mm. Three samples were prepared for testing from each mix. Although the specification recommends ± 0.5% air voids as a reasonable tolerance for the cut samples, it does not specify a target air void content. A target air void content of 7% was selected for this project. This is a common target for the air void content of an in-place pavement post-compaction and is a typical target for HMA performance testing samples in the laboratory.

Dynamic modulus testing was performed in an IPC Global Asphalt Mixture Performance Tester (AMPT), shown in Figure 1. Dynamic modulus testing is performed in order to quantify the stiffness of the asphalt mixture over a wide range of testing temperatures and loading rates (or frequencies). The temperatures and frequencies used for testing these mixes are those recommended by AASHTO PP61-10. For this methodology, the high test temperature is dependent on the high PG grade of the base binder utilized in the mix being tested.

Figure 1 IPC Global Asphalt Mixture Performance Tester

Dynamic modulus testing was performed in accordance with AASHTO TP62 in an unconfined condition. Unconfined data is most commonly used for dynamic modulus testing since current mechanistic design software packages were calibrated using unconfined dynamic modulus data. Unconfined testing is also significantly easier to perform than confined testing. The collected data was used to generate a master curve for each individual mix. The master curve uses the principle of time-temperature superposition to horizontally shift data at multiple temperatures and frequencies to a reference temperature so that the stiffness data can be viewed without temperature as a variable. This method of analysis allows for visual relative comparisons to be made between multiple mixes. An example of using the time-temperature superposition principle to generate a master curve is shown in Figure 2.

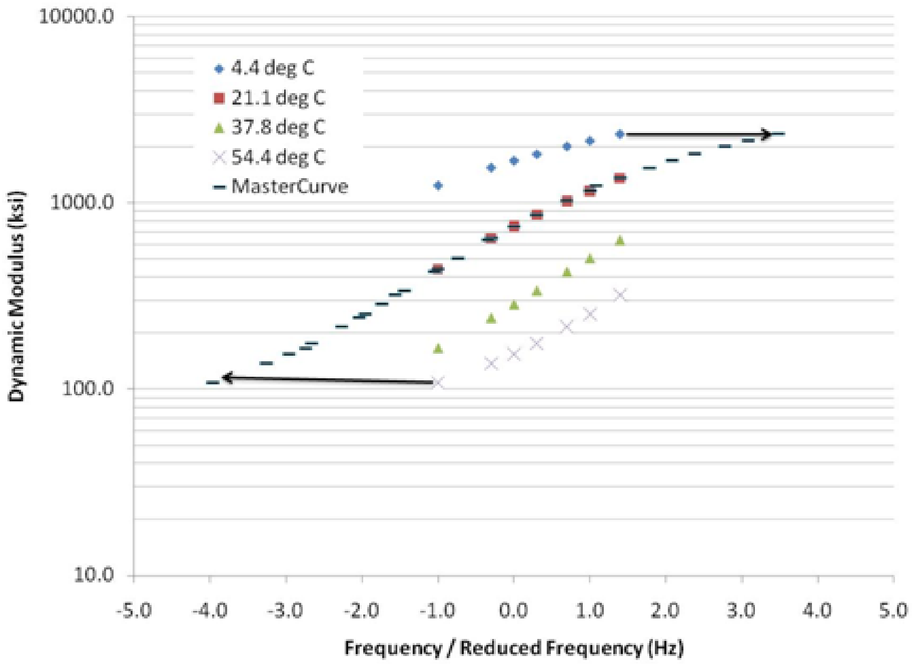

Figure 2 Example Master Curve Generation

Data analysis was conducted per the methodology in AASHTO PP61-10. The general form of the master curve equation is shown as Equation 1. As mentioned, the dynamic modulus data are shifted to a reference temperature. This is done by converting testing frequency to a reduced frequency using the Arrhenius equation (Equation 2). Substituting Equation 2 into Equation 1 yields the final form of the master curve equation, shown in Equation 3. The shift factors required at each temperature are given in Equation 4. A reference temperature of 20°C was used for this analysis. The limiting maximum modulus in Equation 1 is calculated using the Hirsch Model, shown as Equation 5. A limiting binder modulus of 1 GPa is assumed for this equation. Non-linear regression is then conducted using the Solver function in EXCEL® to develop the coefficients for the master curve equation. Typically, these curves have an S_e/S_y term of less than 0.05 and an R^2 value of greater than 0.99. Definitions for the variables in Equations 1-6 are given in Table 8.

$$Log|E^*| = \partial + \frac{(Max-\partial)}{1+e^{\beta+\gamma log f_r}} \tag{1}$$

$$log f_r = log\ f + \frac{\Delta E_a}{19.14714}\left[\frac{1}{T} - \frac{1}{T_r}\right] \tag{2}$$

$$log|E^*| = \partial + \frac{(Max-\partial)}{1+e^{\beta+\gamma\left\{log f+\frac{\Delta E_a}{19.14714}\left[\frac{1}{T}-\frac{1}{T_r}\right]\right\}}} \tag{3}$$

$$log\ [a(T)] = \frac{\Delta E_a}{19.14714}\left[\frac{1}{T} - \frac{1}{T_r}\right] \tag{4}$$

$$|E^*|_{max} = P_c\left[4{,}200{,}000\left(1 - \frac{VMA}{100}\right) + 435{,}000\left(\frac{VFA*VMA}{10{,}000}\right) + \frac{1-P_c}{\frac{\left(1-\frac{VMA}{100}\right)}{4{,}200{,}000} + \frac{VMA}{435{,}000(VFA)}}\right] \tag{5}$$

$$P_c = \frac{\left(20+\frac{435{,}000(VFA)}{VMA}\right)^{0.58}}{650+\left(\frac{435{,}000(VFA)}{VMA}\right)^{0.58}} \tag{6}$$

Table 8 Master Curve Equation Variable Descriptions

Variable	Definition
$\|E^*\|$	Dynamic Modulus, psi
δ, β, γ	Fitting Parameters
Max	Limiting Maximum Modulus, psi
f_r	Reduced frequency at reference temperature, Hz
f	Loading frequency at test temperature, Hz
ΔE_a	Activation Energy (treated as a fitting parameter)
T	Test Temperature, ºK
T_r	Reference Temperature, ºK
$a(T)$	Shift factor at Temperature, T
$\|E^*\|_{max}$	Limiting maximum HMA dynamic modulus, psi
VMA	Voids in Mineral Aggregate, %
VFA	Voids filled with asphalt, %

Uniaxial Fatigue (S-VECD)

Uniaxial fatigue testing based on continuum damage mechanics has been studied and conducted in universal servo-hydraulic load frames to characterize the fatigue characteristics of asphalt mixtures. The theoretical background of this method has been presented in several publications (*35*-*37*). However, the recent draft test procedure by Dr. Richard Kim at North Carolina State University allows the uniaxial fatigue test (known as the S-VECD test) to be conducted in the AMPT (*37*).

To characterize the fatigue characteristics of a mixture using the S-VECD model, two tests are performed in the AMPT. First, the dynamic modulus of the mixture is determined according to the AASHTO TP 79-10 test protocol to quantify the linear viscoelastic (LVE) characteristics of the mix. Second, a controlled crosshead (CX) cyclic fatigue test is performed using the fatigue testing software in the AMPT to acquire the necessary fatigue data. The test protocol this software utilizes is discussed by Hou et al. (*36*). To conduct this test, an AMPT sample is glued with a steel epoxy to two end platens. The test specimen and end platens are then attached with screws to the actuator and reaction frame of the AMPT prior to installing on-specimen LVDTs. A photo of this test setup is shown in Figure 3.

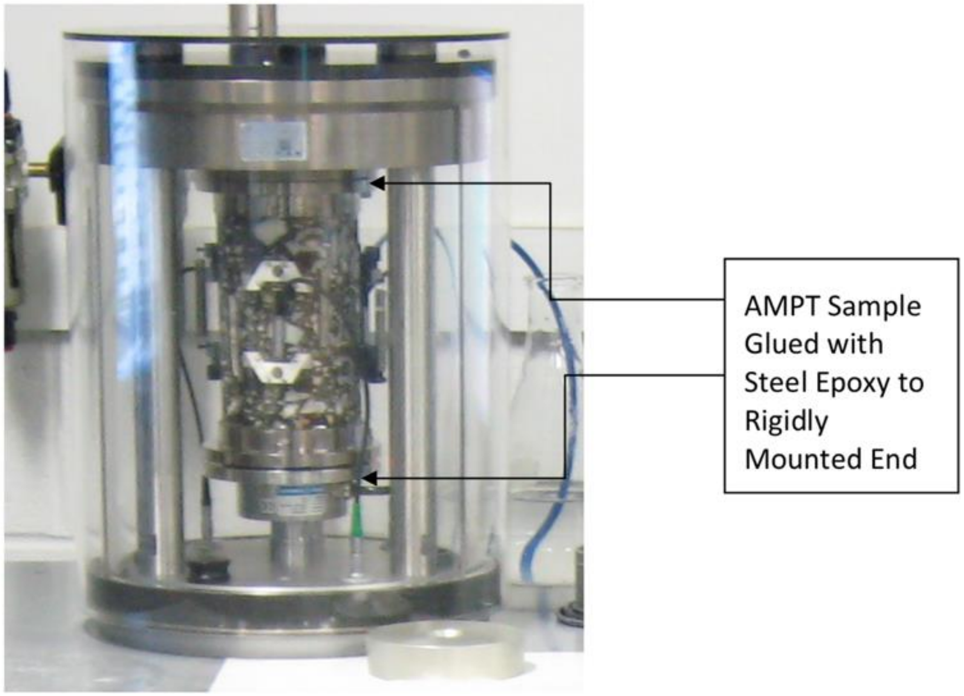

Figure 3 AMPT S-VECD Fatigue Test Setup

The CX test was performed at 20°C with a frequency of 10 Hz. Testing consisted of two phases. First, a small strain (50 to 75 on-specimen microstrain) test was performed to determine the fingerprint dynamic modulus of the specimen. This was done to determine the ratio of the fingerprint dynamic modulus ($|E^*|_{Fingerprint}$) of the testing specimen to the dynamic modulus determined from AMPT dynamic modulus testing ($|E^*|_{LVE}$). This value is known as the dynamic modulus ratio (DMR) and is expected to fall between 0.9 and 1.1 using Equation 7 (*36*). This ratio is used for controlling the quality of the fatigue testing and is incorporated into the S-VECD fatigue model (*35*). Second, the specimen was subjected to a fatigue test in which the AMPT actuator was programmed to reach a constant peak displacement with each loading cycle. During this test, the dynamic modulus and phase angle of the sample were recorded. Failure of the specimen was defined as the point at which the phase angle peaks and then dropped off (*35*). This concept is demonstrated graphically in Figure 4.

$$DMR = \frac{|E^*|_{Fingerprint}}{|E^*|_{LVE}} \tag{7}$$

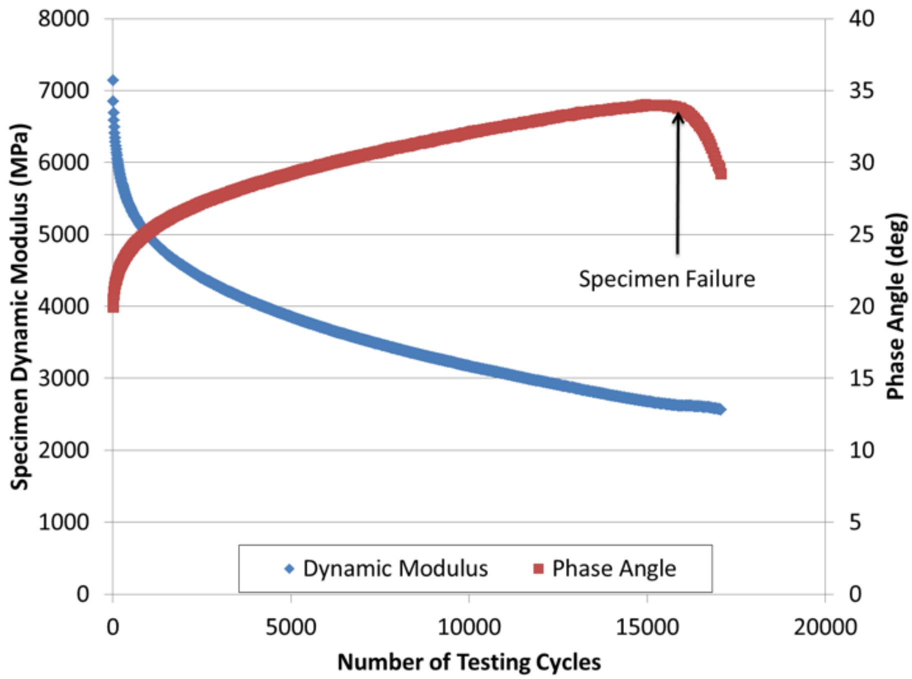

Figure 4 Determination of Cycles to Failure for S-VECD Fatigue Test

The initial target peak-to-peak on-specimen strain was specified in the software prior to the start of the test. Four fatigue specimens were tested with two replicates at two different strain levels. These strain levels were selected empirically so that the cycles to failure of the mix at the two strain levels were approximately an order of magnitude apart (i.e. 1,000 cycles to failure for one strain level versus 10,000 cycles to failure for another strain level). However, past research has shown that sufficient S-VECD fatigue predictions can be made with only two specimens (*35*). Both the dynamic modulus test and controlled crosshead cyclic test were performed using samples prepared in accordance with AASHTO PP 60-09. All samples were prepared to 7 ± 0.5% air voids. Typically, three specimens of mix were required for dynamic modulus testing and four to six specimens were needed to get sufficient fatigue data.

The S-VECD fatigue data analysis was performed using an analysis package developed at North Carolina State University. This software has been used for S-VECD fatigue testing on servo-hydraulic load frames in the past but was updated to process the data generated by the fatigue testing software in the AMPT. Five primary steps were needed for the data processing, as follows.

1. The number of testing cycles to failure was determined for each specimen based on the phase angle curve (see Figure 4).
2. The AMPT dynamic modulus data were entered into the fatigue analysis software. The software utilized these data to compute the Prony series coefficients for creep compliance and relaxation modulus of the mixture (*36*). The dynamic modulus data were also used to determine the dynamic modulus master curve and the DMR value as discussed earlier.

3. The individual fatigue data files were individually analyzed to determine the *C* (pseudo-stiffness) versus *S* (damage parameter) curve. During this step, the individual files were examined to determine the value of *C* that corresponded to the 'failure' cycle for each mix.
4. The combined *C* versus *S* curve for the mix was then determined based on the individual *C* versus *S* curves. The composite *C* versus *S* curve was fit using a power law, shown as Equation 8 (where C_{11} and C_{12} are the regression coefficients) (*36*). These curves are fit to the point of failure (defined by *C* at failure) for each mix.

$$C = 1 - C_{11} S^{C_{12}} \tag{8}$$

5. Finally, a fatigue prediction was made using the S-VECD model. Fatigue predictions for this study were made using the controlled-strain assumption based on the formula in Equation 9 (*37*). These fatigue simulations can be performed in the fatigue analysis software package. However, for this project, these simulations were performed in an EXCEL® spreadsheet using the parameters developed by the fatigue analysis software for each mix.

$$Nf = \frac{(f_R)(2^{S\alpha}) S_f^{\alpha - \alpha \cdot C_{12} + 1}}{(\alpha - \alpha \cdot C_{12} + 1)(C_{11} \cdot C_{12})^{\alpha} \left[(\beta + 1)(\varepsilon_{0,pp})(|E^*|_{LVE})\right]^{2\alpha} K_1} \tag{9}$$

Where:

N_f	=	number of cycles until fatigue failure,
C	=	pseudo-stiffness,
S	=	damage parameter,
f_R	=	reduced frequency for dynamic modulus shift factor at fatigue simulation temperature and loading frequency,
α	=	damage evolution rate for S-VECD model,
$\varepsilon_{0,pp}$	=	peak-to-peak strain for fatigue simulation,
$\|E^*\|_{LVE}$	=	dynamic modulus of mix from dynamic modulus master curve at the fatigue simulation temperature and loading frequency,
C_{11}, C_{12}	=	power law coefficients from *C* vs *S* regression,
β	=	mean strain condition (assumed to be zero for this project), and
K_1	=	adjustment factor based on time history of loading – function of α and β.

Energy Ratio

The energy ratio test procedure was developed to assess an asphalt mixture's resistance to top-down or surface cracking (*38*). Energy ratio is determined using a combination of three tests: resilient modulus, creep compliance, and indirect tensile strength. These tests were performed at 10°C using an MTS® testing device. The tests were conducted on three specimens 150 mm diameter by approximately 38 to 50 mm thick, cut from gyratory compacted samples (Figure 5). The target air voids for the specimens was 7 ± 0.5 percent. The energy ratio test method was selected because each individual parameter can be also used to characterize asphalt mixture and provide relative cracking performance in the laboratory.

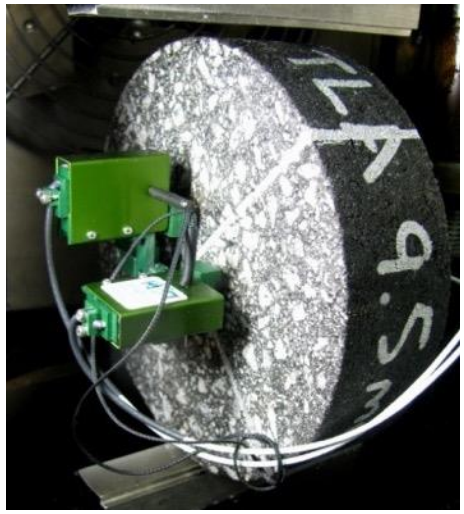

Figure 5 Energy Ratio Test Specimen Setup

The resilient modulus is obtained by applying a repeated haversine waveform load in load-control mode. The load is applied for 0.1 second followed by a 0.9 second rest. The resilient modulus is calculated using the stress-strain curve as shown in Figure 6. The creep compliance test is performed as described in AASHTO T322-07; however, the temperature of the test is 10°C with a test duration of 1000 seconds. The power function properties of the creep compliance test can be determined by curve-fitting the results obtained during constant load control mode. Finally, the tensile strength and dissipated creep strain energy (DCSE) at failure are determined from the stress-strain curve of the given mixture during the indirect tensile strength test. The results from these tests are then used to evaluate each mixture's surface cracking resistance using Equation 10.

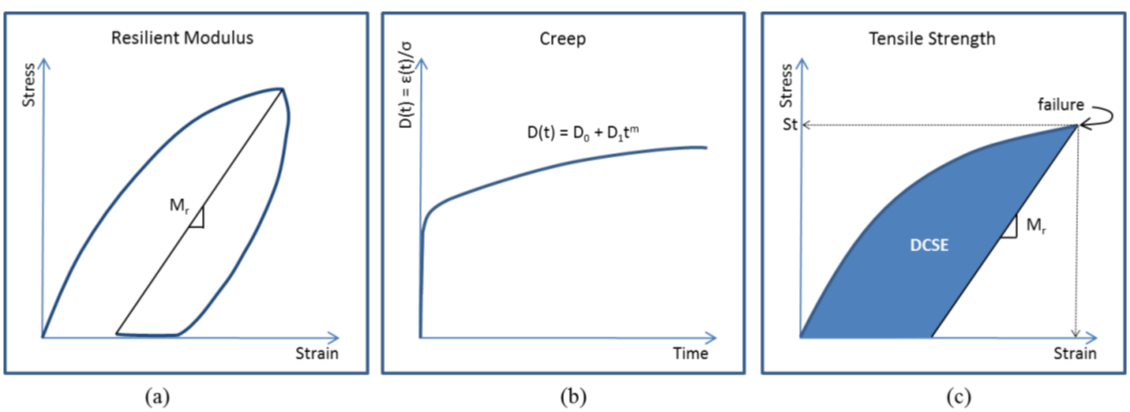

Figure 6 Parameters Determined from (a) Resilient Modulus, (b) Creep, and (c) Strength Tests

$$ER = \frac{DSCE_f\left[7.294\times10^{-5}\times\sigma^{-3.1}(6.36-S_t)+2.46\times10^{-8}\right]}{m^{2.98}D_1} \tag{10}$$

Where:

σ = tensile stress at the bottom of the asphalt layer, 150 psi;

M_r = resilient modulus;
D_1, m = power function parameters;
S_t = tensile strength;
$DSCE_f$ = dissipated stress creep energy at failure; and
ER = energy ratio.

Florida researchers found that the ER criteria distinguished cracked and un-cracked sections in 19 of the 22 pavements studied by Roque, et al. (*38*). An additional parameter was recommended to supplement the ER criteria for the sections that did not fit the ER criteria. Mixtures from two sections had energy ratios of greater than 1.0 but still exhibited top-down cracking. Both sections had $DCSE_{HMA}$ thresholds less than 0.75 kJ/m^3 while an un-cracked mixture that had an ER of less than 1.0 had a $DCSE_{HMA}$ threshold of 2.5 kJ/m^3. Therefore, an additional criterion for $DCSE_{HMA}$ was added to screen out very stiff and brittle mixtures. Table 9 shows the ER criteria by Roque, et al. (*38*) for mixtures with different traffic ranges and the supplemental criteria based on the $DCSE_{HMA}$.

Table 9 Recommended Energy Ratio Criteria (*38*)

Mix Property	Criterion	
Energy Ratio	Traffic MESALs: <250 <500 <1000	Min. Energy Ratio 1.0 1.3 1.95
$DCSE_{HMA}$	> 0.75 kJ/m^3	
$DCSE_{HMA}$	Recommended Range: 0.75 – 2.5 kJ/m^3	

Semi-Circular Bending (SCB) Test

An MTS servo-hydraulic testing system equipped with an environmental chamber was used to perform the SCB test. As shown in Figure 7, the SCB samples are symmetrically supported by two fixed rollers and have a span of 120 mm. Teflon tape is used to minimize friction between the specimen and the rollers. The plot of the load versus the external displacement is used to compute the area under the curve to the peak load.

Figure 8 shows typical load-vertical deflection curves obtained in the SCB test at three nominal notch depths of 25.4, 31.8, and 38.0 mm. In order to obtain the critical value of fracture resistance, J_C, the area under the loading portion of the load deflection curves, up to the maximum load, needs to be measured for each notch depth of each mixture. This area represents the strain energy to failure, U. The average values of U at each notch depth are then plotted versus notch depth to obtain a changing slope of U from a regression line, as shown in Figure 9. This slope is the value of dU/dA in Equation 11. Finally, the J_C can be computed by dividing the dU/dA value by the specimen width of b.

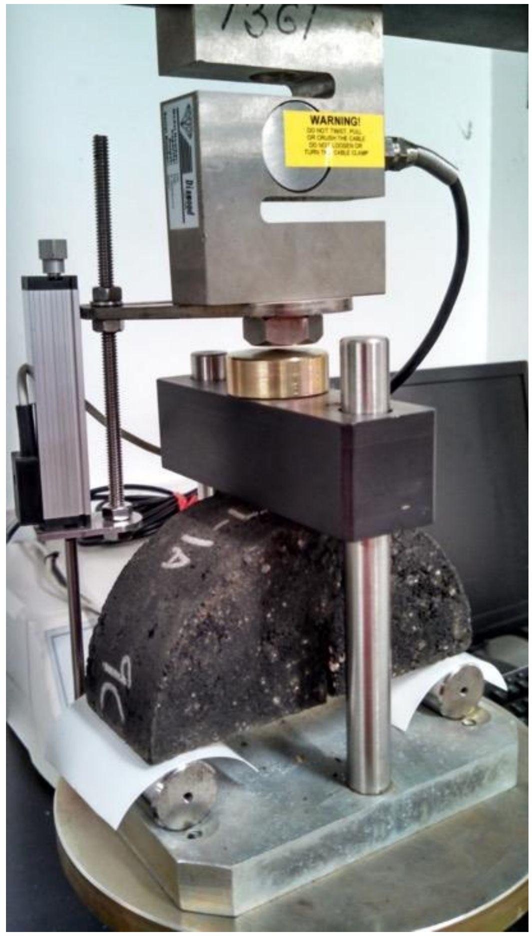

Figure 7 Semi-Circular Bending Test

$$J_c = -\left(\frac{1}{b}\right)\frac{dU}{dA} \tag{11}$$

Where:

- b = sample thickness,
- a = notch depth,
- U = strain energy to failure, and
- dU/dA = slope of the area vs. displacement curve.

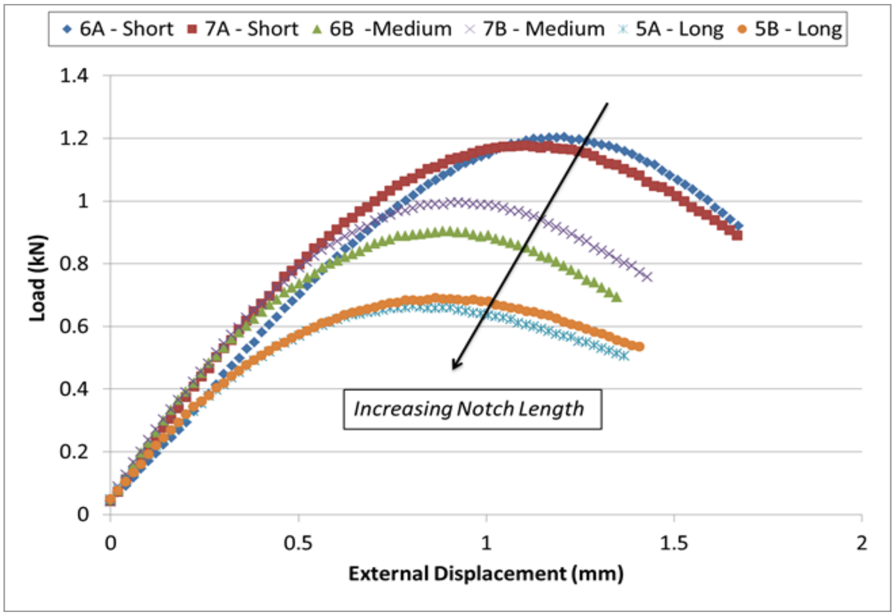

Figure 8 Typical Plot of Load versus Load Line Displacement

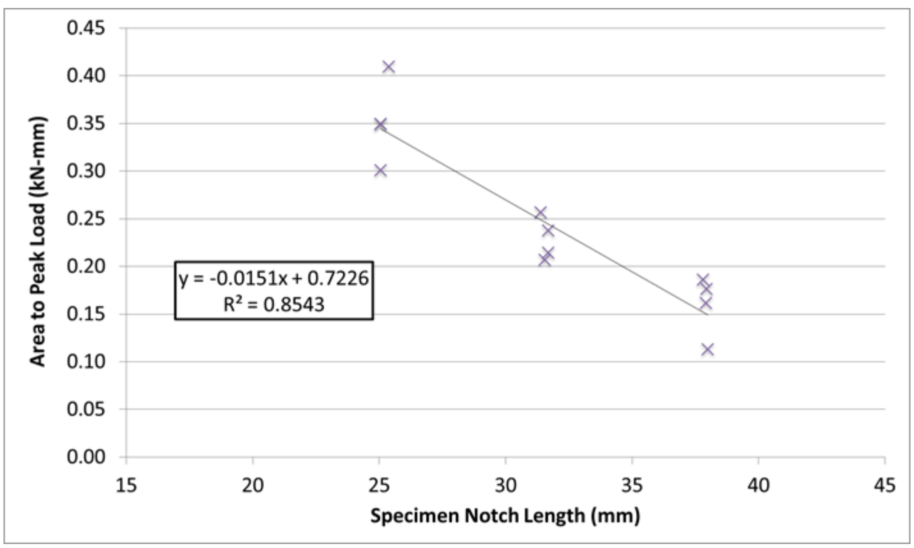

Figure 9 Plot of Area versus Specimen Notch Length

Fracture Energy and Flexibility Index

Illinois Flexibility Index Testing (I-FIT) was performed at NCAT for this project using an I-FIT device. Semi-circular asphalt specimens were prepared to an air void level of 7.0 ± 0.5% after trimming. Each specimen was trimmed from a larger 160 mm tall by 150 mm diameter gyratory specimen. Four replicates could be obtained per specimen. A notch was then trimmed into each specimen at a target depth of 15 mm and width of 1.5 mm along the center axis of the specimen. The specimens were tested at a target test temperature of 25.0 ± 0.5°C after being conditioned in an environmental chamber for two hours. Specimens were loaded monotonically at a rate of 50 mm/min until the load dropped below 0.1 kN after the peak was recorded. Both force and actuator displacement were recorded at a rate of 50 Hz by the system. Figure 10 shows the Test Quip® I-FIT device at NCAT.

Figure 10 Test Quip® I-FIT Device

To calculate the flexibility index (Equation 12), the slope of the post-peak portion of the curve must be determined. This is the maximum slope of the curve immediately after the peak. The flexibility index is calculated by dividing the fracture energy by the post-peak slope and then multiplying that quotient by a scaling factor. In general, a higher fracture energy is indicative of a mix with better cracking resistance. A higher flexibility index is indicative of a mix exhibiting a more ductile failure while a lower flexibility index indicates a more brittle failure. Data analysis can be performed using software developed by ICT. The fracture energy (Equation 2-13) represents the area under the stress-strain curve normalized for the specimen dimensions. It is calculated by integrating the area under the load-displacement curve and dividing it by the ligament area (the area of the semi-circular specimen through which the crack will propagate). An example of processed I-FIT data from this software is shown in Figure 11.

$$FI = \frac{G_f}{|m|} \; x \; A \tag{12}$$

$$G_f = \frac{w_f}{a_{lig}} \tag{13}$$

Where:

G_f = fracture energy (J/m^2);
w_f = work of fracture (J);
a_{lig} = ligament area (mm^2) = (specimen radius – notch length) x specimen width;
FI = Flexibility Index;
m = post-peak slope (kN/mm); and
A = scaling factor (0.01 for gyratory specimens).

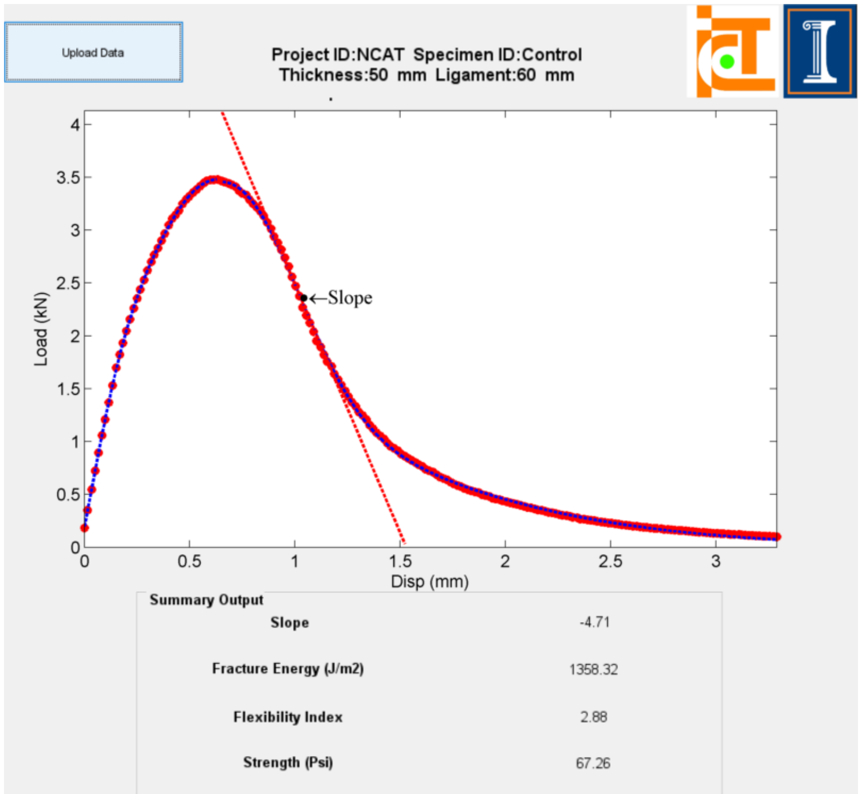

Figure 11 Example of Processed I-FIT Data using UIUC/ICT IL-SCB Analysis Tool

The development of flexibility index threshold values is ongoing, but research conducted for the Illinois Center for Transportation by the University of Illinois at Urbana-Champaign has shown some lab to field comparisons between the FI and field cracking performance (*39*). Comparisons between the FI results from loose mix samples and mixture performance at FHWA's accelerated loading facility (ALF) showed good agreement between the FI and load repetitions to failure of the accelerated sections. For the FHWA ALF, the three poor-performing sections had an FI less than 2, whereas the control section (which was among the top performers) had an FI value of 10. Additionally, some correlation was seen between the FI and cores obtained from nine different IDOT (Illinois DOT) districts. The FI clearly showed the effects of aging on these cores, with a reduction in FI for cores that were from pavements that were more than 10 years old. Sections with FI less than 4 to 5 on the field cores generally exhibited premature cracking. Currently, a very aggressive recommendation of 8 has been given for minimum flexibility index and a preliminary value of 8 has been adopted. However, IDOT stated that the value of 8 was for Illinois' climate only. They recommended other states to review their typical mixes and climate before adopting IDOT's specification.

4 LABORATORY TEST RESULTS AND ANALYSIS

4.1 Binder Properties

Binders were extracted using ASTM D2172 method B (centrifuge) and recovered using ASTM D5404 (rotary evaporation). Recovered binders were tested to determine the performance grade of the material using AASHTO M320. A summary of the test results is given in Table 10. The ΔTc results shown in this table are based on 20-hour PAV aging of base binders and binders recovered from the mixtures. The Tcrit-low values of the binders recovered from the mixtures were compared to the predicted low critical temperature values from LTPPBind Online using the MERRA data for each location. The 98% reliability low critical temperature is -7.7°C for all cases, so despite the relatively high low critical temperatures for the recovered binders from these mixtures, the test sections are not likely to have thermal cracking with the exception of US-29.

Considering the low temperature behavior, the ΔTc parameter represents a means of indexing the non-load associated cracking potential of asphalt binders and it is predicted using the Bending Beam Rheometer (BBR) Stiffness (S) and m-slope (m-value) parameters. As shown in Table 10, all of the mixtures had ΔTc values below -5.0. Therefore, all of them are likely to have block cracking according to the ΔTc parameter limits.

Table 10 Binder Performance Grade

Location	Mix ID	T_{crit} High	RAP/RAS	T_{crit} Int	T_{crit} Low S	T_{crit} Low m	True-Grade	PG	ΔTc
AL-50	101	87.1	35% RAP	22.4	-28.2	-17.2	87.1 - 17.2	82 - 16	-11.0
I-65	201	87.4	35% RAP	23.5	-27.2	-18.8	87.4 - 18.8	82 - 16	-8.4
AL-137	301	79.5	35% RAP	29.9	-24.7	-15.6	79.5 - 15.6	76 - 10	-9.1
US-29	401	105.4	32% RAP, 3% RAS	34.9	-21.8	-7.5	105.4 - 7.5	100 - 4	-14.3
AL-35	501	90.9	35% RAP	26.4	-24.4	-16.0	90.9 - 16.0	88 - 16	-8.4
US-80	601	92.3	40% RAP	30.3	-24.4	-16.1	92.3 - 16.1	88 - 16	-8.2

In addition to determining the performance grade of the binder, multiple stress creep recovery (MSCR) was conducted in accordance with AASHTO MP 19 on the binders recovered from all mixtures. These results are provided in Table 11. All of the recovered binders graded as the highest level of trafficking, “E,” which indicates a high resistance to rutting under extremely heavy traffic. In general, mixtures containing RAP should be very resistant to permanent deformation.

Table 11 Binder Permanent Deformation and Grading Test Results

MSCR @ 64°C		Avg % Recovery		Average Jnr, k / Pa		Diff, % recovery	Diff, % Jnr	MP 19 Grade
Location	Mix ID	100 Pa	3200 Pa	100 Pa	3200 Pa			
AL-50	101	55.9	53.7	0.078	0.082	3.98	4.86	E
I-65	201	25.9	23.9	0.174	0.179	7.67	3.10	E
AL-137	301	40.7	37.8	0.113	0.119	7.05	5.15	E
US-29	401	*	*	*	*	*	*	*
AL-35	501	*	*	*	*	*	*	*
US-80	601	50.4	48.8	0.066	0.068	3.25	3.04	E

* Data not available at the time.

By using the Linear Amplitude Sweep (LAS) test (AASHTO TP 101), the fatigue cracking resistance (measured at the binders’ intermediate PG temperature) was evaluated and the results are presented in Table 12. Currently, the strain values of 2.5 and 5% strain are used to calculate the LAS fatigue life. Overall, there was a narrow range in LAS results from project to project. Overall, the AL-137 can be considered more susceptible to fatigue cracking despite of having the lowest overall critical high temperature.

Table 12 Binder Fatigue Test Results

Location	Mix ID	RAP/RAS	N_f @ Applied Strain	
			2.50%	5.00%
AL-50	101	35% RAP	234,835	4,280
I-65	201	35% RAP	227,520	4,146
AL-137	301	35% RAP	158,593	2,890
US-29	401	32% RAP, 3% RAS	257,235	4,688
AL-35	501	35% RAP	215,641	3,930
US-80	601	40% RAP	184,208	3,357

4.2 Laboratory Performance Test Results

All test results except the energy ratio and SCB-LTRC were checked for outliers in accordance to ASTM E178-08. All results that failed ASTM E178-08 at a significance level of 0.10 were eliminated. Results of the E*testing and the IFIT were analyzed using an ANOVA with a Tukey-Kramer test to stablish statistical groupings. Letters are used to designate statistical groups in the results section. Mixtures that shared a letter were not deemed to be statistically different (i.e. they are considered similar).

Energy ratio results are a product of three tests conducted on the same set of three specimens. For each specimen, resilient modulus and creep compliance results are calculated from data collected from gauges on both faces, giving two results per specimen. For a set of three specimens, this yields six results from which the high and low values are removed to determine trimmed means for the individual properties. Although there are replicates for each test in the ER protocol, a single ER value is calculated from the trimmed means from the component tests. Therefore, statistical analyses of ER results were not possible.

Dynamic Modulus Test Results

While the master curves are not direct indicators of performance, they are used in mechanistic pavement design and can give an indication of relative mixture stiffness and susceptibility to fatigue and permanent deformation. This is particularly useful for mixtures containing RAP where the actual degree of binder blending is unknown. Of the six master curves (Figure 10), the mixture from the AL-137 project was the “softest” at high temperatures and low frequencies. On the other hand, the mixture from the US-29 project exhibited higher dynamic moduli at high temperatures and low frequencies. These results were expected since the AL-137 mixture had the lowest true-grade critical temperature (79.5°C) and the US-29 mixture had the highest true-grade critical temperature (105.4°C). All mixtures tend to have similar dynamic moduli at low temperatures and high frequencies, which are also reflected on the similar master curve coefficients (Max E*) and regression parameters for all mixtures shown in Table 13.

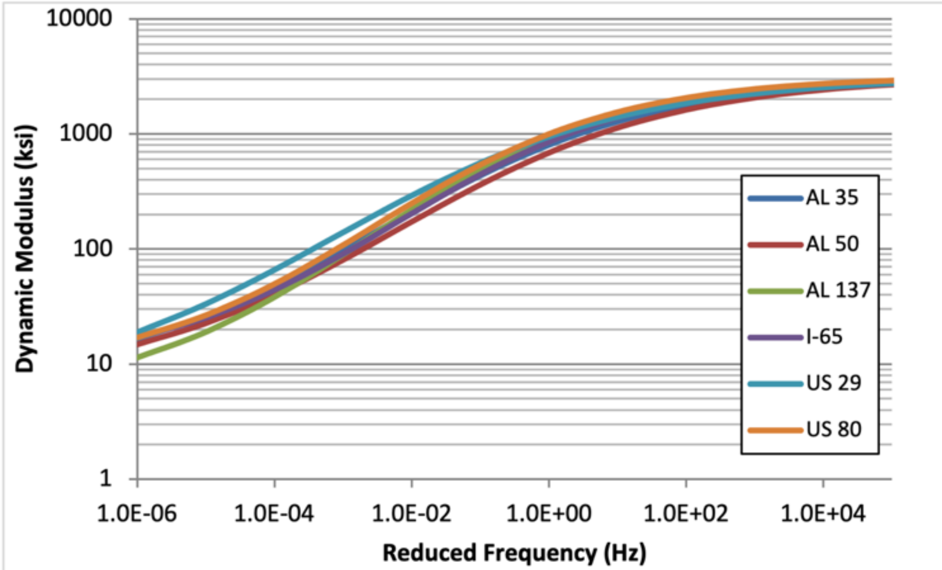

Figure 12 Asphalt Mixture Master Curves

Table 13 Master Curve Regression Parameters

Mix ID	RAP/RAS	Max E* (Ksi)	Min E* (Ksi)	Beta	Gamma	EA	R^2	Se/Sy
AL 50	35% RAP	3,164.8	6.44	-1.116	-0.497	190,572	0.997	0.036
I-65	35% RAP	3,197.8	9.15	-1.235	-0.558	185,575	0.996	0.043
AL 137	35% RAP	3,186.0	4.70	-1.469	-0.554	175,850	0.993	0.059
US 29	32% RAP, 3% RAS	3,138.6	4.88	-1.476	-0.468	195,920	0.997	0.036
AL 35	35% RAP	3,236.8	6.84	-1.224	-0.501	193,880	0.998	0.032
US 80	40% RAP	3,162.0	8.17	-1.428	-0.565	179,978	0.995	0.048

To assess statistical differences, a general linear model (GLM) ($\alpha = 0.05$) was conducted on the test data measured at three temperatures and four frequencies. Thus, the GLM was completed four times to assess statistical differences at each temperature. The Tukey-Kramer test ($\alpha = 0.05$) was used to determine where these statistical differences occurred and how the mixtures grouped within each project.

Table 14 shows the results of the Tukey-Kramer test on E* values for all tested frequencies and temperatures. As expected, at low temperatures and high frequencies, E* values were not statistically different in most cases. The mixture from US-80 was significantly different (higher E* values) from the mixture from AL-50. In addition, the mixture from US-80 tended to have higher E* values at intermediate temperatures, as well. However, the mixture from US-29 had the highest dynamic modulus and was statistically different from several mixtures.

Table 14 Statistical Analysis of Dynamic Moduli

Mix ID	4°C - 10 Hz		4°C - 1 Hz		4°C - 0.1 Hz			
	Mean	**Grouping**	**Mean**	**Grouping**	**Mean**	**Grouping**		
US 80	2669.4	A	2096.1	A	1533.9	A		
AL 137	2528.3	A B	2040.5	A B	1544.8	A		
I-65	2484.8	A B C	1945.2	A B	1412.5	A B		
US 29	2372.9	A B C	1914.9	A B C	1467.3	A B		
AL 35	2252.6	B C	1775.3	B C	1304.3	B C		
AL 50	2153.3	C	1645.7	C	1171.2	C		
Mix ID	20°C - 10 Hz		20°C - 1 Hz		20°C - 0.1 Hz			
	Mean	**Grouping**	**Mean**	**Grouping**	**Mean**	**Grouping**		
US 80	1431	A	919.6	A	534.2	A		
AL 137	1331	A B	844.7	A B	476.4	A B		
I-65	1269	A B	792.1	A B	445.6	B		
US 29	1313	A B	887.7	A B	550.4	A		
AL 35	1206	B C	771.5	B C	443.8	B C		
AL 50	1048	C	644.3	C	360.2	C		
Mix ID	45°C - 10 Hz		45°C - 1 Hz		45°C - 0.1 Hz		45°C - 0.01 Hz	
	Mean	**Grouping**	**Mean**	**Grouping**	**Mean**	**Grouping**	**Mean**	**Grouping**
US 80	372.892	A	166.503	A	72.4077	A B	34.5915	A B
AL 137	360.225	A	149.505	A B	59.3059	B C	26.7498	B
I-65	295.635	B C	126.101	B C	56.8306	B C	31.2846	A B
US 29	351.426	A B	171.29	A	80.7522	A	38.5849	A
AL 35	276.249	C D	118.38	B C	54.8629	B C	31.5457	A B
AL 50	232.737	D	103.243	C	49.4289	C	26.8126	B

In an attempt to identify testing variability and/or non-linearity in the material behavior due to non-compliance to the recommended micro-strain levels, the dynamic modulus and phase angle were averaged for each laboratory's data and plotted in Black Space (*40, 41*). Figure 11 contains the Black Space plots for all the different mixes. It should be noted that all plots show good uniformity in their respective Black Space diagrams, as noted with their R^2 values being greater than 0.94 for a 4th-order polynomial fitted function. Due to the interaction of the asphalt binder with aggregate, the Black Space diagram for a mixture shows a peak phase angle value at intermediate dynamic modulus. At high temperatures, the aggregate structure begins to dominate behavior of the mixture, while volumetric properties and binder stiffness control the behavior at lower temperatures. Higher E* peak values indicate stiffer and less viscous mixtures due to aging (in this case mixtures from US-80 and I-65).

Additional analysis of the Black Space diagram indicates that mixtures with lower phase angle values are more elastic. On the other hand, if the phase angle is high, the mixture is more viscous (*42*). In addition, stiffer mixtures at lower phase angles are more susceptible to cracking (*43*). In this case, the US-80 mixture has slightly higher moduli at low phase angles than the other mixtures.

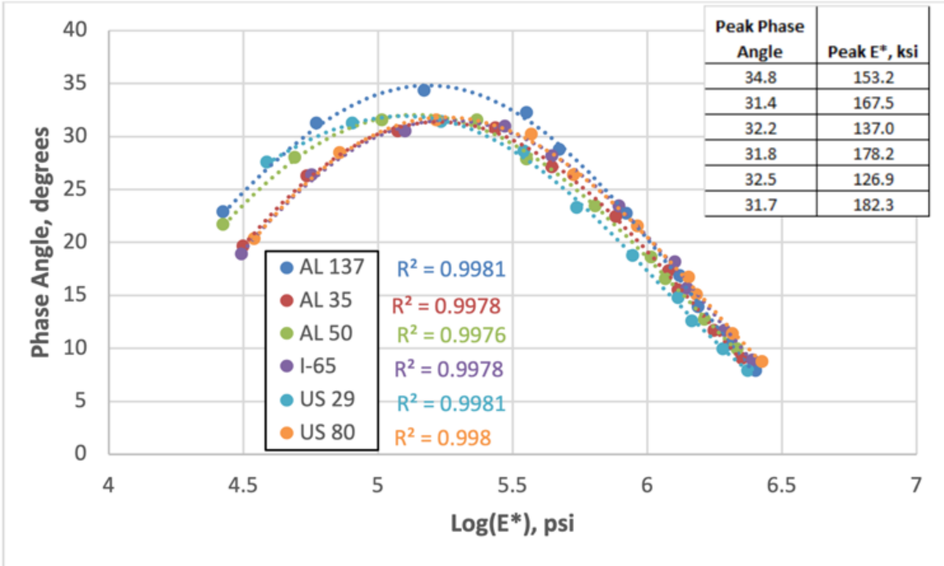

Figure 13 Black-Space Diagram of All Mixtures

Cyclic Fatigue (S-VECD) Test Results

A summary of the results from the individual S-VECD tests is included in Table 15 below. Of the 19 individual specimens tested, 8 had a DMR value in the recommended range (0.9 to 1.1). The remaining specimens had borderline DMR values, indicating a slight disconnect between the mixture E* tested during dynamic modulus and E* verified during the fatigue testing.

The damage characteristic (C vs. S) curves for this project are shown in Figure 12 while the energy release (G^R vs. N_f) curves are shown in Figure 13. A power model of standard form was fit to the G^R versus N_f curves with the model coefficients summarized in Table 16. Figure 12 shows three of the mixtures (AL-50, AL-137, and I-65) to have virtually identical damage characteristic curves, while the US-80 mixture has the greatest stiffness as additional damage is applied to the specimens. The energy release curves all had power model R^2 values of 0.94 or above, indicating a good model fit. The curve with the highest slope and highest intercept was the US-80 mixture. This indicates that at low energy release rates (10 or 100), this mixture has poor fatigue resistance relative to the other mixture designs. Three of the mixtures (AL-50, US-29, and I-65) had virtually identical slopes at the low end of the spectrum, indicating improved fatigue resistance relative to the other mixtures. The AL-50 mixture had the highest intercept of this grouping of three mixtures; it is further to the right of the plot in Figure 13 and has the highest estimated endurance limit (Figure 14), indicating it would be the most fatigue resistant mixture in this grouping.

Table 15 Summary of S-VECD Individual Test Results

Test Name	Temperature (C°)	Frequency (Hz)	$\lvert E^* \rvert_{LVE}$ (MPa)	$\lvert E^* \rvert_{finger}$ (MPa)	Nf	G^R	DMR
ADHR AL 35 #1	20.8	10	8363.4	9688	37301	1.38E+01	1.158
ADHR AL 35 #10	20.9	10	8318.3	9246	3075	3.39E+02	1.112
ADHR AL 35 #11	20.9	10	8363.4	9006	2375	1.16E+03	1.077
ADHR AL 35 #12	20.9	10	8363.4	7641	7315	1.79E+02	0.914
ADHR AL 50 #8	20.9	10	7412.7	6530	24899	7.81E+01	0.881
ADHR AL 50 #9	20.9	10	7371.1	6430	29330	5.20E+01	0.872
ADHR AL 50 #10	21	10	7412.7	6301	5575	4.78E+02	0.850
ADHR AL 137 #6	20.9	10	9816.1	9759	35989	1.34E+01	0.994
ADHR AL 137 #9	20.9	10	9768.8	9275	6255	2.68E+02	0.949
ADHR AL 137 #10	20.9	10	9816.1	10737	32991	2.08E+01	1.094
ADHR AL 137 #13	21	10	9862.9	11067	12229	8.03E+01	1.122
ADHR I65 #3	20.8	10	9101.7	7636	148206	4.40E+00	0.839
ADHR -I65 #5	21	10	9054	8321	3355	1.17E+03	0.919
ADHR I-65 #8	20.9	10	9054	8740	1715	1.47E+03	0.965
ADHR US 29 #5	20.9	10	9214.1	9458	8655	1.84E+02	1.026
ADHR US 29 #13	21	10	9170.2	7684	2435	8.20E+02	0.838
ADHR US 80 #2	21	10	10216.4	8566	87214	4.95E+00	0.838
ADHR US 80 #4	20.9	10	10216.4	8684	3495	1.36E+03	0.850
ADHR US 80 #7	20.9	10	10169	8741	9515	2.05E+02	0.860

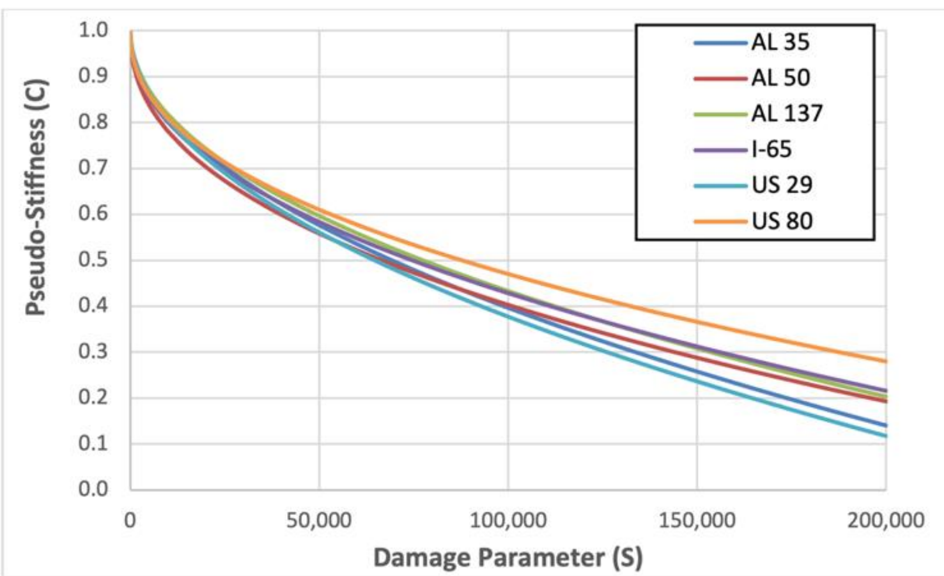

Figure 14 S-VECD: C versus S Curves

Table 16 Summary of S-VECD G^R versus N_f Power Model Coefficients

Variable	AL 35	AL 50	AL 137	I-65	US 29	US 80
y	1.68E-03	4.00E-03	2.00E-03	2.99E-03	1.90E-03	3.25E-03
z	5.11E-01	4.35E-01	4.91E-01	4.56E-01	5.03E-01	4.43E-01
alpha	4.015	3.992	3.592	3.854	4.026	3.786
Parameter r in Failure Crit.	1.96E+05	6.62E+05	1.90E+05	4.40E+05	6.04E+05	2.15E+05
Parameter s in Failure Crit.	-6.53E-01	-7.72E-01	-6.13E-01	-7.27E-01	-8.17E-01	-5.76E-01

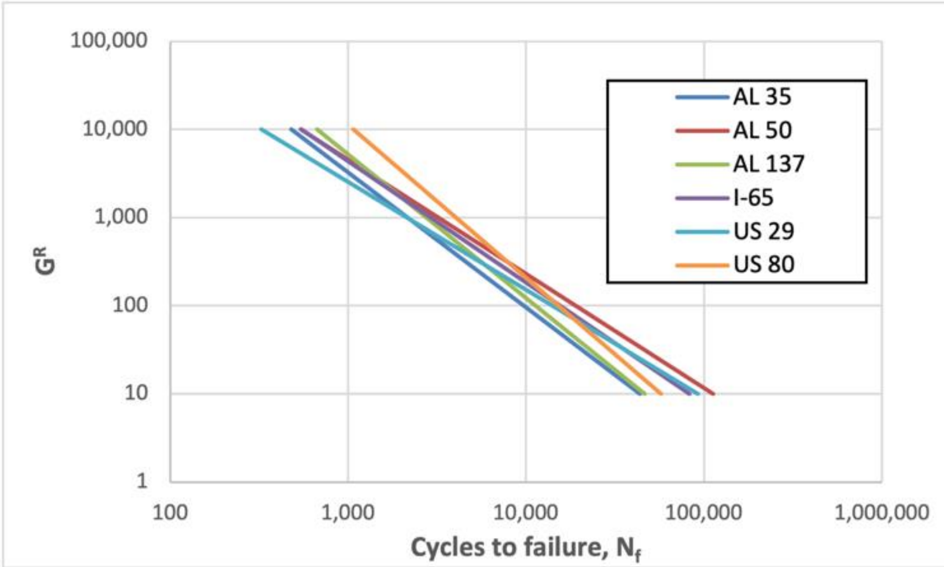

Figure 15 S-VECD: GR versus N_f Curves

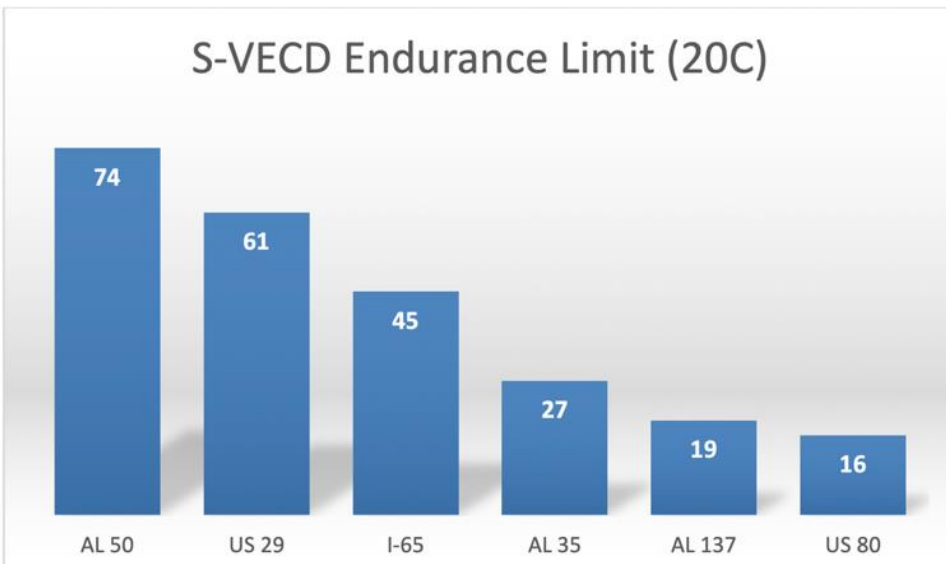

Figure 16 S-VECD Endurance Limit

SCB-LTRC Test Results

Figure 15 shows results of the average strain energy to failure of each notch depth for every mixture. For each mixture, four specimens at three notch depths (25.4, 31.8, and 38.0 mm) are loaded monotonically at a rate of 0.5 mm/minute at a 25°C test temperature until failure. The area under the load-displacement curve to the peak for each specimen is measured and then plotted against the specimen notch depth. Little differences were obtained in terms of slope for all mixtures except AL-35, indicating a mixture more susceptible to loading, notch depths, or both.

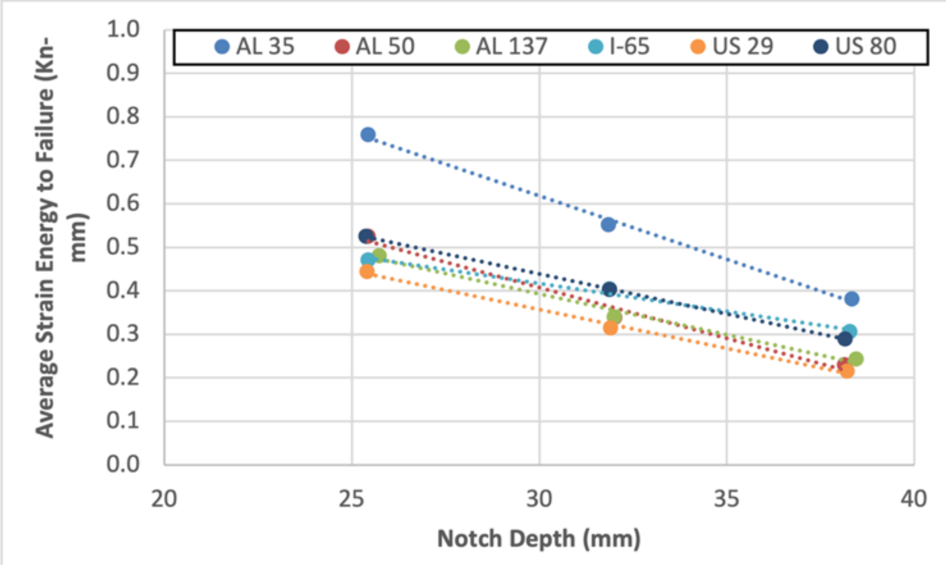

Figure 17 Area versus Specimen Notch Length

Figure 16 shows the results of the semi-circular bending (SCB) tests conducted on re-heated samples. The SCB-LTRC method yields a singular J_c result with a minimum specified value of 0.5 or 0.6 depending on the traffic level. The higher J_c value on AL-137 mixture could indicate an improvement in fracture resistance due to the softer binder.

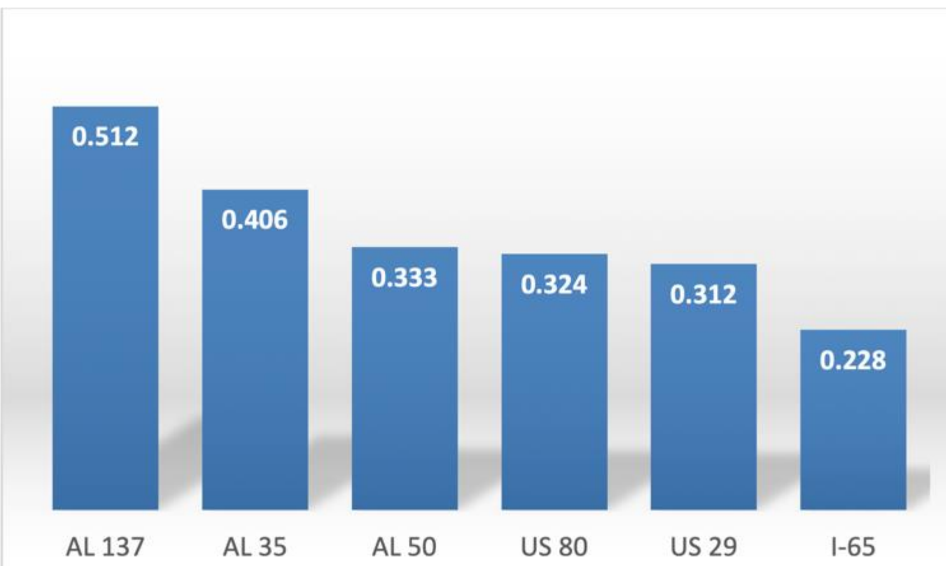

Figure 18 J_c Values

Flexibility Index Test Results

The test results from all mixtures are given in Figure 17. All mixtures had FI values below the preliminary criterion of 8.0, but significant differences were obtained among them. The AL-50 and AL-35 mixtures were statistically the top performers. For practical purposes, all these mixtures could be considered susceptible to fatigue cracking based on the current criteria.

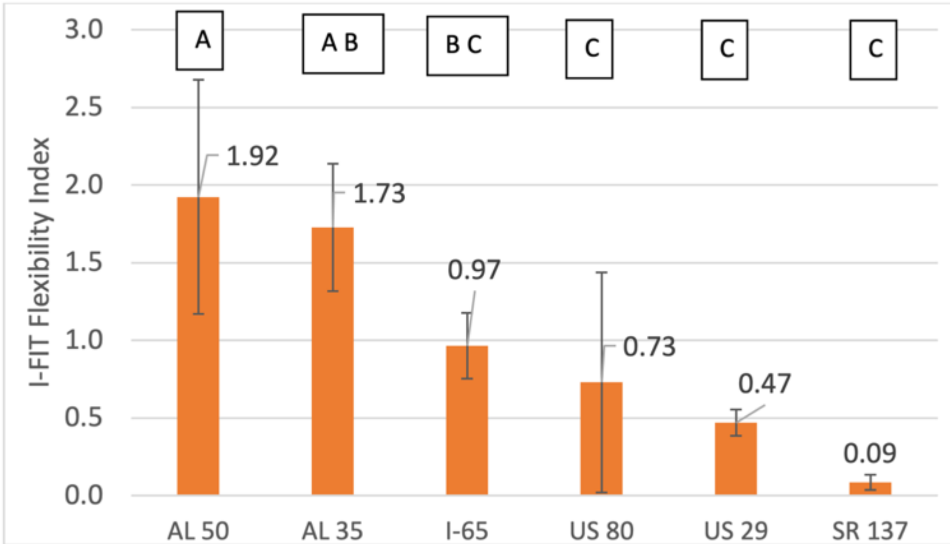

Figure 19 Flexibility Index Results

Energy Ratio Results

The energy ratio intermediate properties and ER values are summarized in Table 17. All mixtures had mean $DCSE_{HMA}$ results within the recommended range of 0.75 to 2.5 kJ/m^3 except for the AL-50 mixture, which exceeded the upper limit. This indicates that these mixtures are not likely to fracture. In addition, all mixtures easily passed the minimum ER criterion of 1.95 for high traffic volume, indicating that these mixtures are not susceptible to top-down cracking. It is important to recall that the ER criteria were based on field cores from pavements that were at least 10 years old (*37*). Whereas, the results analyzed here were from tests on reheated plant mix, which can cause different field performance than expected with this test.

Table 17 Energy Ratio Test Results

	AL 50	I-65	AL 137	US 29	AL 35	US 80
m-value	0.418	0.354	0.356	0.285	0.373	0.328
D_1 (E-07)	4.22	2.70	2.57	3.20	3.74	2.91
S_t (MPa)	2.48	2.66	2.91	2.35	2.38	2.70
M_R (GPa)	12.81	17.07	17.56	14.35	13.89	15.53
FE (kJ/m3)	4.00	1.70	1.20	1.10	1.10	1.90
$DCSE_{HMA}$ (kJ/m3)	3.76	1.49	0.96	0.91	0.90	1.67
A (E-08)	4.62	4.52	4.38	4.70	4.68	4.50
$DCSE_{MIN}$ (kJ/m3)	0.677	0.270	0.270	0.161	0.422	0.233
ER	5.55	5.52	3.55	5.64	2.12	7.14
Creep Rate (E-09)	3.16	1.10	1.07	0.65	1.83	0.92
Failure Strain (FE)	2,038.1	922.8	636.7	665.8	727.1	958.1

4.3 Correlations Among Cracking Test Results

An analysis was conducted to determine how well results of the five cracking tests correlated with one another. This analysis was first conducted using the Pearson product moment correlation, which evaluates the linear relationship between two continuous variables. The result of a Pearson correlation is a coefficient, *R*, that ranges between -1 and +1 where *R* values close to +1 indicate that the two variables are related in a positive and proportional (linear) fashion, *R* values close to -1 indicate that the two variables are inversely related in a proportional fashion, and *R* values closer to zero indicate that the two variables have little to no relationship. In general, *R* values less than -0.8 or greater than +0.8 are considered to indicate strong correlations. The results of the correlation analysis are shown in Table 18.

For the energy ratio test, the three intermediate properties, resilient modulus, creep compliance rate, and dissipated creep strain energy ($DSCE_{HMA}$) were included in analysis as well as the calculated energy ratio. The cells shaded in green indicate the test results that are strongly correlated based on the testing of all mixtures in this study. This shows that E* at 20C-10 Hz is highly correlated to creep rate and flexibility index. In addition, creep rate is highly correlated to flexibility index and $DCSE_{HMA}$. An elementary plot evaluation was conducted to identify any non-linear correlations between the measured parameters. With only six data points it was not possible to identify non-linear relationships.

Table 18 Pearson Correlation Coefficients among Cracking Test Results

	E* 20C, 10 Hz	S-VECD	SCB - J_c	FI	$DCSE_{HMA}$ (kJ/m3)	ER	Creep Rate	M_R
E* 20C, 10 Hz	1							
S-VECD	-0.710	1						
SCB - Jc	0.063	-0.485	1					
FI	-0.802	0.425	-0.249	1				
$DCSE_{HMA}$ (kJ/m3)	-0.686	0.608	-0.281	0.612	1			
ER	0.296	0.243	-0.659	-0.193	0.386	1		
Creep Rate	-0.910	0.506	0.040	0.842	0.832	-0.174	1	
M_R	0.638	-0.590	0.163	-0.749	-0.522	0.010	-0.656	1

Finally, Table 19 shows a ranking analysis to organize all six mixtures from top performer to bottom based on several laboratory cracking related parameters. Most parameters seem to put the AL-50 mixture as the most resistant to cracking and mixture AL-137 as the bottom performer. Notice the parameter observed from the dynamic modulus test also tend to agree with all laboratory results with the exception that the SCB-Jc and the ER results tend to disagree. A combined ranking obtained based on the summation of all individual rankings also helps identify the AL-50 mixture as the top performer and the AL-137 mixture as the potentially poor performer.

Table 19 Ranking Analysis

Mix ID	E* 20C, 10 Hz (ksi)	S-VECD	SCB - J_c	FI	$DCSE_{HMA}$ (kJ/m3)	ER	Creep Rate	M_R (GPa)	Combined Ranking
AL-50	1048	74	0.33	1.92	3.76	5.55	3.16	12.81	
I-65	1269	45	0.23	0.97	1.49	5.52	1.1	17.07	
AL-137	1331	19	0.51	0.10	0.96	3.55	1.07	17.56	
US-29	1313	61	0.31	0.47	0.91	5.64	0.65	14.35	
AL-35	1206	27	0.41	1.73	0.9	2.12	1.83	13.89	
US-80	1431	16	0.32	0.73	1.67	7.14	0.92	15.53	
Individual Ranking									
AL-50	1	1	3	1	1	3	1	1	1
I-65	3	3	6	3	3	4	3	5	3
AL-137	5	5	1	6	4	5	4	6	6
US-29	4	2	5	5	5	2	6	3	4
AL-35	2	4	2	2	6	6	2	2	2
US-80	6	6	4	4	2	1	5	4	5

5 FIELD MIXTURE TEST RESULTS AND ANALYSIS

After five to six years in service, cores were taken from all six sites to conduct mixture characterization, laboratory performance testing, and evaluation of the existing pavement condition. Binders were extracted using ASTM D2172 method B (centrifuge) and recovered using ASTM D5404 (rotary evaporation). Recovered binders were tested to determine the performance grade of the material using AASHTO M320. A summary of the test results is given in Table 20. The most significant change after five to six years in place was the critical low temperature, which tended to increase for most cases (except US-80 and AL-35). In two cases (US-29 and I-65), the critical high temperature significantly increased by more than three PG grade bump, indicating that those mixtures either aged more than the rest, the concentration of RAP was higher for those samples, or both. Small changes in ΔTc were also obtained, indicating that binders became slightly more susceptible to cracking.

Table 20 Binder Performance Grade from Field Cores

Location	Mix ID	RAP/RAS	T_{crit} High	T_{crit} Int	T_{crit} Low S	T_{crit} Low m	True-Grade	PG	ΔTc
AL-50	151	35% RAP	100.8	30.8	-24.1	-11.4	100.8 - 11.4	100 - 10	-12.7
I-65	121	35% RAP	103.1	28.9	-25.2	-14.5	103.1 - 14.5	100 - 10	-10.7
AL-137	101	35% RAP	91.4	33.8	-21.7	-9.5	91.4 - 9.5	88 - 4	-12.3
US 29	111	32% RAP, 3% RAS	103.7	34.1	-16.7	-9.0	103.7 - 9.0	100 - 4	-7.8
AL-35	161	35% RAP	85.6	17.9	-24.1	-19.4	85.6 - 19.4	82 - 16	-4.8
US-80	131	40% RAP	96.0	33.0	-21.8	-7.5	96.0 - 7.5	94 - 4	-14.3

In addition to determining the performance grade of the binder, multiple stress creep recovery (MSCR) was conducted in accordance with AASHTO MP19 on the binders recovered from all mixtures. These results are provided in Table 21. All of the recovered binders graded as the highest level of trafficking, “E,” which indicates a high resistance to rutting under extremely heavy traffic. In general, all mixtures became stiffer and even less susceptible to permanent deformation.

Table 21 Binder MSCR Results from Field Cores

MSCR @ 64°C		Avg % Recovery		Average Jnr, k/Pa		Diff, % recovery	Diff, % Jnr	M332 Grade
Location	Mix ID	100 Pa	3200 Pa	100 Pa	3200 Pa			
AL-137	101	43.21	38.77	0.061	0.067	10.27	8.297	E
US 29	111	75.54	70.88	0.008	0.010	6.166	21.39	E
I-65	121	87.49	82.64	0.008	0.011	5.542	37.48	E
US-80	131	63.86	58.43	0.028	0.032	8.5	15.52	E
AL-50	151	76.25	71.49	0.013	0.015	6.243	21.66	E
AL-35	161	46.01	40.04	0.115	0.129	12.98	11.96	E

The results of LAS binder testing of field core samples are shown in Table 22. Overall, there was a wider range in LAS results compared to samples obtained during production. Slightly higher cycles to failure were observed at 2.5% strain level for mixtures AL-137, US-29, and US-80, and slightly lower cycles were observed at 5.0% strain level for all cases except AL-137. In this case, the AL-35 and I-65 mixtures were the most affected by aging and became more susceptible to fatigue cracking.

Table 22 Binder Fatigue Results from Field Cores

Location	Mix ID	RAP/RAS	N_f @ Applied Strain	
			2.50%	**5.00%**
AL-50	151	35% RAP	598,900	1,895
I-65	121	35% RAP	177,480	1,881
AL-137	101	35% RAP	249,221	3,794
US 29	111	32% RAP, 3% RAS	294,943	2,930
AL-35	161	35% RAP	166,271	2,293
US-80	131	40% RAP	197,571	2,815

Small E* samples were obtained from field cores and the master curves for all six mixtures are shown in Figure 18. Small E* samples are 38 mm in diameter and have a height of 110 mm. In this case, the difference among mixtures is more evident in the curves and in the regression parameters (Table 23). The mixture from AL-137 has higher E* vales for most temperatures and frequencies, followed by mixture AL-50. On the other hand, mixture US-80 has significantly lower E* values at high temperatures and low frequencies, which could be attributed to damage of the sample and/or larger air voids contents compared to samples prepared during production. The tendency for field samples was to have 2% to 3% air voids above production samples, except for the AL-137 mixture where no difference in air voids was obtained.

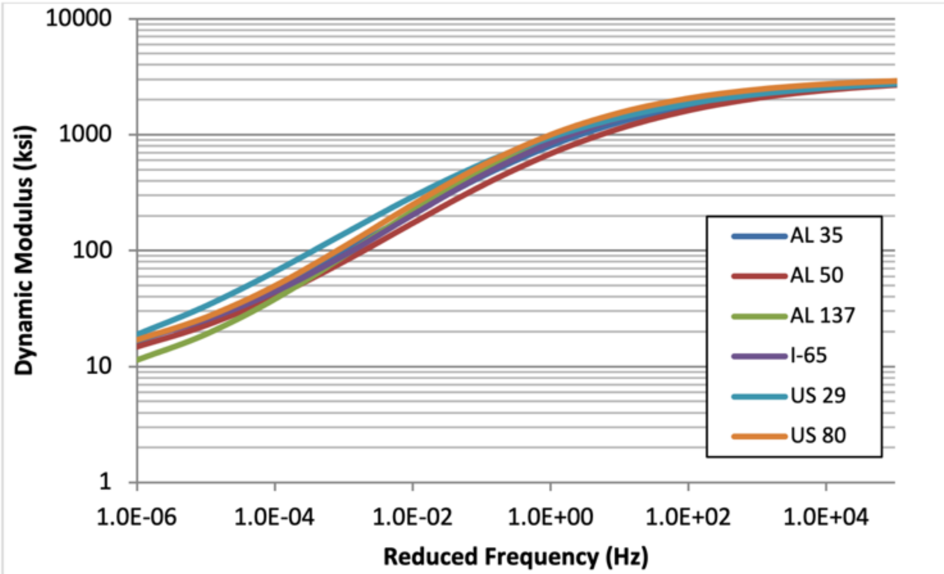

Figure 20 Small Samples Master Curve from Field Cores

Table 23 Small Samples Master Curve Regression Parameters

Mix ID	RAP/RAS	Max E* (Ksi)	Min E* (Ksi)	Beta	Gamma	EA	R^2	Se/Sy
AL 50 FC	35% RAP	2,946.1	6.20	-1.250	-0.409	214,752	0.998	0.034
I-65 FC	35% RAP	2,936.9	4.74	-0.840	-0.419	198,115	0.999	0.022
AL 137 FC	35% RAP	3,209.9	7.13	-1.813	-0.541	209,015	0.983	0.092
US 29 FC	32% RAP, 3% RAS	2,866.9	0.06	-1.435	-0.262	177,194	0.994	0.056
AL 35 FC	35% RAP	3,019.9	6.77	-0.620	-0.486	193,012	0.997	0.038
US 80 FC	40% RAP	2,875.7	0.05	-1.554	-0.354	195,808	0.996	0.047

To further evaluate the effect of aging, larger air voids, and potential damage of the mixture due to traffic loading, a Black Space diagram was created (Figure 19). In all mixtures, phase angles at high temperatures and low frequencies increased, suggesting that mixtures became less elastic. In addition, a reduction of the peak E* values and peak phase angles suggests a reduction in mixture stiffness. Based on the changes in Black Space curve peak values, the most affected mixture was US-80 and the least affected mixtures were AL-50 followed by AL-137.

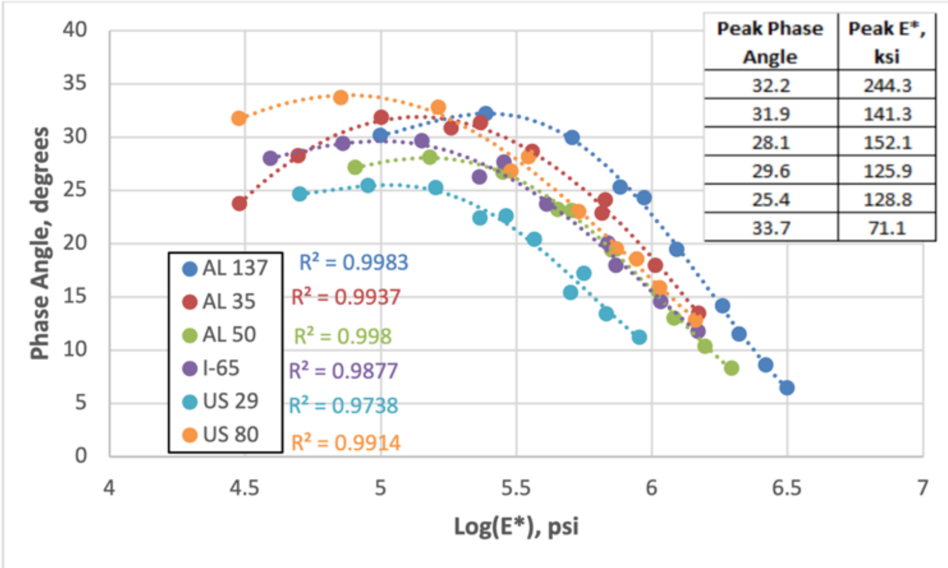

Figure 21 Black-Space Diagram from Field Cores

Flexibility index was obtained from field cores, and the test results from all mixtures are given in Figure 22. All mixtures had FI values below the preliminary criterion, but there were significant differences; mixtures AL-35 and I-65 were statistically the top performers, and the remaining mixtures were statistically different. For all mixtures but AL-50, FI values were slightly higher for field cores than samples prepared during production. This could be due to material variability and differences in the thickness of the sample, despite the application of a correction factor. For practical purposes, all of these mixtures could be considered susceptible to fatigue cracking based on the current criteria. For all sections, these results were higher (almost double) than laboratory compacted samples. Core samples were thinner compared to laboratory produced samples and had higher air voids that ranged from 7.0% to 12.4%.

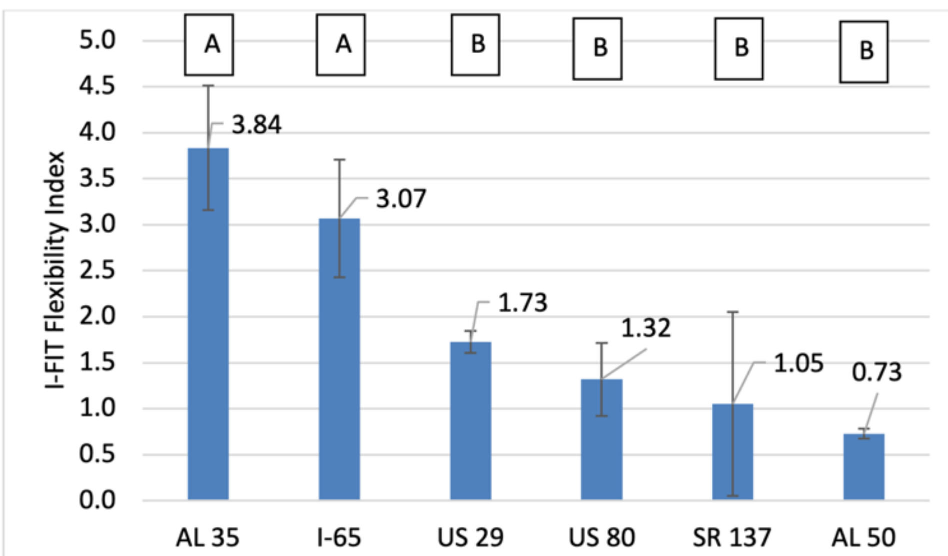

Figure 22 Flexibility Index from Field Cores

Field Performance after Six Years in Service

A visit was conducted at each site in 2017 after the mixtures had been in service approximately five to six years. Six cores were taken from between the wheelpaths during the visit. The cores were taken from the location of the sample truck, which was marked by GPS during construction. Each section was inspected to assess performance and current pavement condition. Detailed results of these evaluations are exhibited in Appendix A and a summary of field performance is shown in Table 24. In most cases, little to no-cracking was observed. The only site that showed signs of fatigue cracking in both wheelpaths was US-29. These results could be explained by the use of RAS in this project, which resulted in significantly higher recycled binder content and more overall oxidized binder.

Table 24 Summary of Field Performance of New Projects

Mix ID/Location	Mix Variables	Age, Years	Field Performance
AL-50 Lafayette, Chambers County	35% RAP Fine-graded 12.5-mm NMAS Water injection (ASTEC)	6	Low-severity transverse cracking
I-65 Calera, Shelby County	35% RAP Fine-graded 19.0-mm NMAS Water injection (Gencor)	6	No cracking or other distresses
AL-137 Wing, Covington County	35% RAP Fine-graded 12.5-mm NMAS Water injection	6	No cracking or other distresses
US-29 Troy, Pike County	32% RAP/ 3% RAS Fine-graded 12.5-mm NMAS Water injection	6	Low-severity fatigue cracking on both wheelpaths
AL-35 Fort Payne, Cherokee County	35% RAP Fine-graded 19.0-mm NMAS Evotherm 3G	6	Low-severity raveling
US-80 Lowndes, Lowndes County	40% RAP Fine-graded 12.5-mm NMAS Evotherm 3G	5	Low-severity fatigue cracking

6 CONCLUSIONS AND PROPOSED ACTIONS

Based on the results of this research project, the following conclusions can be made.

- No issues were found during production and construction of the different mixtures utilized in this project. However, results from field cores obtained after six years in service indicated that almost all mixtures had 2% to 3% higher air voids than the expected 7% target during construction.
- All of the mixtures used in this study were either binder or bottom asphalt layers, neither of which were directly exposed to environmental conditions and traffic loading. Therefore, it is expected that mixture properties and performance should not affected as much as if they were surface layers.
- An analysis of binder properties for all mixtures indicated that only one mixture was likely to experience thermal cracking (US-29), and all mixtures were highly susceptible to

block cracking. In addition, all mixtures were classified with the highest level of trafficking, indicating low permanent deformation susceptibility. LAS binder test results were similar at the two evaluated strain levels and these results did not agree with the expected fatigue behavior based on binder performance grading.

- Dynamic modulus test results performed on reheated mix indicated that all mixtures tend to behave similarly within the range of frequencies and temperatures evaluated. The largest differences in E* values were found at high temperatures where the aggregate structure began to dominate the behavior of the mixture.
- Cyclic fatigue (S-VECD) test results performed on reheated mix indicated that all mixtures follow a similar damage behavior. However, this test method was sensitive to the small differences in mixture properties. Three of the mixtures (AL-50, US-29, and I-65) had virtually identical slopes at the low end of the spectrum, indicating improved fatigue resistance relative to the other mixtures.
- SCB-LTRC rest results performed on reheated mix indicate that all mixtures except AL-137 failed the criteria based on low traffic level. Therefore, five of six mixtures were considered susceptible to fatigue cracking.
- Flexibility index test results performed on reheated mix indicate that all of these mixtures could be considered susceptible to fatigue cracking based on the current criteria. However, this test method was sensitive to the small differences in mixture properties (AL-50 and AL-35 mixtures were statistically the top performers).
- Energy ratio test results performed on reheated mix indicate that all of these mixtures were considered not susceptible for top-down cracking. In addition, the individual parameters DSCE and creep rate showed strong correlation and followed the same trend of other cracking indices such as FI.
- A ranking analysis performed on reheated mix laboratory performance tests indicated that most parameters seemed to put the AL-50 mixture as the most resistant to cracking and mixture AL-137 as the bottom performer. However, two parameters SCB-J_c and the ER results did not follow the same trend.
- Mixture characterization and performance test results performed on field cores indicated that most mixtures were affected by aging and traffic loading. Most mixtures showed significant differences in the binder performance grade due to aging. Most mixtures had lower dynamic moduli within the evaluated range of frequencies and temperatures due to traffic loading effect and environmental conditions (damage).
- Flexibility indices obtained from field cores did not agree with the indices obtained from reheated mix samples. In most cases, the trend was to have higher FI values from cores, which was unexpected. Differences in air voids and sample thickness could be responsible for these results.
- An evaluation of the current pavement condition indicates that most pavement structures have little to no cracking except for the site on US-29. However, there is not

enough evidence at this point to conclude that cracking was due to the presence of the high RAP/RAS content mixture.

- Overall, no detrimental effect of using high RAP content mixtures with WMA technologies was observed in the field. The fact that these mixtures were not built as surface layers could be the main reason why the field pavement structures have only shown low severity cracking. Even though laboratory performance test results indicated that most mixtures were classified as cracking susceptible, after six years in service that was not the case. Therefore, the location of mixtures with high content of recycled material in the pavement structure other than the surface seems to be the best option to minimize their potential cracking susceptibility.
- Condition of the evaluated mixtures could not be determined over time since the methodology used was based on superficial evaluation of the pavement. Therefore, for future projects where the mixture being studied is not the surface layer, a non-destructive pavement evaluation technique such as FWD testing may be suitable to account for the structural condition of that particular asphalt layer.
- Finally, asphalt mixture design using recycled materials is still a challenge, specifically due to the uncertainty of the level of activation of the recycled binder and the level of blending between aged and new binder. Most asphalt mix design experts agree that the ultimate solution to solving many of the unknown impacts of recycled materials and other additives on asphalt mixtures is to implement the use of mixture performance tests in mix design and quality assurance. Although a few state DOTs have already implemented a Balanced Mix Design approach with some of the performance tests used in this study, most highway agencies are uncertain as to which performance tests and what criteria should be used in their specifications. NCHRP Project 20-07, Task 406 is currently developing a framework for Balanced Mix Design and identifying research to address knowledge gaps. Several studies are also underway to help determine which performance tests provide a good relationship to field performance and are suitable for implementation.

7. REFERENCES

1. Kandhal, P. S., and S. S. Rao. Performance of Recycled Hot Mix Asphalt Mixtures. NCAT Report 95-01. National Center for Asphalt Technology, Auburn, Ala., 1995.
2. National Asphalt Pavement Association, http://www.hotmix.org.
3. McDaniel, R. S., H. Soleymani, R.M. Anderson, P. Turner, and R. Peterson. Recommended Use of Reclaimed Asphalt Pavement in the Superpave Mix Design Method. Contractor's Final Report, NCHRP Web Document 30 (Project 09-12), 2000.
4. Huang, B., G. Li, D. Vukosavljevic, X. Shu, and B. K. Egan. Laboratory Investigation of Mixing Hot-Mix Asphalt with Reclaimed Asphalt Pavement. *Transportation Research Record, No. 1929*, Transportation Research Board of the National Academies, Washington, D.C., 2005, pp. 37–45.
5. Bonaquist, R. Can I Run More RAP? *Hot Mix Asphalt Technology*, Vol. 12, No.5, National Asphalt Pavement Association, September/October 2007.
6. Mogawer, W. S., T. Bennert, J. S. Daniel, R. Bonaquist, A. Austerman, and A. Booshehrian. Performance Characteristics of Plant Produced High RAP Mixtures. *Presented at the 87th Annual Meeting of the Association of Asphalt Paving Technologists*, Austin, Texas, 2012.
7. McDaniel, R. S., A. Shah, and G. Huber. Investigation of Low- and High-Temperature Properties of Plant-Produced RAP Mixtures. FHWA-HRT-11-058, Federal Highway Administration, 2012.
8. Michael, J. E. Evaluation of Non-Solvent Methods to Characterize Reclaimed Asphalt Pavement Binder. Thesis, Auburn University, 2011.
9. Turner, P. A Solventless Method for Estimating Asphalt Binder Properties of Recycled Asphalt Pavement. Master's Thesis, Auburn University, 2012.
10. Scholz, Todd V. Preliminary Investigation of RAP and RAS in HMAC. Final Report 500-291, 2010.
11. Wu, S., J. Qiu, L. Mo, J. Yu, Y. Zhang, and B. Li. Investigation of Temperature Characteristics of Recycled Hot Mix Asphalt Mixtures. *Resources, Conservation, and Recycling*, Vol. 51, No. 3, 2007, pp. 610–620.
12. West, R., R. Willis, M. and Marasteanu. Improved Mix Design, Evaluation, and Materials Management Practices for Hot Mix Asphalt with High Reclaimed Asphalt Pavement Content. Final Report, NCHRP 752 (Project 9-46).
13. Li X., T. R. Clyne, and M. O. Marasteanu. Recycyled Asphalt Pavement (RAP) Effects on Binder and Mixture Quality. Minnesota Department of Transportation, 2004.
14. Daniel, J. S., and A. Lachance. Mechanistic and Volumetric Properties of Asphalt Mixtures with Recycled Asphalt Pavement. In *Transportation Research Record 1929*, TRB, National Research Council, Washington, D.C., 2005, pp. 28–36.
15. Willis, R., D. Timm, R. West, B. Powell, M. Robbins, A. Taylor, A. Smit, N. Tran, M. Heitzman, and A. Bianchini. Phase III NCAT Test Track Findings. NCAT Report 09-08. National Center for Asphalt Technology, Auburn, Ala., 2009.
16. Huang, B., W. R. Kingery, and Z. Zhang. Laboratory Study of Fatigue Characteristics of HMA Mixtures Containing RAP. *Presented at the International Symposium on Design and Construction of Long Lasting Asphalt Pavements*, Auburn, Alabama, 2004.

17. Puttagunta, R., S. Y. Oloo, and A. T. Bergan. A Comparison of the Predicted Performance of Virgin and Recycled Mixes. *Canadian Journal of Civil Engineering*, Vol. 24, National Research Council of Canada, Ottawa, 1997. pp. 115–121.
18. Sargious, M., and N. Mushule. Behaviour of Recycled Asphalt Pavements at Low Temperatures. *Canadian Journal of Civil Engineering*, Vol. 18, National Research Council of Canada, Ottawa, 1991. pp. 428–435.
19. Vargas-Nordcbeck, A. Evaluation of the use of Reclaimed Asphalt Pavement in Stone Matrix Asphalt Mixtures. Thesis, Auburn University, 2007.
20. Hajj, E. Y., P. E. Sebaaly, L. Loria, S. Kass, and T. Liske. Impact of High RAP Content on the Performance Characteristics of Asphalt Mixtures in Manitoba. Presented at *2011 Annual Conference of the Transportation Association of Canada,* Innovative Developments in Sustainable Pavements Session, 2011.
21. West, R., A. Kvasnak, N. Tran, R. Powell, and P. Turner. Testing of Moderate and High RAP Content Mixes: Laboratory and Accelerated Field Performance at the National Center for Asphalt Technology Test Track. Transportation Research Record. No. 2126, Transportation Research Board of the National Academies, Washington, D.C., 2009, pp.100–108.
22. Willis, J. R., P. Turner, G. Julian, A. J. Taylor, N. Tran, and F. D. G. Padula. Effects of Changing Virgin Binder Grade and Content on RAP Mixture Properties. NCAT Report 12-03, National Center for Asphalt Technology, Auburn, Ala., 2012.
23. Tran, N., A. Taylor, and J. R. Willis. Effect of Rejuvenator on Performance Properties of HMA Mixtures with High RAP and RAS Contents. NCAT Report 12-05, National Center for Asphalt Technology, Auburn, Ala., 2012.
24. Paul, H. R. Evaluation of Recycled Projects for Performance. *Journal of the Association of Asphalt Paving Technologists*, Proceedings, Technical Sessions, Baltimore, Maryland, 1996, pp. 18–20.
25. Peters, A. J., R. H. Gietz, and J. P. Walter. Hot Mix Recycling Evaluation in Washington. Report WA-RD-98.1, Washington State Department of Transportation, Olympia, 1986.
26. West, R., A. Kvasnak, N. Tran, B. Powell, and P. Turner. Laboratory and Accelerated Field Performance Testing of Moderate and High RAP Content Mixes at the NCAT Test Track. 88th Annual Meeting of the Transportation Research Board, 2009.
27. Rorrer, T., A. Appea, and T. Clark. Case Studies on Processes Involved in the Production and Placement of High RAP Asphalt Concrete Mixes in 2007 on Selected Routes in Virginia. 88th Annual Meeting of the Transportation Research Board, 2009.
28. Shoenberger, J. E., and T. A. Demoss. Hot-mix Recycling of Asphalt Concrete Airfield Pavements. *International Journal of Pavement Engineering*, Vol. 6, No. 1, 2005, pp. 17–26.
29. Hachiya, Y., Y. Tsubokawa, and O. Takahashi. Laboratory and Field Studies of Recycled Asphalt Concrete for Airport Pavement Surface Courses. *International Society for Asphalt Pavements, 10th International Conference on Asphalt Pavements*, Quebec City, 2006.
30. Cline, G. D., and M. C. Hironaka. Performance of Recycled Asphalt Concrete Airport Pavement Surfaces. Final Report. Naval Civil Engineering Laboratory, Port Hueneme, California, 1986.
31. Prowell, B., and C. Hurley. Warm Mix Asphalt: Best Practices. National Asphalt Pavement Association, Lanham, Md., 2007.

32. Mallick, R. P. Kandhal, and R. Bradbury. Using Warm-Mix Asphalt Technology to Incorporate High Percentage of Reclaimed Asphalt Pavement Material in Asphalt Mixtures. *Transportation Research Record: Journal of the Transportation Research Board, No. 2051,* Transportation Research Board of the National Academies, Washington, D.C., 2008, pp. 71–79.
33. Wielinski, J., A. Hand, and D. Rausch. Laboratory and Field Evaluations of Foamed Warm-Mix Asphalt Projects. *Transportation Research Record: Journal of the Transportation Research Board*, No. 2126, Transportation Research Board of the National Academies, Washington, D.C., 2009, pp. 125–131.
34. Anderson, R. M., G. N. King, D. I. Hanson, and P. B. Blankenship. Evaluation of the Relationship Between Asphalt Binder Properties and Non-Load Related Cracking. Journal of the Association of Asphalt Paving Technologists, Vol. 80, 2011, pp. 615–663.
35. Underwood S., R. Kim, and C. Baesk. Simplified Viscoelastic Continuum Damage Model as Platform for Asphalt Concrete Fatigue Analysis. *Transportation Research Record: Journal of the Transportation Research Board,* No. 2296, Transportation Research Board of the National Academies, Washington, D.C., 2012, pp. 36–45.
36. Hou, T., B. S. Underwood, and Y. R. Kim. Fatigue Performance Prediction of North Carolina Mixtures Using the Simplified Viscoelastic Continuum Damage Model. *Journal of the Association of Asphalt Paving Technologists*, Vol. 79, 2010, pp. 35C80.
37. Kim R., B.S. Underwood, and M. Guddati. Development of a Multiaxial Viscoelastoplastic Continuum Damage Model for Asphalt Mixtures. Report FHWA-HRT-08-073, 2009.
38. Roque, R., B. Birgisson, C. Drakos, and B. Dietrich. Development and Field Evaluation of Energy-Based Criteria for Top-down Cracking Performance of Hot Mix Asphalt. *Journal of the Association of Asphalt Paving Technologists*, Vol. 73, 2004, pp. 229–260.
39. Al-Qadi, I., H. Ozer, J. Lambros, A. El Khatib, P. Singhvi, T. Khan, J. Rivera Pérez, and B. Doll. *Testing Protocols to Ensure Performance of High Asphalt Binder Replacement Mixes using RAP and RAS*. Illinois Center for Transportation Series No. 15-017, Illinois Center for Transportation, University of Illinois at Urbana-Champaign, 2015.
40. Airey G.D., Use of Black Space Diagrams to Identify Inconsistencies in Rheological Data. Journal of Road Materials and Pavement Design. Vol. 3, No. 4, 2002, pp 403–424.
41. Christensen, D. W., T. Pellinen, and R. Bonaquist. Hirsch Model for Estimating the Modulus of Asphalt Concrete. *Journal of the Association of Asphalt Paving Technologists*, Vol. 72, 2003, pp. 97–121.
42. Rastegar, R. Plant versus Laboratory Production: Impact on Measure Properties of Mixtures with RAP and RAS. *Transportation Research Board 2016 Annual Meeting Presentation*, Washington, D.C., 2016.
43. Rowe, G. Evaluation of the Relationship Between Asphalt Binder Properties and Non-Load Related Cracking. *Journal of the Association of Asphalt Paving Technologists*, Vol. 80, 2011, pp. 649–662.

APPENDIX Production, Construction and Pavement Condition

AL 50 in Chambers County

A field test section was placed on AL 50 in Chambers County in March 2011. This test section contained 35% RAP and was placed as the binder layer by East Alabama Paving. The mix used was a 19.0mm maximum aggregate size (MAS) Superpave mix design using a compactive effort of 60 gyrations. The mix was produced using the foaming warm mix technology with an Astec Double Barrrel Green plant. Table 25 shows the aggregate percentages from mix design.

Table 25 Aggregate Percentages Used on AL 50

Aggregate Type	Mix Design (%)
Limestone #78s	28
Limestone Screenings	6
Granite M10s	8
Sand	22
Baghouse Fines	1
RAP	35

A PG 67-22 supplied by Hunt Refining Company was used as the base binder for this mix, with 0.25% Adhere HP+ added as an antistrip agent. Water was injected at a rate of 2.0% by virgin binder. The design aggregate gradation, optimum asphalt content, design volumetrics, and specifications are shown in Table 26.

Table 26 Design Gradation, Asphalt Content, and Volumetrics for Mix Design

Sieve Size, mm (in.)	Mix Design	Specifications
	% Passing	
19.0 (3/4")	100	100 max
12.5 (1/2")	96	90 – 100
9.5 (3/8")	85	90 max
4.75 (#4)	62	--
2.36 (#8)	49	28 – 58
1.18 (#16)	40	--
0.6 (#30)	28	--
0.3 (#50)	15	--
0.15 (#100)	9	--
0.075 (#200)	5.6	2 – 10
AC, %	5.1	5.1 minimum
Air Voids, %	4.0	--
VMA, %	15.2	> 14.5
D/A Ratio	1.16	0.6 – 1.2

Production

The mix was produced at East Alabama Paving’s Opelika plant. This plant is an Astec Double Barrel Green Plant that uses water injection as the WMA technology. Figure 23 shows a general overview of the asphalt plant used by East Alabama Paving for this field demonstration.

Figure 23 East Alabama Paving's Astec Double Barrel Green Plant

Production temperatures were monitored and recorded throughout production of the mix. Table 27 shows production temperature information.

Table 27 Production Temperatures for AL 50 Site

	Temperature, °F
Target	275-280
Average	275.6
Standard Deviation	9.8
Max	294
Min	260

Volumetric Mix Properties

NCAT personnel collected several buckets during production in order to fabricate a variety of reheated performance specimens in the main laboratory. These results are presented later in this report. The samples were also extracted in the NCAT lab and the recovered binder was graded. When the sample was taken at the plant, the truck was noted and communicated to NCAT personnel on the roadway. The location of the sample truck was then marked using GPS for future revisits to the site.

The asphalt content of each mix was determined both by ignition furnace (AASHTO T308), and by TCE extraction (AASHTO T164). The liquid binder was also recovered and graded after TCE extraction. The gradation of each sample was also determined for both extractions and ignitions per AASHTO T30. Table 28 shows the results from NCAT's gradation, asphalt content, and binder grading testing. The contractor's quality control (QC) data was also collected from the sample nearest to the sample truck and can be seen in Table 29.

Table 28 Gradation, Asphalt Content, and Binder Grades at Construction for AL 50

	JMF	Extraction (AASHTO T164)	Ignition (AASHTO T308)	Specifications
Sieve Size	**% Passing**			
19.0 mm (3/4")	100	99.1	100.0	100 max
12.5 mm (1/2")	96	96.6	97.7	90 – 100
9.5 mm (3/8")	85	88.2	88.3	90 max
4.75 mm (#4)	62	65.3	66.2	--
2.36 mm (#8)	49	50.0	51.0	28 – 58
1.18 mm (#16)	40	39.0	39.8	--
0.60 mm (#30)	28	28.1	29.0	--
0.30 mm (#50)	15	16.2	17.0	--
0.15 mm (#100)	9	9.3	10.2	--
0.075 mm (#200)	5.6	5.6	6.5	2 – 10
AC, %	5.1	4.9	5.1	5.1 minimum
True Binder Grade	NA	87.1 - 17.2	NA	NA
PG Grade	NA	82 - 16	NA	NA
ΔT_c	NA	-11.0	NA	NA

Table 29 Contractor's Quality Control Results for AL 50

Sieve Size, mm (in.)	Mix Design	Contractor's QC	Specifications
	% Passing		
19.0 (3/4")	100	100	100 max
12.5 (1/2")	96	96	90 – 100
9.5 (3/8")	85	91	90 max
4.75 (#4)	62	67	--
2.36 (#8)	49	52	28 – 58
1.18 (#16)	40	41	--
0.6 (#30)	28	28	--
0.3 (#50)	15	16	--
0.15 (#100)	9	9	--
0.075 (#200)	5.6	5.8	2 – 10
AC, %	5.1	5.25	5.1 minimum
Air Voids, %	4.0	3.44	--
VMA, %	15.2	14.9	> 14.5
D/A Ratio	1.16	1.17	0.6 – 1.2

Construction

The high RAP test section was placed on AL 50 in Chambers County in the westbound lane as the binder layer at a target thickness of 1.75 inches. This yielded a haul distance of approximately 20 miles, or 25-35 minutes. Figure 24 shows the location of the test section and plant. The asphalt mix was delivered to the paving site using 10-11 trucks. Once on site, a RoadTec SB-2500C MTV was used to transfer the mixes to the IR Blaw-Knox PF-3200 paver, which can be seen in Figure 25.

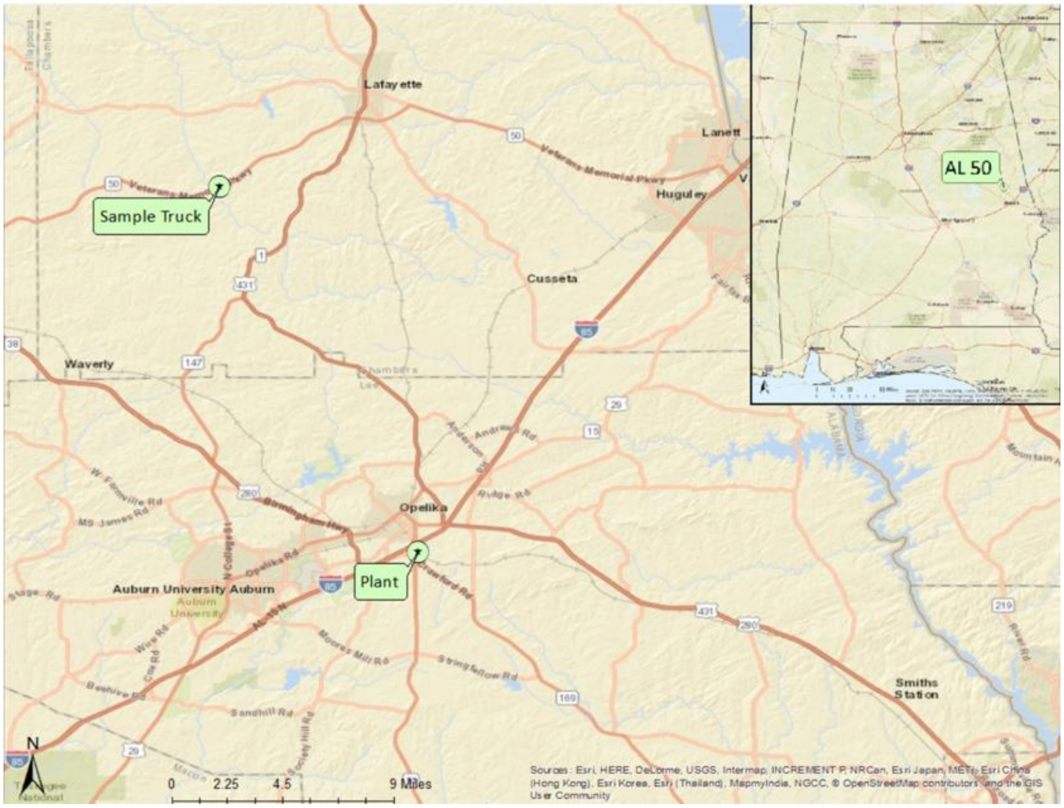

Figure 24 Location of Test Section on AL 50

Figure 25 MTV Transferring Mix to Paver on AL 50

The temperature of the mix behind the paver was measured periodically using a hand-held temperature gun. The temperatures measured behind the screed are shown in Table 30.

Table 30 Temperatures Behind the Screed on AL 50

	Temperature, °F
Average	243.3
Standard Deviation	14.6
Max	258
Min	198

Two rollers were used for compaction. The breakdown roller was a Dynapac CC522 VHF operated in the vibratory mode, while the finishing roller was a Hypac C350D operated in static mode. Figure 26 shows both rollers used for this site.

Figure 26 Breakdown and Finishing Rollers Used on AL 50

Field Performance After Six Years in Service

A site visit was conducted on July 19, 2017 after the pavement had been in service approximately six years. Six cores were taken from between the wheelpaths during the site visit and brought back to NCAT for further testing. During coring operations, the area of pavement where the sample truck was placed was assessed. Several low-severity transverse cracks were observed in this location. After driving the job, these transverse cracks seem typical of the project. There was no rutting apparent in the section. Figure 27 shows an example of the transverse cracking observed after six years.

Figure 27 Low-Severity Transverse Cracking Observed on AL 50

I-65 in Shelby County

In late March 2011, a field test section was placed on I-65 northbound in Shelby County. This section contained 35% RAP and was placed as the upper binder layer by Wiregrass Construction Company, Inc. The mix was a 25.0 mm maximum aggregate size (MAS) Superpave mix design using a compactive effort of 60 gyrations. The mix was produced as WMA using the Gencor Green Machine foaming system. Table 31 shows the aggregate percentages from mix design.

Table 31 Aggregate Percentages Used on I-65

Aggregate Type	Mix Design (%)
#67 Limestone	25
#78 Limestone	9
1/4" Limestone	7
#89 Gravel	5
Sand	18
Baghouse Fines	1
RAP	35

The base binder for this mix was a PG 67-22 from Ergon. Adhere HP was added as an antistrip agent at a rate of 0.25%. Water was injected at a rate of 2.0% by virgin binder using the Gencor Green Machine. The design aggregate gradation, optimum asphalt content, design volumetrics, and specifications are shown in Table 32.

Table 32 Design Gradation, Asphalt Content, and Volumetrics for Mix Design

Sieve Size, mm (in.)	Mix Design	Specifications
	% Passing	
25.0 (1")	100	100 max
19.0 (3/4")	99	90 – 100
12.5 (1/2")	88	90 max
9.5 (3/8")	75	--
4.75 (#4)	55	--
2.36 (#8)	41	23 – 49
1.18 (#16)	33	--
0.6 (#30)	23	--
0.3 (#50)	12	--
0.15 (#100)	7	--
0.075 (#200)	4.0	2 – 8
AC, %	4.4	4.4 minimum
Air Voids, %	3.9	--
VMA, %	14.2	>13.5
D/A Ratio	1.12	0.6 – 1.2

Production

The mix for this site was produced at Wiregrass Construction’s Calera plant. This Gencor counter-flow drum plant is shown in Figure 28, and the Gencor Green Machine used for water injection is shown in Figure 29.

Figure 28 Wiregrass Construction’s Gencor Plant in Calera, Alabama

Figure 29 Gencor Green Machine in Calera, Alabama

Production temperatures and rates were monitored and recorded throughout production of the mix. Table 33 shows production temperature information.

Table 33 Production Temperatures for I-65 Site

	Temperature, °F
Target	270
Average	275.5
Standard Deviation	13.2
Max	298
Min	249

Volumetric Mix Properties

Samples were collected and the sample truck was marked as described earlier. Table 34 shows the results from NCAT's gradation, asphalt content, and binder grading testing, and Table 35 shows the contractor's QC results.

Table 34 Gradation, Asphalt Content, and Binder Grades at Construction for I-65

	JMF	Extraction (AASHTO T164)	Ignition (AASHTO T308)	Specifications
Sieve Size	**% Passing**			
25.0 (1")	100	100.0	100.0	100 max
19.0 mm (3/4”)	99	99.2	99.4	90 – 100
12.5 mm (1/2”)	88	89.6	89.2	90 max
9.5 mm (3/8”)	75	77.5	78.4	--
4.75 mm (#4)	55	54.8	55.8	--
2.36 mm (#8)	41	42.3	43.2	23 – 49
1.18 mm (#16)	33	33.3	34.1	--
0.60 mm (#30)	23	24.0	25.1	--
0.30 mm (#50)	12	12.0	13.0	--
0.15 mm (#100)	7	6.7	7.7	--
0.075 mm (#200)	4.0	4.9	5.7	2 – 8
AC, %	4.4	3.7	4.1	4.4 minimum
True Binder Grade	NA	87.4 – 18.8	NA	NA
PG Grade	NA	82 – 16	NA	NA
ΔT_c	NA	-8.4	NA	NA

Table 35 Contractor’s Quality Control Results for I-65

Sieve Size, mm (in.)	Mix Design	Contractor’s QC	Specifications
	% Passing		
25.0 (1")	100	100	100 max
19.0 (3/4")	99	98	90 – 100
12.5 (1/2")	88	89	90 max
9.5 (3/8")	75	78	--
4.75 (#4)	55	53	--
2.36 (#8)	41	40	23 – 49
1.18 (#16)	33	32	--
0.6 (#30)	23	23	--
0.3 (#50)	12	11	--
0.15 (#100)	7	6	--
0.075 (#200)	4.0	4.3	2 – 8
AC, %	4.4	4.35	4.4 minimum
Air Voids, %	3.9	4.30	--
VMA, %	14.2	15.4	>13.5
D/A Ratio	1.12	0.99	0.6 – 1.2

Construction

This test section was placed on I-65 northbound in the outside lane, which was about 21 miles north of the plant. This yielded a haul time of approximately 25 to 30 minutes. The test mix was placed as the binder layer at a target thickness of two inches. Figure 30 shows the location of the test section and plant.

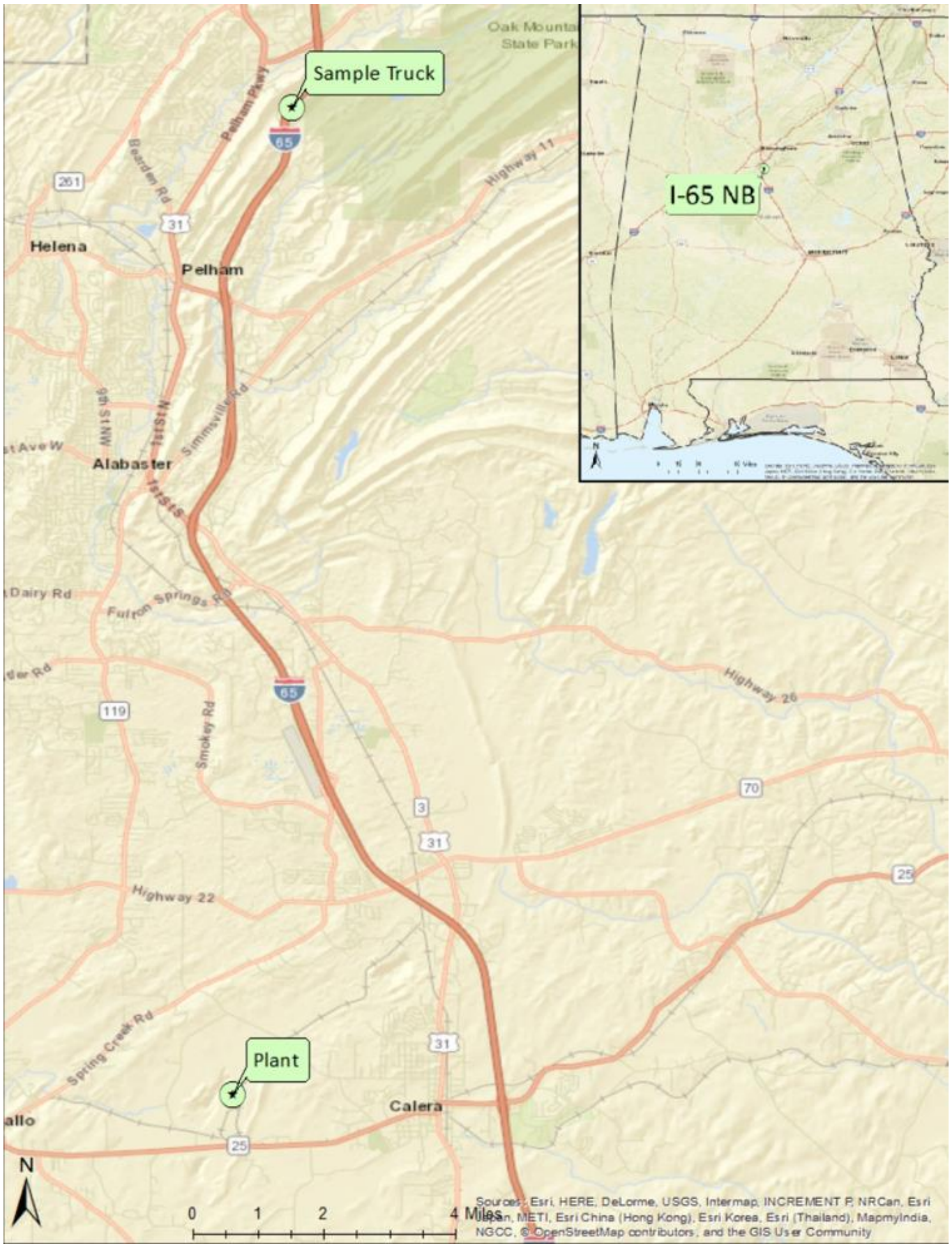

Figure 30 Location of Test Section on I-65

The asphalt mix was delivered to the paving site using 10-12 trucks. Once on-site, a RoadTec SB-2500D MTV was used to transfer the mixes to the Caterpillar AP1000E paver, which can be seen in Figure 31.

Figure 31 MTV Transferring Mix to Paver on I-65

The temperature of the mix behind the paver was measured periodically using a hand-held temperature gun. The temperatures measured behind the screed are shown in Table 36.

Table 36 Temperatures Behind the Screed on I-65

	Temperature, °F
Average	238.2
Standard Deviation	20.1
Max	264
Min	191

Two rollers were used for compaction of this layer. The breakdown roller was a Caterpillar CB54-XW operated in the vibratory mode, and the finishing roller was an Ingersoll Rand DD-138 operated in static mode. Figure 32 shows the breakdown roller used on I-65.

Figure 32 Breakdown Roller Used on I-65

Field Performance After Six Years in Service

A site visit was conducted on May 25, 2017 after the pavement had been in service for approximately six years. Six cores were taken from between the wheelpaths during the site visit. These cores were then brought back to NCAT for further testing. During coring operations, the area of pavement where the sample truck was placed was assessed. The pavement was deemed to be performing very well with no noticeable cracking or rutting in the area. Figure 33 shows an example of the test section after six years in service.

Figure 33 Example of Test Section on I-65

AL 137 in Covington County

In April of 2011, a field test section was placed on AL 137 in Covington County. This test section contained 35% RAP and was placed as the upper binder layer by Wiregrass Construction. The mix was a 19.0mm maximum aggregate size (MAS) Superpave mix design using a compactive effort of 60 gyrations. The mix was produced using the Terex foaming WMA system. Table 25 shows the aggregate percentages from mix design.

Table 37 Aggregate Percentages Used on AL 137

Aggregate Type	Mix Design (%)
#67 Limestone	15
#8910 Limestone	11
Shot Gravel	17
Coarse Sand	21
Baghouse Fines	1
RAP	35

A PG 67-22 supplied by Gulf Coast Asphalt Company in Mobile, Alabama was used as the base binder for this mix, with 0.5% Adhere LOF added as an antistrip agent. Water was injected at a rate of 3.0% by virgin binder. The design aggregate gradation, optimum asphalt content, design volumetrics, and specifications are shown in Table 38.

Table 38 Design Gradation, Asphalt Content, and Volumetrics for Mix Design

Sieve Size, mm (in.)	Mix Design	Specifications
	% Passing	
19.0 (3/4")	100	100 max
12.5 (1/2")	92	90 – 100
9.5 (3/8")	83	90 max
4.75 (#4)	61	--
2.36 (#8)	47	28 – 58
1.18 (#16)	40	--
0.6 (#30)	31	--
0.3 (#50)	15	--
0.15 (#100)	8	--
0.075 (#200)	4.4	2 – 10
AC, %	5.1	5.1 minimum
Air Voids, %	3.5	--
VMA, %	14.9	> 14.5
D/A Ratio	0.90	0.6 – 1.2

Production

The mix was produced at Wiregrass Construction’s Brantley plant and used the Terex water injection system as the WMA technology. This plant was a parallel-flow drum plant manufactured by CMI. Figure 34 shows a general overview of the asphalt plant used for this field demonstration, and Table 39 shows the Terex water injection system.

Figure 34 Wiregrass Construction’s CMI Plant in Brantley, Alabama

Figure 35 Terex Foaming System in Brantley, Alabama

Production temperatures were monitored and recorded throughout production of the mix. Table 39 shows production temperature information.

Table 39 Production Temperatures for AL 137 Site

	Temperature, °F
Target	275 - 285
Average	297.6
Standard Deviation	11.5
Max	320
Min	270

Volumetric Mix Properties

Samples were collected, and the sample truck was marked as described earlier. Table 40 shows the results from NCAT's gradation, asphalt content, and binder grading testing, and Table 41 shows the contractor's QC results.

Table 40 Gradation, Asphalt Content, and Binder Grades at Construction for AL 137

	JMF	Extraction (AASHTO T164)	Ignition (AASHTO T308)	Specifications
Sieve Size	**% Passing**			
19.0 mm (3/4”)	100	99.4	99.7	100 max
12.5 mm (1/2”)	92	94.6	94.2	90 – 100
9.5 mm (3/8”)	83	88.8	87.7	90 max
4.75 mm (#4)	61	70.7	68.6	--
2.36 mm (#8)	47	51.1	49.2	28 – 58
1.18 mm (#16)	40	42.5	41.1	--
0.60 mm (#30)	31	34.9	34.3	--
0.30 mm (#50)	15	19.2	19.9	--
0.15 mm (#100)	8	8.5	10.0	--
0.075 mm (#200)	4.4	4.3	6.2	2 – 10
AC, %	5.1	5.1	5.0	5.1 minimum
True Binder Grade	NA	79.5 – 15.6	NA	NA
PG Grade	NA	76 – 10	NA	NA
ΔT_c	NA	-9.1	NA	NA

Table 41 Contractor’s Quality Control Results for AL 137

Sieve Size, mm (in.)	Mix Design	Contractor’s QC	Specifications
	% Passing		
19.0 (3/4")	100	100	100 max
12.5 (1/2")	92	93	90 – 100
9.5 (3/8")	83	84	90 max
4.75 (#4)	61	65	--
2.36 (#8)	47	46	28 – 58
1.18 (#16)	40	38	--
0.6 (#30)	31	31	--
0.3 (#50)	15	17	--
0.15 (#100)	8	8	--
0.075 (#200)	4.4	3.9	2 – 10
AC, %	5.1	4.90	5.1 minimum
Air Voids, %	3.5	3.84	--
VMA, %	14.9	15.0	> 14.5
D/A Ratio	0.90	0.80	0.6 – 1.2

Construction

The test section was placed as the binder layer in the northbound lane of AL 137 in Covington county. The target thickness was 1.75 inches. This yielded a haul distance of approximately 40 miles, or 50 to 60 minutes. Figure 36 shows the location of the test section and plant.

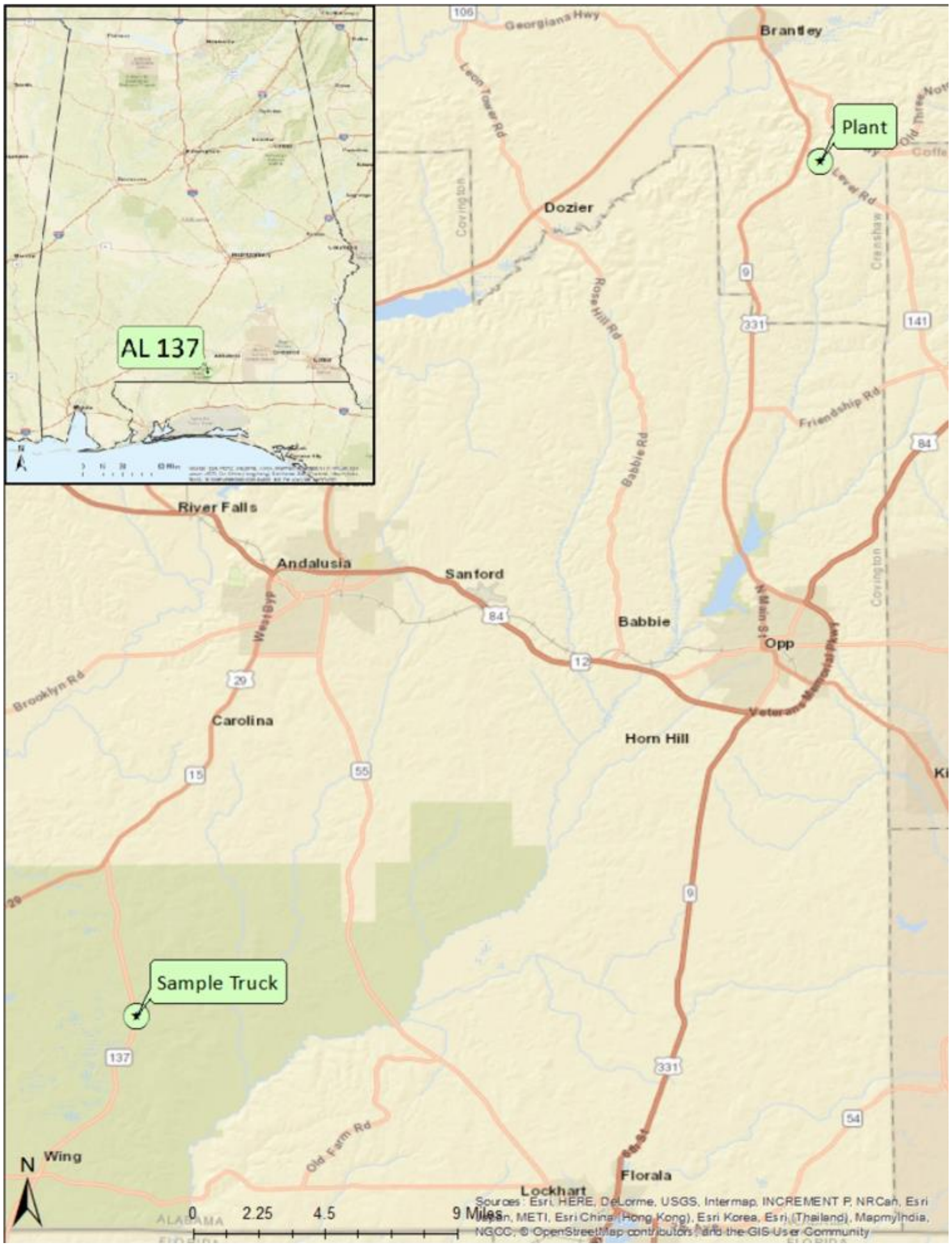

Figure 36 Location of Test Section on AL 137 Test Site

The asphalt mixes were delivered to the paving site using a cycle of 25 trucks. Once on site, a RoadTec SB-2500C MTV was used to transfer the mixes to the Vogele paver, which can be seen in Figure 37.

Figure 37 MTV Transferring Mix to Paver on AL 137

The temperature of the mix behind the paver was measured periodically using a hand-held temperature gun. The temperatures measured behind the screed are shown in Table 42.

Table 42 Temperatures behind the Screed on AL 137

	Temperature, °F
Average	263.4
Standard Deviation	10.6
Max	286
Min	239

Two rollers were used for compaction for this site. The breakdown roller was a Volvo DD118-HF operated in the vibratory mode, while the finishing roller was an Ingersoll Rand DD11-HF operated in static mode. Figure 38 shows both rollers operating on AL 137.

Figure 38 Breakdown and Finishing Rollers Operating on AL 137

Field Performance After Six Years in Service

A site visit was conducted on May 9, 2017 after the pavement had been in service for approximately six years. Six cores were taken from between the wheelpaths during the site visit. These cores were then brought back to NCAT for further testing. During coring operations, the area of pavement where the sample truck was placed was assessed. The pavement was observed to be performing very well with no noticeable cracking or rutting in the area. Figure 39 shows an example of the test section after six years in service.

Figure 39 Example of Test Section on AL 137

US 29 in Pike County

A test section was placed on US 29 in Pike County west of Troy in July 2011. This test section contained 32% RAP and 3% post-consumer RAS. The mix used was a 19.0mm maximum aggregate size (MAS) Superpave mix design using a compactive effort of 60 gyrations and was placed as the upper binder course. The mix was produced using the Terex foaming WMA system. Table 43 shows the aggregate percentages from mix design.

Table 43 Aggregate Percentages Used on US 29

Aggregate Type	Mix Design (%)
#67 Limestone	10
#8910 Limestone	9
½" Crushed Gravel	23
Shot Gravel	11
Coarse Sand	11
Baghouse Fines	1
Post-Consumer RAS	3
RAP	32

A PG 67-22 supplied by Gulf Coast Asphalt Company in Mobile, Alabama was used as the base binder for this mix. No antistrip was used, and water was injected at a rate of 2.0% by virgin binder. The design aggregate gradation, optimum asphalt content, design volumetrics, and specifications are shown in Table 44.

Table 44 Design Gradation, Asphalt Content, and Volumetrics for Mix Design

Sieve Size, mm (in.)	Mix Design	Specifications
	% Passing	
19.0 (3/4")	100	100 max
12.5 (1/2")	94	90 – 100
9.5 (3/8")	87	90 max
4.75 (#4)	65	--
2.36 (#8)	44	28 – 58
1.18 (#16)	31	--
0.6 (#30)	21	--
0.3 (#50)	12	--
0.15 (#100)	8	--
0.075 (#200)	6.6	2 – 10
AC, %	5.1	5.1 minimum
Air Voids, %	4.0	--
VMA, %	15.3	> 14.5
D/A Ratio	1.11	0.6 – 1.2

Production

The mix was produced at Wiregrass Construction’s Brantley plant and used the Terex water injection system as the WMA technology. This plant was a parallel-flow drum plant manufactured by CMI. This was the same plant used for the AL 137 test site as previously discussed and shown in Figure 34. Production temperatures were monitored and recorded throughout production of the mix. Table 45 shows production temperature information.

Table 45 Production Temperatures for US 29 Site

	Temperature, °F
Target	280-285
Average	285.3
Standard Deviation	9.8
Max	320
Min	278

Volumetric Mix Properties

Samples were collected, and the sample truck was marked as described earlier. Table 46 shows the results from NCAT’s gradation, asphalt content, and binder grading testing, and Table 47 shows the contractor’s QC results.

Table 46 Gradation, Asphalt Content, and Binder Grades at Construction for US 29

	JMF	Extraction (AASHTO T164)	Ignition (AASHTO T308)	Specifications
Sieve Size	**% Passing**			
19.0 mm (3/4")	100	99.2	98.5	100 max
12.5 mm (1/2")	94	92.6	95.7	90 – 100
9.5 mm (3/8")	87	84.7	89.2	90 max
4.75 mm (#4)	65	60.4	66.9	--
2.36 mm (#8)	44	41.8	46.9	28 – 58
1.18 mm (#16)	31	32.9	34.7	--
0.60 mm (#30)	21	25.5	28.4	--
0.30 mm (#50)	12	15.2	17.1	--
0.15 mm (#100)	8	8.7	10.4	--
0.075 mm (#200)	6.6	5.7	7.2	2 – 10
AC, %	5.1	4.7	5.3	5.1 minimum
True Binder Grade	NA	105.4 – 7.5	NA	NA
PG Grade	NA	100 – 4	NA	NA
ΔT_c	NA	-14.3	NA	NA

Table 47 Contractor's Quality Control Results for US 29

Sieve Size, mm (in.)	Mix Design	Contractor's QC	Specifications
	% Passing		
19.0 (3/4")	100	100	100 max
12.5 (1/2")	94	96	90 – 100
9.5 (3/8")	87	89	90 max
4.75 (#4)	65	66	--
2.36 (#8)	44	45	28 – 58
1.18 (#16)	31	35	--
0.6 (#30)	21	26	--
0.3 (#50)	12	16	--
0.15 (#100)	8	10	--
0.075 (#200)	6.6	5.8	2 – 10
AC, %	5.1	5.15	5.1 minimum
Air Voids, %	4.0	3.94	--
VMA, %	15.3	14.8	> 14.5
D/A Ratio	1.11	1.23	0.6 – 1.2

Construction

The test section was placed as the binder layer in the westbound lane of US 29 in Pike county just west of Troy. This yielded a haul distance of approximately 30 miles, or 35 to 45 minutes. Figure 40 shows the location of the test section and plant.

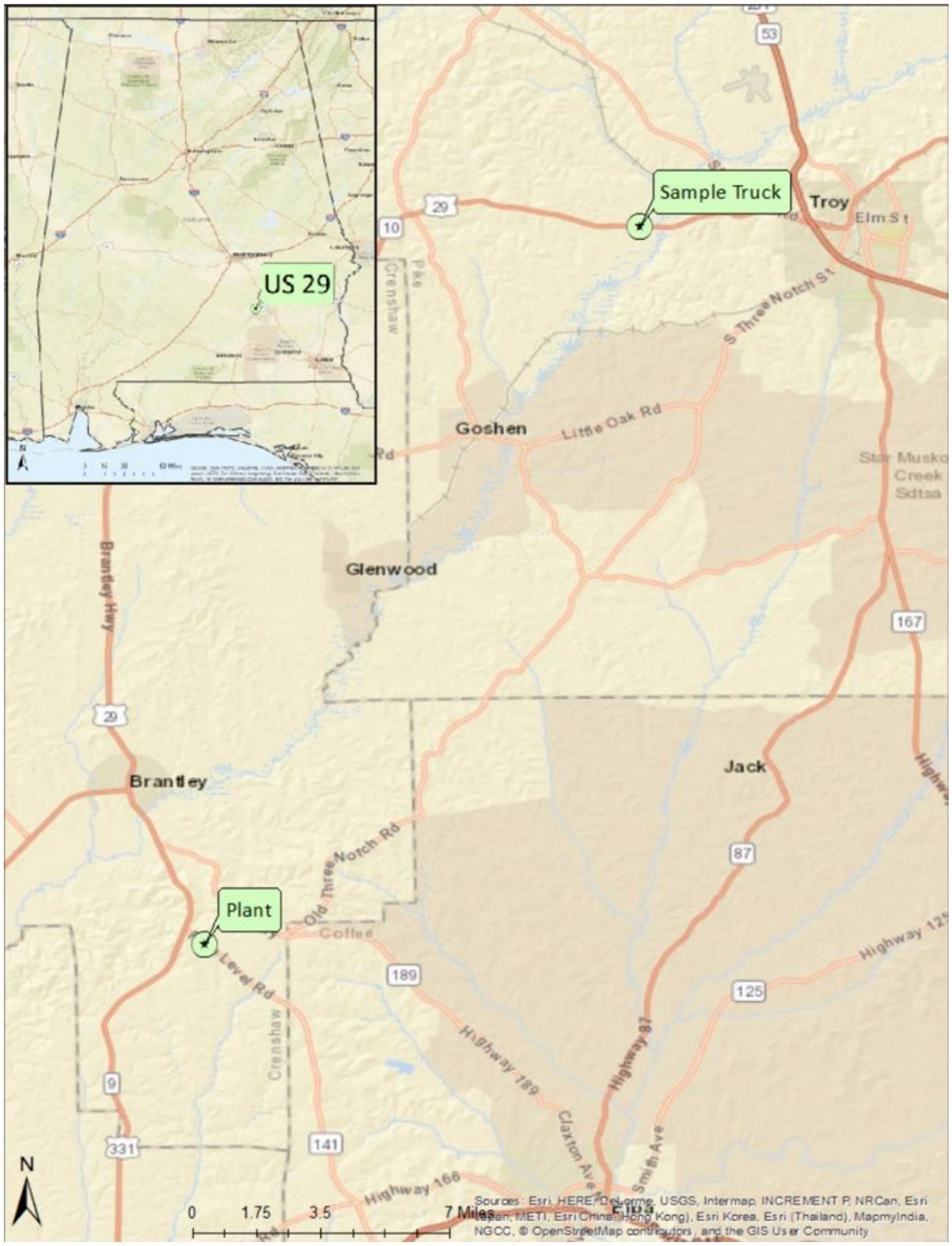

Figure 40 Location of Test Section on US 29

The asphalt mixes were delivered to the paving site using a cycle of 25 trucks. Once on site, an Ingersoll Rand MC-330 mix transfer vehicle was used to transfer the mixes to the Caterpillar AP-1000D paver, which can be seen in Figure 41.

Figure 41 MTV Transferring Mix to Paver on US 29

The temperature of the mix behind the paver was measured periodically using a hand-held temperature gun. The temperatures measured behind the screed are shown in Table 48.

Table 48 Temperatures Behind the Screed on US 29

	Temperature, °F
Average	253.3
Standard Deviation	9.9
Max	275
Min	231

Two rollers were used for compaction. The breakdown roller was a Volvo DD138-HF operated in the vibratory mode, while the finishing roller was a Volvo DD18-HF operated in static mode. Figure 42 shows the breakdown roller operating on US 29.

Figure 42 Breakdown Roller Operating on US 29

Field Performance After Six Years in Service

A site visit was conducted on May 10, 2017 after the pavement had been in service approximately six years. Six cores were taken from between the wheelpaths during the site visit. These cores were then brought back to NCAT for further testing. During coring operations, the area of pavement where the sample truck was placed was assessed. Both wheelpaths have begun to exhibit fatigue cracking as seen in Figure 43.

Figure 43 Fatigue Cracking in Wheelpaths Observed on US 29

AL 35 in Cherokee County

In September 2011, a test section was placed on AL 35 in Cherokee county. This test section was placed as the upper binder course at a target thickness of two inches by Good Hope Contracting. The mix used was a 25.0 mm maximum aggregate size (MAS) Superpave mix design using a compactive effort of 60 gyrations. The mix was produced using the WMA additive Evotherm 3G. Table 49 shows the aggregate percentages from mix design.

Table 49 Aggregate Percentages Used on AL 35

Aggregate Type	Mix Design (%)
#67 Limestone	20
#78 Limestone	20
Limestone Screenings	10
Sand	15
RAP	35

A PG 67-22 supplied by Ergon was used as the base binder for this mix. The Evotherm 3G, which acted as both a WMA additive and antistrip, was terminally blended with the virgin binder at a rate of 0.7% by mass of virgin binder. The design aggregate gradation, optimum asphalt content, design volumetrics, and specifications are shown in Table 50.

Table 50 Design Gradation, Asphalt Content, and Volumetrics for Mix Design

Sieve Size, mm (in.)	Mix Design	Specifications
	% Passing	
25.0 (1”)	100	100 max
19.0 (3/4")	98	90 – 100
12.5 (1/2")	90	90 max
9.5 (3/8")	81	--
4.75 (#4)	55	--
2.36 (#8)	37	23 – 49
1.18 (#16)	28	--
0.6 (#30)	20	--
0.3 (#50)	13	--
0.15 (#100)	7	--
0.075 (#200)	4.0	2 – 8
AC, %	4.4	4.4 minimum
Air Voids, %	4.1	--
VMA, %	14.2	>13.5
D/A Ratio	0.92	0.6 – 1.2

Production

The test mix was produced in Collinsville, Alabama at Good Hope Contracting’s portable plant manufactured by Asphalt Drum Mixers Inc. (ADM). This portable plant was a parallel flow drum plant. Figure 44 shows a general overview of the asphalt plant used for this field demonstration.

Figure 44 Good Hope Contracting’s ADM Plant in Collinsville, Alabama

Production temperatures were monitored and recorded throughout production of the mix. Table 51 shows production temperature information.

Table 51 Production Temperatures for AL 35 Site

	Temperature, °F
Target	260
Average	257.4
Standard Deviation	5.9
Max	270
Min	249

Volumetric Mix Properties

Samples were collected, and the sample truck was marked as described earlier. Table 52 shows the results from NCAT's gradation, asphalt content, and binder grading testing, and Table 53 shows the contractor's QC results.

Table 52 Gradation, Asphalt Content, and Binder Grades at Construction for AL 35

	JMF	Extraction (AASHTO T164)	Ignition (AASHTO T308)	Specifications
Sieve Size	**% Passing**			
25.0 (1")	100	100.0	100.0	100 max
19.0 mm (3/4")	98	98.4	98.9	90 – 100
12.5 mm (1/2")	90	90.2	92.4	90 max
9.5 mm (3/8")	81	78.1	80.7	--
4.75 mm (#4)	55	52.0	55.5	--
2.36 mm (#8)	37	38.7	42.4	23 – 49
1.18 mm (#16)	28	28.8	31.4	--
0.60 mm (#30)	20	21.5	24.1	--
0.30 mm (#50)	13	13.5	15.7	--
0.15 mm (#100)	7	8.2	10.5	--
0.075 mm (#200)	4.0	6.0	8.1	2 – 8
AC, %	4.4	4.5	4.8	4.4 min
True Binder Grade	NA	90.9 – 16.0	NA	NA
PG Grade	NA	88 – 16	NA	NA
ΔT_c	NA	-8.4	NA	NA

Table 53 Contractor's Quality Control Results for AL 35

Sieve Size, mm (in.)	Mix Design	Contractor's QC	Specifications
	% Passing		
25.0 (1")	100	100	100 max
19.0 (3/4")	98	99	90 – 100
12.5 (1/2")	90	95	90 max
9.5 (3/8")	81	80	--
4.75 (#4)	55	53	--
2.36 (#8)	37	38	23 – 49
1.18 (#16)	28	27	--
0.6 (#30)	20	20	--
0.3 (#50)	13	12	--
0.15 (#100)	7	7	--
0.075 (#200)	4.0	5.1	2 – 8
AC, %	4.4	4.38	4.4 minimum
Air Voids, %	4.1	4.14	--
VMA, %	14.2	14.2	>13.5
D/A Ratio	0.92	1.19	0.6 – 1.2

Construction

The test section was placed on AL 35 in Cherokee County, approximately 25 miles from the plant. This produced a haul time of approximately 30 to 40 minutes. The mix was placed as the binder course at a target thickness of two inches. Figure 45 shows the location of the test section in relation to the plant.

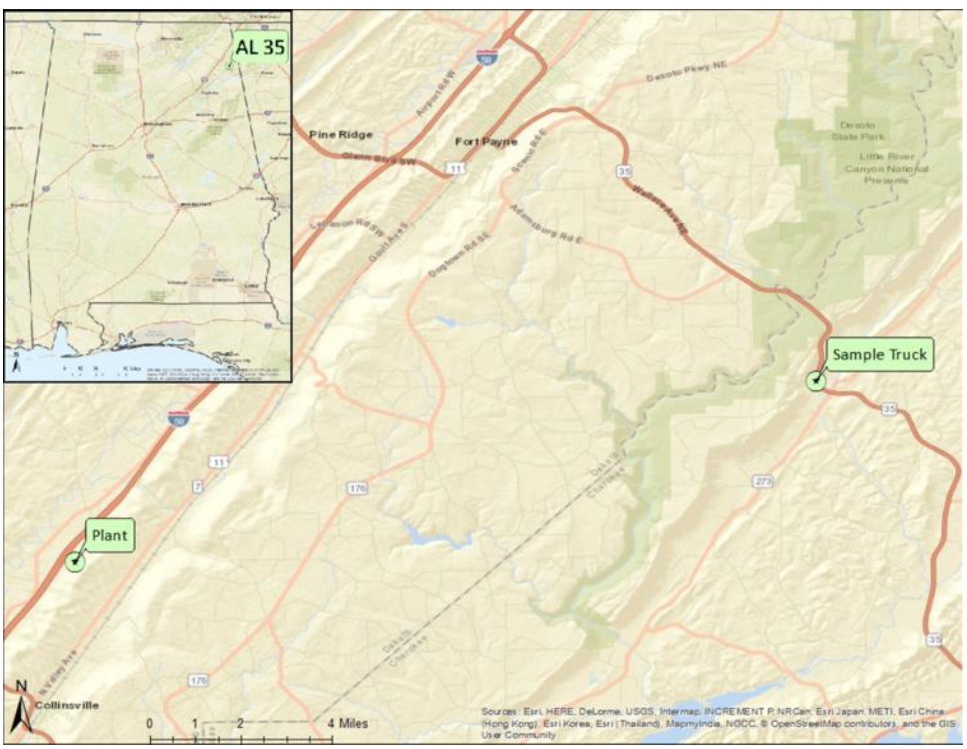

Figure 45 Location of Test Section on AL 35

The asphalt mixes were delivered to the paving site using an 18-truck cycle. Once on site, the mixes were transferred to the paver using a RoadTec SB-2500B MTV. The paver was a RoadTec RP190 and can be seen in Figure 46.

Figure 46 MTV Transferring Mix to Paver on AL 35

The temperature of the mix behind the paver was measured periodically using a hand-held temperature gun. The temperatures measured behind the screed are shown in Table 54.

Table 54 Temperatures Behind the Screed on AL 35

	Temperature, °F
Average	233.0
Standard Deviation	7.1
Max	246
Min	219

Two rollers were used for compaction. The breakdown roller was a Caterpillar CB-564D operated in the vibratory mode. The finishing roller was also a Caterpillar CB-564D operated in vibratory mode. Figure 47 shows both roller operating on AL 35.

Figure 47 Roller Compacting on AL 35

Field Performance After Six Years in Service

A site visit was conducted on August 11, 2017 after the pavement had been in service for approximately six years. The area where the sample truck was located was in a dangerous turn coming down the mountain, so the location of the cores had to be moved to a safer location in the northbound lane. Six cores were taken from between the wheelpaths during the site visit. These cores were then brought back to NCAT for further testing. During coring operations, the area of pavement near the cores was assessed. The surface pavement had begun to ravel in the wheelpaths, but no major cracking or rutting was evident. Figure 48 shows an example of the pavement and Figure 49 shows a close-up example of the raveling observed after six years.

Figure 48 Example of Pavement on AL-35

Figure 49 Close-Up View of Raveling on AL-35

US 80 in Lowndes County

A field test section was placed on US 80 in Lowndes County in February 2012. This test section contained 40% RAP and was placed as the upper binder layer by Wiregrass Construction Inc. The mix used was a 19.0 mm maximum aggregate size (MAS) Superpave mix design using a compactive effort of 60 gyrations. The mix was produced using the WMA additive Evotherm 3G. Table 55 shows the aggregate percentages from mix design.

Table 55 Aggregate Percentages Used on US 80

Aggregate Type	Mix Design (%)
#78 Limestone	5
Limestone Screenings	5
Crushed Gravel	21
Shot Gravel	17
Sand	11
Baghouse Fines	1
RAP	40

A PG 67-22 from Ergon was used as the base binder for this mix. The Evotherm 3G acted as both a WMA additive and antistrip. The Evotherm was terminally blended with the virgin binder at a rate of 0.7% by mass of virgin binder. The design aggregate gradation, optimum asphalt content, design volumetrics, and specifications are shown in Table 56.

Table 56 Design Gradation, Asphalt Content, and Volumetrics for Mix Design

Sieve Size, mm (in.)	Mix Design	Specifications
	% Passing	
19.0 (3/4")	100	100 max
12.5 (1/2")	97	90 – 100
9.5 (3/8")	89	90 max
4.75 (#4)	68	--
2.36 (#8)	45	28 – 58
1.18 (#16)	32	--
0.6 (#30)	23	--
0.3 (#50)	12	--
0.15 (#100)	8	--
0.075 (#200)	5.4	2 – 10
AC, %	5.1	5.1 minimum
Air Voids, %	3.4	--
VMA, %	15.0	> 14.5
D/A Ratio	1.08	0.6 – 1.2

Production

The mixture was produced at Wiregrass Construction’s Montgomery plant. This plant is a counter-flow drum plant manufactured by CMI. Figure 50 shows a general overview of the asphalt plant used Wiregrass Construction to produce this mix.

Figure 50 Wiregrass Construction's CMI Plant in Montgomery, Alabama

Production temperatures were monitored and recorded throughout production of the mix. Table 57 shows production temperature information.

Table 57 Production Temperatures for US 80 Site

	Temperature, °F
Target	280
Average	281.6
Standard Deviation	24.5
Max	345
Min	237

Volumetric Mix Properties

Samples were collected, and the sample truck was marked as described earlier. Table 58 shows the results from NCAT's gradation, asphalt content, and binder grading testing, and Table 59 shows the contractor's QC results.

Table 58 Gradation, Asphalt Content, and Binder Grades at Construction for US 80

	JMF	Extraction (AASHTO T164)	Ignition (AASHTO T308)	Specifications
Sieve Size	**% Passing**			
19.0 mm (3/4”)	100	99.4	100.0	100 max
12.5 mm (1/2”)	97	95.9	95.6	90 – 100
9.5 mm (3/8”)	89	86.5	86.0	90 max
4.75 mm (#4)	68	59.1	58.1	--
2.36 mm (#8)	45	39.6	39.0	28 – 58
1.18 mm (#16)	32	28.6	28.0	--
0.60 mm (#30)	23	20.0	20.1	--
0.30 mm (#50)	12	10.7	11.3	--
0.15 mm (#100)	8	6.8	7.7	--
0.075 mm (#200)	5.4	4.8	5.8	2 – 10
AC, %	5.1	4.6	4.7	5.1 minimum
True Binder Grade	NA	92.3 – 16.	NA	NA
PG Grade	NA	88 – 16	NA	NA
ΔT_c	NA	-8.2	NA	NA

Table 59 Contractor’s Quality Control Results for US 80

Sieve Size, mm (in.)	Mix Design	Contractor’s QC	Specifications
	% Passing		
19.0 (3/4")	100	100	100 max
12.5 (1/2")	97	97	90 – 100
9.5 (3/8")	89	90	90 max
4.75 (#4)	68	64	--
2.36 (#8)	45	41	28 – 58
1.18 (#16)	32	28	--
0.6 (#30)	23	20	--
0.3 (#50)	12	11	--
0.15 (#100)	8	7	--
0.075 (#200)	5.4	4.5	2 – 10
AC, %	5.1	5.00	5.1 minimum
Air Voids, %	3.4	4.17	--
VMA, %	15.0	16.8	> 14.5
D/A Ratio	1.08	0.90	0.6 – 1.2

Construction

The high RAP test section was placed on US 80 westbound in the passing lane. This yielded a haul distance of approximately 19 miles, or 20 to 25 minutes. The mix was placed as the upper binder layer at a target thickness of 1.75 inches. Figure 51 shows the location of the test section in relation to the plant.

Figure 51 Location of Test Section on US 80

A cycle of 26 to 27 trucks was used to deliver the mix to the paving site. Once on site, a Weiler E1250A mix transfer vehicle was used to transfer the mixes to the Caterpillar AP1000D paver, which can be seen in Figure 52.

Figure 52 MTV Transferring Mix to Paver on US 80

The temperature of the mix behind the paver was measured periodically using a hand-held temperature gun. The temperatures measured behind the screed are shown in Table 60.

Table 60 Temperatures Behind the Screed on US 80

	Temperature, °F
Average	238.5
Standard Deviation	14.0
Max	270
Min	201

Two rollers were used for compaction. Both rollers were Volvo DD138-HF models. The breakdown roller operated in vibratory mode, and the finishing roller operated in static mode. Figure 53 shows the breakdown roller operating on US 80.

Figure 53 Breakdown Roller Operating on US 80

Field Performance After Five Years in Service

A site visit was conducted on June 8, 2017 after the pavement had been in service for approximately five years. Six cores were taken from between the wheelpaths during the site visit. These cores were then brought back to NCAT for further testing. During coring operations, the area of pavement where the sample truck was placed was assessed. Some low-severity cracks were observed in this location as shown in Figure 54. The surface had also begun to show signs of raveling as shown in Figure 55.

Figure 54 Low-Severity Cracking Observed on US 80

Figure 55 Example of Raveling Observed on US 80