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PHASE VI (2015-2017) NCAT TEST TRACK FINDINGS



By
Randy West
David Timm
Buzz Powell
Michael Heitzman
Nam Tran
Carolina Rodezno
Don Watson
Fabricio Leiva
Adriana Vargas

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National Center for Asphalt Technology
NCAT
at AUBURN UNIVERSITY

277 Technology Parkway ■ Auburn, AL 36830

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Randy West, Ph.D., P.E.
David Timm, Ph.D., P.E.
Buzz Powell, Ph.D., P.E.
Michael Heitzman, Ph.D., P.E.
Nam Tran, Ph.D., P.E., LEED GA, MBA
Carolina Rodezno, Ph.D.
Don Watson, P.E.
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Adriana Vargas, Ph.D.

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CHAPTER 1 INTRODUCTION

1.1 NCAT Test Track Background

The National Center for Asphalt Technology (NCAT) Test Track is a pavement proving ground that was originally constructed from 1998-2000. This 1.7-mile oval track is a unique accelerated pavement testing facility that utilizes full-scale pavement construction of test sections and highway-speed, heavy trafficking to provide analysis of asphalt pavement responses and performance in just a few years. This provides the opportunity to realistically evaluate cutting-edge technologies and assist sponsors in implementation of materials and design methods that advance safe, durable, and sustainable asphalt pavements.

Figure 1 Aerial Photograph of the NCAT Test Track

Sixteen highway agencies and private sector partners funded experiments in the sixth cycle of the Test Track. Experiments included single test sections and groups of test sections. Since the results of the experiments are typically evident by the performance of the test sections, the findings are generally easy to interpret. This gives highway agency sponsors confidence to make decisions regarding their specifications for materials and construction practices as well as pavement design methods that can improve the performance of their roadways. Industry sponsors can use the results to publicly and convincingly demonstrate the value of their product or technology to the pavement engineering community.

There are 46 main test sections on the track. Each section is nominally 200 ft in length. In some cases, test sections are divided into subsections. Twenty-six of the main test sections are located on the two straight segments of the track, and ten sections are located in each of the two curves.

Experiments are sponsored for three-year cycles, and each cycle consists of three major parts. The first part of each cycle begins with building or replacing test sections, which normally takes about six months, including material acquisition and mixture and pavement designs. The second part of each cycle involves trafficking of the test sections, collection of field performance data and pavement response data, and laboratory testing of the plant-produced materials sampled during construction. Trafficking is accomplished with five heavily loaded tractor-trailer rigs providing approximately 10 million 18,000-pound equivalent single-axle loads (ESALs) using legally loaded axles over a two-year period. The final part of the cycle involves forensic analyses of damaged sections to determine factors that may have contributed to the observed distresses.

1.2 Research Cycles

The first Test Track cycle began in 2000. Experiments in the inaugural cycle focused only on surface mixtures. Test sections were built with stone matrix asphalt (SMA), Superpave, and Hveem mixes using a wide variety of aggregate types, gradations, and asphalt binders. The pavement structure under the experimental surface mixes was built with approximately 20 in. of asphalt pavement over a granular base and a stiff subgrade to isolate damage to only the surface layers.

The second cycle began in 2003 and included the continued evaluation of 24 of the original test sections. New experiments included 14 test sections with new surface layers and 8 sections that were completely rebuilt from the subgrade up. These were the first "structural experiments" designed and built to analyze the entire pavement structure, not just the surface layers. Construction of the structural experimental sections began by removing the original thick pavement structure down to the subgrade material, then rebuilding the subgrade, aggregate base, and asphalt layers to result in test sections with asphalt pavement thicknesses of 5, 7, and 9 inches. Strain gauges, pressure plates, and temperature probes were built into the structural sections to monitor how the different thicknesses and mix designs responded to traffic and temperature changes.

The third cycle of the track began in 2006. Twenty-two new test sections were built, including fifteen new surface mix experiments, four new structural experiment sections, and three reconstructed structural sections. Eight of the original sections built in 2000 remained in place and accumulated 30 million ESALs by the end of the third cycle, and sixteen sections from the second cycle remained in place and carried a total of 20 million ESALs by the end of the third cycle.

Twenty-five new test sections (twelve mix performance and thirteen structural) were built for the track's fourth research cycle in 2009. Three of the original surface mix performance sections from the first cycle remained in place and had accumulated 40 million ESALs by the end of the fourth cycle. Nine sections from the 2003 track (seven mix performance and two structural) remained in place and had accumulated 30 million ESALs. Nine sections from the 2006 track (eight mix performance and one structural) remained in place and had accumulated 20 million ESALs. In summary, the fourth cycle included sixteen structural sections, thirty surface mix performance sections, and twenty-one test sections remaining from previous research cycles (three from 2000, nine from 2003, and nine from 2006).

The fifth cycle of the track began in 2012 and included 21 new experimental test sections. Fourteen test sections were left in place for continued evaluation from the 2009 cycle, six were left in place from the 2006 cycle, three sections remained from 2003, and two sections remained from the original construction. The fifth cycle included a more complex range of experiments than any of the previous cycles. Several experiments focused on the use of recycled materials in pavements, on porous friction course (PFC) mixes, and on pavement preservation. These test sections were built on the Test Track and a local county road, Lee Road 159.

In 2015, the sixth cycle began a new chapter in full-scale pavement research through a partnership with the Minnesota Department of Transportation's MnROAD facility. The NCAT-MnROAD partnership features a collaboration to address two national research needs. The first research need is to validate asphalt mixture cracking tests that are suitable for routine use in mix design and quality assurance testing. The experiment for the validation of cracking tests is called the Cracking Group Experiment and includes seven new test sections on the NCAT Test Track and eight rebuilt test sections on MnROAD's main-line test road. The second national research need addressed by the partnership is to objectively quantify the life-extending benefits of pavement preservation treatments. This research significantly expanded the 2012 pavement preservation experiment on Lee Road 159 by installing 34 additional pavement preservation treatment sections on U.S. Highway 280 near the Test Track. To complement the pavement preservation treatments on Lee Road 159 and US 280, the same treatments were also applied to low traffic volume and high traffic volume routes in Minnesota. The sixth cycle of the Test Track also includes 11 other new surface layer sections, 2 new structural sections, and 17 sections left in place from previous cycles for continued evaluation.

1.3 Sixth Cycle Sponsors

Sponsors of the Cracking Group Experiment include the Federal Highway Administration (FHWA), the Alabama Department of Environmental Management (ADEM), and the Departments of Transportation for Alabama, Florida, Illinois, Michigan, Minnesota, New York, North Carolina, Oklahoma, and Wisconsin.

Sponsors of the expanded Preservation Group Experiment include the Foundation for Pavement Preservation (FP^2, Inc.) and the Departments of Transportation for Alabama, Colorado, Georgia, Illinois, Kentucky, Maryland, Michigan, Minnesota, Mississippi, Missouri, New York, Oklahoma, South Carolina, Tennessee, and Wisconsin.

Sponsors of individual experiments for the 2015 Test Track are listed below in alphabetical order.

Alabama Department of Transportation (ALDOT)

ALDOT sponsored the continued evaluation of three experimental porous friction course (PFC) test sections built in 2012. One section is a 9.5 mm NMAS PFC and the other two are 12.5 mm NMAS PFC mixtures. The 9.5 mm NMAS mix contains 0.3% cellulose fiber to prevent drain-down. One of the 12.5 mm NMAS PFC mixes contains 0.05% synthetic fiber and the other contains 12% ground tire rubber.

Collaborative Aggregates

Collaborative Aggregates became a new Test Track sponsor in 2015. They funded the evaluation of a surface mix containing their bio-based Delta S rejuvenator and 35% RAP. The test section is compared to the 20% RAP control test section in the Cracking Group Experiment.

Federal Highway Administration (FHWA)

The FHWA provided funding to evaluate high friction surface treatments.

Florida Department of Transportation (FDOT)

FDOT sponsored two new sections to evaluate the cracking performance of surface mixes containing 20 to 30% RAP with a PG 76-22 binder and a PG 58-28 binder.

Georgia Department of Transportation (GDOT)

GDOT sponsored the continued evaluation of the experiment to compare the effectiveness of two alternative treatments for mitigating reflective cracking. Two test sections built in 2012 began by making deep saw cuts in the existing pavements to simulate a cracked pavement. One section used a stress absorbing interlayer consisting of a double chip seal surface treatment application with a sand seal surface (often referred to as a triple chip seal). The second section used what is referred to as a plant produced open-graded interlayer (OGI), which is similar to a PFC mix without fiber stabilizer and has a lower asphalt content.

Kentucky Transportation Cabinet (KYTC)

KYTC sponsored an experiment to evaluate the performance of longitudinal joints and durability for two mix designs. One was a coarse-graded Superpave mix typical of surface mixes in Kentucky, and the other used a fine-graded, lower-gyration mix design. The surface layers were constructed in both the inside and outside lanes of the Test Track to specifically evaluate longitudinal joint performance of the mix variations.

Mississippi Department of Transportation (MDOT)

MDOT sponsored the continuation of traffic on their section containing 45% RAP and a new low-cost thin overlay test section.

Oklahoma Department of Transportation (ODOT)

ODOT sponsored a new PFC test section to assess friction and the effect of tack coat rate on bond strength.

Tennessee Department of Transportation (TDOT)

TDOT sponsored a new surface mix performance experiment with a 4.75 mm NMAS thinlay.

Virginia Department of Transportation (VDOT)

VDOT sponsored the continued evaluation of three structural performance sections built in 2012 with foamed asphalt stabilized RAP using a cold central-plant recycling (CCPR) process. The primary objective of the structural sections was to determine the structural contribution of the stabilized RAP base layer. One of the sections had a cement stabilized base.

1.4 Sixth Cycle Donations

Numerous companies provided generous donations of equipment, materials, and human resources to help build test sections. This support helps to minimize costs and ensures that the highest quality research is achieved. As before, Astec Industries provided personnel and equipment to assist in production of experimental mixes and construction of test sections. Roadtec, East Alabama Paving and Trucking, Sakai America, and Wirtgen Group provided construction equipment. Materials were donated by Ashapura Group, Blacklidge Emulsions, Ergon Asphalt and Emulsions, Hi-Tech Asphalt Solutions, Ingevity, Martin Marietta, Scotty's Contracting and Stone, Vulcan Materials, and Wiregrass Construction. Other significant donations were made by Caterpillar, Colas Solutions, Ozark Materials and Striping, Pathway Services, and Troxler Electronic Laboratories.

1.5 Construction

New test sections were milled to the appropriate depth by Roadtec Inc., who generously provided milling machines and highly skilled operators at no cost. The Test Track manager coordinated milling locations and depths, while NCAT personnel operated dump trucks to collect and haul millings.

The instrumentation system developed through previous cycles of the NCAT Test Track was again used to measure pavement responses in each of the seven Cracking Group Experiment test sections. The instrumentation plan and analysis routines key to gathering data for mechanistic pavement analyses are fully described in NCAT Report 09-01.

East Alabama Paving Company was awarded contracts to produce the asphalt mixtures and construct the test sections through a competitive bidding process through Auburn University. Due to space limitations on the contractor's yard, some materials were temporarily stored on paved surfaces on Test Track property before being moved to the plant site for mix production.

A special production sequence was used for each mix. The plant's cold feed bins were calibrated for each unique stockpile. Production began with running the aggregate through the dryer and mixer without the addition of asphalt binder to achieve a consistent gradation and temperature. This uncoated material was discharged and wasted. Liquid asphalt was then turned on and the mix was discharged at the slat conveyor bypass chute until the aggregates were well coated. The bypass chute was then closed and the mixture was conveyed into the storage silo until the plant controls indicated that approximately one truckload had accumulated. This mix was loaded into a truck and then dumped into a stockpile for future recycling. At this point, the plant was assumed to have reached steady state conditions and that subsequent mix run into the silo would be uniform in terms of aggregate gradation, asphalt content, and temperature. After the desired quantity of mix had been produced, the aggregate and asphalt flows were stopped, the remaining materials in the dryer and mixer were discharged at the bypass chute, and the plant was shut down. The cold feed bins were unloaded and the plant was readied for the next test mix.

Prior to placement of mixes on each test section, at least one trial mix was produced to evaluate the quality control requirements of the sponsor. The trial mixes were hauled to the track and sampled by NCAT personnel for laboratory testing and evaluation. Test results of the trial mixes were presented to each sponsor to determine appropriate adjustments in plant settings for the subsequent production of mix for placement.

Mix produced for placement on the test sections followed the same production sequence described above. Mix production continued until a sufficient quantity of material was available for placement. The contractor was responsible for hauling mixes to the track, and the paving equipment and crew were staged at the track.

Figure 2 Paving a Test Section for the 2015-2017 Research Cycle

Before placing mixtures on the test sections, the contractor tacked the underlying asphalt pavement with a PG 67-22 binder, NTSS-1HM emulsion, or other tack material depending on the sponsor's preference. The target application rates were generally between 0.04 to 0.07 gallons per square yard (residual for emulsion) unless otherwise directed.

Mixtures were dumped from end-dump haul trucks into a Roadtec SB2500 material transfer machine operated from the track's inside lane so that only the paving machine operated on the actual test sections. Compaction was accomplished by at least three passes of a steel-wheeled roller. The roller was capable of vibrating during compaction; however, this technique was not used on every test section. After the steel-wheeled roller was removed from the pavement mat, the contractor continued rolling the mat with a rubber tire roller until the desired density was achieved. A finish roller was then used to eliminate any marks left by the rubber tire roller.

1.6 Trafficking Operations

Trafficking for the 2015 Test Track was applied in the same manner as with previous cycles. Two shifts of professional drivers operated four trucks pulling triple flatbed trailers (Figure 3) and one truck pulling a triple box trailer from 5 a.m. until approximately 10:40 p.m. Tuesday through Saturday. Trafficking began on October 8, 2015 and ended November 28, 2017. The total traffic applied to the test sections during this cycle was 10,009,457 ESALs.

Figure 3 Heavily Loaded Triple-Trailer used for Accelerated Loading on the Test Track

Axle weights for each of the five trucks are shown in Table 1. On some occasions, either due to a specialized study or mechanical malfunction, trailers were removed from the operation. This left the truck pulling either a single flatbed trailer or a combination of double flatbeds.

Table 1 Axle Weights (lbs.) for the 2015 Truck Fleet

Truck ID	Steer	Tandem		Single				
	Axle 1	Axle 2	Axle 3	Axle 4	Axle 5	Axle 6	Axle 7	Axle 8
1	10,150	19,200	18,550	21,650	20,300	21,850	21,100	19,966
2	11,000	20,950	20,400	20,950	21,200	21,000	20,900	20,900
3	10,550	20,550	21,050	21,000	21,150	21,150	21,350	20,850
4	10,550	21,050	20,700	21,100	21,050	21,050	20,900	21,050
5	11,200	19,850	20,750	20,350	20,100	21,500	19,500	20300
Avg.	10,680	20,320	20,290	20,760	20,760	21,310	20,550	20,613
COV, %	3.9	3.9	4.9	2.2	2.5	1.7	3.6	2.2

1.7 Performance Monitoring

Performance of the test sections was evaluated with a comprehensive range of surface measurements. Additionally, the structural health and response of the structural sections were routinely evaluated using embedded stress and strain gauges and falling-weight deflectometer (FWD) testing. Table 2 summarizes the performance monitoring plan. Rut depths, International Roughness Index (IRI), mean texture depth, and cracking results were reported on the Test Track website.

Table 2 NCAT Test Track Performance Monitoring Plan

Activity	Sections	Frequency	Method
Rut depth	all	weekly	ARAN van, AASHTO R 48
Mean texture depth	all	weekly	ARAN van, ASTM E1845
Mean texture depth	select	quarterly	CTM, ASTM E2157-09
International Roughness Index	all	weekly	ASTM E950, AASHTO R 43
Crack mapping	sponsored	weekly	Jason 3000
FWD	structural	3 times/mo.	AASHTO T 256-01
Stress/strain response to live traffic	structural	weekly	NCAT method
Pavement temperature at four depths	all	hourly	Campbell Sci. 108 thermistors
Pavement reflectivity/albedo	sponsored	quarterly	ASTM E 1918-06
Field permeability	OGFC/PFCs	quarterly	NCAT method
Core density	sponsored	quarterly	ASTM D979, AASHTO T 166
Friction	all	monthly	ASTM E274, AASHTO T 242
Friction	select	quarterly	DFT, ASTM E1911
Tire-pavement noise	all	quarterly	OBSI, AASHTO TP 76-11, CPX, ISO 11819-2, Absorption, ASTM E1050-10

1.8 Laboratory Testing

Mixture samples for quality assurance (QA) testing were obtained from the beds of the haul trucks using a sampling stand located at the track. Typical quality assurance tests (listed in table 3) were conducted immediately on the hot samples and the results were reviewed by the respective test section sponsor for acceptance. In cases where the QA results did not meet sponsor approval, the mixture placed on the section was removed, adjustments were made at the plant, and another production run was made until the mix properties were satisfactory. Results of the QA tests and the mix designs for each layer for all test sections were reported on the Test Track website.

Table 3 Tests Used for Quality Assurance of Mixes

Test Description	Test Method	Replicates
Splitting samples	AASHTO T 328-05	as needed
Asphalt content	AASHTO T 308-10	2
Gradation of recovered aggregate	AASHTO T 30-10	2
Laboratory compaction of samples	AASHTO T 312-12	2
Maximum theoretical specific gravity	AASHTO T 209-12	2
Bulk specific gravity of compacted specimens	AASHTO T 166-12	2
Mix moisture content	AASHTO T 329-15	1

NCAT staff obtained large representative samples of each experimental mixture placed on the Test Track for additional testing by diverting mix from the conveyor of the material transfer machine going into the paver into the bucket of a front-end loader. The front-end loader then brought the mix to the rear of the track laboratory where it was shoveled into five-gallon buckets and labeled. In total, nearly 1300 buckets of mix were sampled for additional testing. Samples of the asphalt binders were also obtained at the plant for characterization.

A testing plan for advanced characterization of the 22 unique mixtures was established to meet section-specific and general Test Track research objectives. Table 4 summarizes the tests and materials/layers that were typically evaluated. Results of these tests are maintained in a database at NCAT.

Table 4 Summary of Testing for Advanced Materials Characterization

Test Description	Test Method	Material or Layer
PG grade	AASHTO R 29-08	Tank binders and recovered binders from mixes containing RAP, RAS, and/or WMA (No GTR modified binders were recovered)
Multiple stress creep recovery	AASHTO TP 70-09	Same as above
Cantabro	AASHTO TP 108-14	ALDOT and ODOT OGFCs, NCAT Cracking Group surface mixes, KTC mixes
Moisture susceptibility	AASHTO T 283-14	KYTC mixes
Hamburg wheel tracking	AASHTO T 324-14	FDOT surface cracking study, KYTC mixes, Collaborative Aggregates mix
Dynamic modulus	AASHTO TP 79-13	FDOT surface cracking study, TDOT Thinlay mix
Energy ratio	Univ. of Florida	Surface mixes from NCAT Cracking Group and FDOT cracking experiments
SCB-Jc (Louisiana)	LADOTD TR 330-14	Surface mixes from NCAT Cracking Group and FDOT cracking experiments
Overlay tester (Texas)	Tex-248-F	Surface mixes from NCAT Cracking Group, FDOT cracking, and KYTC mix design experiments
Overlay tester (NCAT modified)	NCAT	Surface mixes from NCAT Cracking Group and FDOT cracking experiments
Illinois Flexibility Index Test	IL TP 405	Surface mixes from NCAT Cracking Group, Collaborative Aggregates mix, and FDOT cracking experiment mixes
IDEAL Cracking Test	Texas A&M Trans. Inst.	Surface mixes from NCAT Cracking Group experiment
NCAT TWPD, Dynamic Friction Test, and Circular Track Meter	ASTM E1911, ASTM E2157	TDOT Thinlay mix

1.9 Key Findings from Previous Cycles

Many highway agencies have used Test Track findings to refine their materials specifications, construction practices, and pavement design procedures for asphalt pavements. This section summarizes key track research findings resulting in more cost-effective asphalt mixtures, refined specifications, and improved pavement designs for the sponsoring agencies. Some of the findings have already been implemented by several states and have the potential for broader implementation. These key findings are organized into the following six areas:

1. Mix design,
2. Aggregate characteristics,
3. Binder characteristics,
4. Structural pavement design and analysis,
5. Relationships between laboratory results and field performance, and
6. Tire-pavement interaction.

Mix Design

Fine-Graded vs. Coarse-Graded Mixtures. In the early years of Superpave implementation, coarse-graded mixtures were promoted to improve rutting resistance. However, that notion was called into question when the results of Westrack showed that a coarse-graded gravel mix was less resistant to rutting and fatigue cracking than a fine-graded mix with the same aggregate. In the first cycle of the NCAT Test Track, the issue was examined more completely. Twenty-seven sections were built with a wide range of aggregate types to compare coarse-, intermediate-, and fine-graded mixtures. Results demonstrated that fine-graded Superpave mixes perform as well as coarse-graded and intermediate-graded mixes under heavy traffic and tend to be easier to compact, less prone to segregation, less permeable, and quieter (*1*). Based on these findings, many state highway agencies revised their specifications to encourage or require more fine-graded mix designs.

Design Gyrations. Another mix design issue dealt with the number of gyrations (N_{design}) used to compact specimens for mix design and quality assurance testing. The performance of mixes on the Test Track, along with data from field projects across the U.S. collected as part of NCHRP project 9-29 (*2*), demonstrated the Superpave N_{design} levels specified in AASHTO R 35 are too high. High N_{design} numbers tend to grind aggregate particles and break them down much more than what occurs during construction or under traffic, so high N_{design} levels do not represent what actually occurs in pavements. Some mix designers use coarse gradations to meet the volumetric mix design criteria, but those mixes are more challenging to compact in the field and tend to be more permeable, making the pavements less durable (*3*). Numerous Superpave and SMA mixes on the Test Track designed with 50 to 70 gyrations in the Superpave gyratory compactor (SGC) have held up to the heavy loading with great performance (*1*). Many states significantly reduced their N_{design} levels as a result of these findings on the Test Track.

Stone-Matrix Asphalt (SMA) Mixtures. Through every cycle of the Test Track, SMA test sections have performed very well under extreme trafficking. Over 30 different SMA mixtures have been evaluated prompting several states to adopt this premium mix type for interstates and other heavy traffic highways. Analysis of SMA and Superpave mix test sections with the same granite aggregate over a 17-year period showed that the SMA maintained a higher surface texture and friction and had a slight advantage in tire-pavement noise as measured with the onboard sound intensity (OBSI) method (*4*). Mississippi, Missouri, Georgia, and South Carolina have used SMA test sections to evaluate lower-cost aggregates which have helped make SMA more economical. Smaller NMAS (e.g. 9.5 mm NMAS) SMA mixtures have proven to be rut resistant and durable on the Test Track giving highway agencies confidence to use the smaller size SMA that can be constructed in thinner lifts. Alabama now primarily uses 9.5 mm NMAS SMA as the final structural layer for projects with design traffic >30 million ESALs.

High Reclaimed Asphalt Pavement (RAP) Content Mixtures. Six test sections built in the third cycle and trafficked through the fourth cycle were built to evaluate surface mixes containing 20% and 45% RAP. After carrying approximately 20 million ESALs, the sections in this experiment had practically no rutting, very little raveling, and small amounts of low severity surface cracking. The use of a softer virgin binder was shown to provide slightly better resistance to raveling and cracking of the 45% RAP mixes. No benefit was observed for using polymer-modified virgin binder in the mixes with 20% or 45% RAP (*5*). In the fourth cycle, structural test sections built with 50% RAP in each asphalt layer outperformed companion virgin test sections in all performance measures, including fatigue cracking. The Mississippi DOT also sponsored a test section containing a 45% RAP surface layer in the fourth cycle that was evaluated for two cycles. While the Mississippi 45% RAP mix contained a PG 67-22 binder, performance on the track was similar to a 15% RAP mix with polymer-modified PG 76-22 (*6*). High RAP mixtures that provide equal or better performance to traditional mixes with moderate RAP contents can yield significant cost savings for highway agencies.

Warm-Mix Asphalt (WMA). An early version of the Evotherm WMA technology was used in the repair of two test sections that had extensive damage near the end of the second research cycle. The two WMA test sections were opened to heavy loading from the track fleet immediately after construction. Both test sections remained in service throughout the 2006 cycle, with rutting performance comparable to HMA for 10.5 million ESALs and no cracking. One of the sections was left in place at the start of the 2009 cycle, enduring more than 16 million ESALs before the test section was used for a different experiment. The performance of those test sections was early evidence that WMA could hold up to extremely heavy traffic. Additional WMA test sections with foamed asphalt and chemical additives built in 2009 also performed very well on the Test Track and helped agencies gain confidence to implement WMA despite concerns of rutting raised by laboratory test results (*5*). WMA technologies are now used during the production of 40% of all asphalt paving mixtures in the U.S. (*7*).

4.75 mm Nominal Maximum Aggregate Size (NMAS) Mix. Thin HMA overlays (less than 1¼-in. thick) are a common treatment for pavement preservation. Currently, about half of U.S. states utilize 4.75 mm NMAS mixtures in thin overlay applications. An advantage of the 4.75 mm mixtures is that they can be placed as thin as one half-inch, allowing the mix to cover a much larger area than thicker overlays. In the second track cycle, the Mississippi DOT sponsored a test section of 4.75 mm surface mix containing limestone screenings, fine crushed gravel, and a native sand. That 15-year old section has now carried more than 50 million ESALs with only 7 mm of rutting and minimal cracking. This section is proof that well-designed 4.75 mm mixes are a durable option for pavement preservation.

Aggregate Characteristics

Polishing and Friction. The South Carolina DOT sponsored a test section to assess the polishing behavior of a new aggregate source in 2003. A surface mix containing the aggregate was designed, produced, and placed on the track. Friction tests conducted at regular intervals showed a sharp decline in friction, indicating that the aggregate was not suitable for use in surface mixes. This experiment enabled South Carolina to make this assessment in less than two years without putting the driving public at risk. Mississippi and Tennessee DOTs sponsored test sections to evaluate blends of limestone and gravel on mix performance and friction. Both experiments showed that mixes containing crushed gravel provided good performance and the DOTs revised their specifications to allow more gravel in their surface mixes. Test sections sponsored by the Florida DOT used a limestone aggregate that was known to polish. When the sections became unsafe for the NCAT track fleet, a high-friction surface treatment using an epoxy binder and calcined bauxite aggregate was placed to restore friction. That surface treatment has provided excellent friction results and endured over 30 million ESALs.

Elimination of the Restricted Zone. Part of the original Superpave mix design procedure included the restricted zone, which limited certain gradations during mix design. Early experiments on the Test Track included several test sections containing a variety of aggregate types with gradations through the restricted zone. Those mixes performed very well and proved that the restricted zone was not necessary (*1*). The restricted zone was subsequently removed from Superpave specifications.

Flat and Elongated. Georgia DOT implemented SMA for interstate pavements in the early 1990s and soon after began to modify their open-graded friction course (OGFC) mixes toward a coarser, thicker porous European mix. Based on European specifications, Georgia established strict aggregate shape limits for these premium mixes. However, only a few aggregate producers invested in the extra processing needed to make the special coarse aggregate for these mixes. As prices for the special aggregates rose to more than four times the price of conventional coarse aggregates, the Georgia DOT used the track to evaluate the effect of using aggregates with a less strict flat and elongated requirement for their OGFC mix. Test Track performance showed that the lower cost aggregates actually improved drainage characteristics (*5*).

Aggregate Toughness. South Carolina DOT used an experimental mix on the Test Track to evaluate an aggregate with an LA abrasion loss that exceeded their specification limit. Aggregate degradation was assessed through mixture production, construction, and under traffic. Although the aggregate did break down more than other aggregates through the plant, the test section performed very well. Rutting performance on the track was similar to other sections, and there were no signs of raveling as indicated by texture measurements. Based on these results, the agency revised its specifications to allow the aggregate source.

Binder Characteristics

Effect of Binder Grade on Rutting. Superpave guidelines recommend using a higher PG grade for high-traffic volume roadways to minimize rutting. Results from the first cycle of testing showed that permanent deformation was reduced by an average of 50% when the high temperature grade was increased from PG 64 to PG 76 (*8*). This two-grade bump is typical for heavy traffic projects. These results validated one of the key benefits of modified asphalt binders. Also, Alabama DOT sponsored test sections in the first cycle to evaluate surface mixes designed with 0.5 percent more asphalt binder. Results of those sections showed that increasing the asphalt content of mixes containing modified binders did not adversely affect rutting resistance; however, mixes produced with neat binders were more susceptible to rutting in very high traffic conditions such as on the Test Track (*8*). Further analysis of rutting data in the second cycle considered many other mix design factors such as volumetric properties, aggregate gradation parameters, and SGC compaction indices. This analysis showed that the most influential factor on rutting was the binder high temperature performance grade (*1*).

Comparison of Different Types of Binder Modification. Experiments in the first cycle compared mixes containing PG 76-22 polymer-modified asphalt binders with styrene butadiene styrene (SBS) and styrene butadiene rubber (SBR). Test sections included dense-graded Superpave mixes, SMA mixes, and PFC mixes. Excellent performance was observed in all mixes produced with modified binders regardless of the type of modifier used (*1*). An experiment in the fourth and fifth cycles sponsored by the Missouri DOT and Seneca Petroleum compared the performance of a surface mix containing an SBS-modified binder and a binder modified with ground tire rubber (GTR). This experiment demonstrated that a GTR-modified binder provides the same performance as SBS modification (*5, 6*). Virginia DOT sponsored a PFC experiment comparing SBS-modified binder to a GTR modified binder. This study also found no difference in performance (*6*).

Evaluation of Alternative Binders. Increasing energy costs and the strong global demand for petroleum spurred the research and development of alternative materials to modify or replace petroleum-based asphalt binders. To this end, Test Track experiments were conducted in 2009 to evaluate Trinidad Lake Asphalt (TLA) and Thiopave pellets for use in asphalt mixtures. TLA pellets are made from a naturally occurring asphalt binder source in Trinidad, while the Thiopave pellets are produced based on a sulfur-modified asphalt formulation. Thiopave pellets must be used in combination with a warm mix additive to lower the mixing temperature to 275°F or less to reduce hydrogen sulfide emissions to an acceptable level. Both TLA and Thiopave pellets were added through the asphalt plant's RAP collar. For the TLA test section, all three asphalt layers were modified with 25% TLA based on weight of total binder. Two Thiopave test sections were constructed, one using 30% Thiopave in the base and intermediate mixes, the other section using 40% Thiopave in the base and intermediate layers. Surface layer mixes were not modified with Thiopave. Structural responses of the test sections to loads and the environment was compared to a control section using conventional asphalt mixtures. After 10 million ESALS, no cracking was observed in any of the test sections, rutting was below the 12.5-mm field threshold, and ride quality in each section was excellent. The pavement response measurements indicated that all of the test sections were structurally sound and proved the engineering viability of the alternative binders (*5*).

Structural Design and Analysis

Asphalt Layer Coefficient for Pavement Design. A recent survey indicates that 14 states have implemented the AASHTOWare Pavement ME program for designing asphalt pavements and overlays (*9*). Several other DOTs use their own ME based design program. However, the majority of DOTs continue to design pavements using the empirical pavement design method largely based on the AASHO Road Test in the late 1950s. A key input in the empirical design method is the layer coefficient of each layer. Many DOTs use 0.44 as the layer coefficient for asphalt concrete established during the AASHO Road Test, long before modern mix design methods, polymer modification, modern construction equipment and methods, and quality assurance specifications. A study funded by the Alabama DOT re-examined the asphalt layer coefficient using the performance and loading history of all structural sections. These test sections represented a broad range of asphalt thicknesses, mix types, bases, and subgrades. The analysis indicated that the asphalt layer coefficient should be increased from 0.44 to 0.54 (*10*). This 18% increase in the layer coefficient translates directly to an 18% reduction in the design thickness for new pavements and overlays. ALDOT implemented the new layer coefficient in its pavement design practice in 2010 and estimates savings of $25 to $50 million per year in construction costs (*11*).

Perpetual Pavement Design Validation. The Perpetual Pavement design concept has been validated using several Test Track sections. This design approach is based on engineering each layer in the pavement to withstand critical stresses so that damage never occurs in lower layers of the structure. On a life-cycle cost basis, Perpetual Pavements are more economical than traditional pavement designs and are less disruptive to traffic since pavement rehabilitation is reduced. Two of the original 2003 structural sections carried more than three times their “design traffic” based on the 1993 AASHTO guide with only minor surface damage before the sections were replaced for another experiment. In the 2006 cycle, Oklahoma sponsored two sections to further validate the concept for pavements built on a very soft subgrade. One of the sections was designed using the 1993 AASHTO Pavement Design Guide and the other section was designed using the PerRoad program. The conventional design resulted in a 10-inch asphalt cross-section, whereas the perpetual design was 14 inches thick. Results validated the concept of limiting critical strains to eliminate bottom-up fatigue cracking (*12*). Economic analysis of the two pavement design alternatives demonstrated that Perpetual Pavement is more cost-effective in a life-cycle cost comparison (*13*).

Measured Performance versus Pavement ME Predicted Performance. Fifteen structural study test sections were analyzed with the AASHTOWare Pavement ME program using the default national calibration coefficients (*14, 15*). For almost all sections, the Pavement ME program over-predicted rutting, generally with errors in the range of 70 to 100%. The rutting predictions for most sections were significantly improved after calibrating the model coefficients. Pavement ME fatigue cracking predictions with the default coefficients were also poor for the majority of the sections. In about half of the cases, the Pavement ME program grossly under-predicted fatigue cracking, but it over-predicted the amount of fatigue cracking in a few cases. Attempts to adjust the fatigue model coefficients did not improve the overall correlation of predicted versus measured fatigue.

Relationships between Laboratory Results and Field Performance

Air Voids. Air voids in laboratory-compacted specimens is one of the most common pay factors for asphalt pavements. The Indiana DOT sponsored research in the third cycle to identify an appropriate lower limit for this acceptance parameter. Surface mixes were intentionally produced with air voids between 1.0 and 3.5% by adjusting the aggregate gradation and increasing the asphalt content. Results showed that rutting increased significantly when the air voids were less than 2.75% (*12*). When test results are below that value and the roadway is to be subjected to heavy traffic, removal and replacement of the surface layer is appropriate. It is important to note that the experiment used only virgin mixes with neat (unmodified) asphalt binder. Other surface mixes containing modified binders or high recycled asphalt binder ratios that were produced with air voids below 2.5% have held up very well under the extreme traffic on the track.

Asphalt Pavement Analyzer (APA). Several DOTs use the APA (AASHTO T 340) to assess the rutting potential of asphalt mix designs. In the first and second cycles, APA tests were conducted on N_{design} samples and the results did not correlate well with field performance. In the first cycle, the correlation yielded an R^2 of 0.31 (*8*) and the second cycle correlation yielded an R^2 of 0.28 (*1*). In the third cycle, APA tests were conducted on specimens compacted to 7.0 ± 0.5% in accordance with the current AASHTO procedure. Based on the results of 11 surface mixes, the correlation improved significantly (R^2 = 0.63). From this correlation, a maximum APA criteria of 5.5 mm was recommended for heavy traffic surface mixes (*12*). Another study in the fourth cycle with 14 surface mixes found a slightly better correlation (R^2 = 0.70) between the APA test results and rutting on the Test Track and confirmed the 5.5 mm criterion (*5*).

Hamburg Wheel Tracking. Popularity of the Hamburg wheel tracking test has increased in recent years and numerous state DOTs now have Hamburg requirements for mix design approval. The test is considered to be a proof test for rutting and moisture damage susceptibility. Although there are no national criteria for Hamburg results, many highway agencies set the maximum rut depth between 4 and 12.5 mm at 20,000 wheel passes. NCAT conducted the Hamburg test in accordance with AASHTO T 324 at 50°C on 18 mixtures from the fourth cycle. The Hamburg results correlated reasonably well (R^2 = 0.74) with rutting measurements on the track (*5*). None of the test sections had any evidence of moisture damage.

Flow Number. NCHRP Report 673, *A Manual for Design of Hot Mix Asphalt with Commentary*, and NCHRP Report 691, *Mix Design Practices for Warm Mix Asphalt*, both recommended the flow number (FN) test for assessing the rutting resistance of mix designs. The testing criteria and traffic level performance thresholds from these reports were adopted in AASHTO TP 79-13. Prior to those NCHRP reports, NCAT used a confined FN test with 10 psi and a repeated axial stress of 70 psi. A strong correlation (R^2 = 0.77) was found between the results of the confined FN test and rutting on the track in the second cycle. Using that method, a minimum FN of 800 cycles was recommended for heavy traffic pavements (*12*). In the fourth cycle, unconfined flow number tests conducted on HMA surface mixes had a weaker correlation (R^2 = 0.54) with measured rutting. However, all HMA results met the NCHRP Report 673 FN criteria for 3 to 10 million ESALs of traffic (*5*).

Lab Testing of Friction and Texture Changes. NCAT used Test Track data to validate a method for evaluating texture and friction changes of any asphalt surface layer subjected to traffic. The procedure involves making slabs of the pavement layer in the laboratory and subjecting the slabs to simulated trafficking in a three-wheel polishing device developed at NCAT. 70,000 cycles in the NCAT Three-Wheel Polishing Device simulates approximately five to eight years of traffic on the Test Track. The slabs are then tested for friction using the Dynamic Friction Tester (ASTM E1911) and texture using the Circular Track Meter (ASTM E2157). Excellent correlations were established between the friction results in the lab and the field. Texture data from the test sections on the track show that surface textures of dense-graded mixtures increase with time and traffic (*1*).

Tire-Pavement Interaction

Tire-Pavement Noise and Pavement Surface Characteristics. Noise generated from tire-pavement interaction is substantially influenced by the macrotexture and porosity of the surface layer. Tire-pavement noise testing on the track indicated that the degree to which these factors influence noise levels is related to the weight of the vehicle and tire pressure. For lighter passenger vehicles, the porosity of the surface, which relates to the degree of noise attenuation, is the dominant factor. For heavier vehicles (with higher tire pressure), the macrotexture of the surface and the positive texture presented at the tire-pavement interface has a greater influence (*12*).

New Generation Open-Graded Friction Course Mixes. Each of the previous cycles of the Test Track included new-generation open-graded friction course (OGFC) mixtures with different types of coarse aggregates including granite, gravel, limestone, sandstone, and slag. Testing has shown that OGFC surfaces, also known as PFCs, eliminate water spray and provide excellent skid resistance.

High-Precision Diamond Grinding. Smoothness is the most important pavement characteristic from the perspective of users. Occasionally, maintenance or rehabilitation of a pavement results in a rough area that needs to be corrected. Precision diamond grinding has been used on the track in each cycle to smooth out transitions between some test sections. None of the areas leveled with the grinding equipment have exhibited any performance issues (*1*). Areas where diamond grinding was done were subject to traffic for up to 10 years with no performance problems. No sealing was applied to these treated surfaces.

1.10 References

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CHAPTER 2 CRACKING GROUP EXPERIMENT: VALIDATION OF CRACKING TESTS FOR BALANCED MIX DESIGN

2.1 Background

For more than a decade, many asphalt technologists have come to realize that volumetric properties currently at the core of asphalt mix design and quality assurance testing are insufficient to ensure adequate long-term performance of asphalt pavements. One of the shortcomings of volumetric properties lies with their dependency on the correct and timely measurement of aggregate specific gravities. Issues regarding accurate aggregate specific gravity measurements and their impact on proper asphalt contents of mixtures are not adequately addressed in many current policies and specifications. Also contributing to interest in a new era of mix design and quality assurance testing are concerns rooted in the use of recycled asphalt materials. Economic and sustainability motivations have led to increased use of reclaimed asphalt pavement (RAP) and recycled asphalt shingles (RAS). However, some asphalt mixtures containing high recycled material contents can be more prone to cracking due to the lower strain tolerance of the recycled binders. To address this issue, some asphalt technologists have used softer grades of virgin asphalt binders or rejuvenators to help restore properties of the binders in recycled materials. Although this practice is promising, there is not a clear understanding of the degree of blending between the virgin binders and the recycled binders and how it affects the cracking performance of the recycled asphalt mixtures. In addition, the long-term effectiveness of the rejuvenators on recycled materials has not been comprehensively evaluated.

These concerns have bolstered research to develop mixture performance tests that are suitable for routine use. Many asphalt technologists now envision a new era of mix design and quality assurance using these asphalt mixture performance tests. Numerous tests have been developed by different organizations to evaluate a mixture's resistance to a specific type of distress. However, many of these tests are not user friendly, have very limited field validation data on which to establish criteria, lack ruggedness evaluations, and lack inter-laboratory studies on which to establish precision information.

2.2 Research Plan

In 2014-2015, the National Center for Asphalt Technology (NCAT) and MnROAD developed an experimental plan to validate asphalt mixture cracking tests. Two complimentary experiments were planned: an experiment to validate tests for top-down cracking and an experiment to validate tests for thermal cracking of asphalt mixtures. The experiment for validating top-down cracking tests would be based on new test sections built on the NCAT Test Track, and the experiment for validating thermal cracking would be based on new test sections built on MnROAD's mainline roadway on I-95 near Alberton, MN. It is worth mentioning that MnROAD later built another set of test sections for evaluating reflection cracking of asphalt overlays on a concrete pavement as part of the National Road Research Alliance (NRRA).

This section provides an interim report on the NCAT top-down cracking experiment, including a summary of the field performance and pavement response data through two years of trafficking and environmental exposure. Also provided are the results of six laboratory cracking tests from samples of the plant-produced mixtures at the time of construction.

For the NCAT Test Track Cracking Group experiment, seven surface mixtures were designed with a range of recycled materials contents, binder types and grades, and in-place densities. The goal in selecting these mixtures was primarily based on the need to have a range of field cracking performance in an experiment where other variables (e.g. traffic, environment, pavement structure) would not confound the results. Table 1 provides a summary of general mix descriptions, compositions, and expected cracking resistance based on the estimations of NCAT researchers.

Table 1 Summary of Surface Mixtures Used in the NCAT Top-Down Cracking Experiment

NCAT Test Section	Mixture Description	NMAS[a] (mm)	RAP Content	RAS Content	Target In-place Density	Expected Cracking Resistance
N1	Control (20% RAP)	9.5	20%	0%	93%	Good
N2	Control, Higher Density	9.5	20%	0%	96%	Better
N5	Control, Low Density, Low AC[b]	9.5	20%	0%	90%	Worse
N8	Control+5% RAS	9.5	20%	5%	93%	Worse
S5	35% RAP, PG 58-28	9.5	35%	0%	93%	Good
S6	Control, HiMA[c] Binder	9.5	20%	0%	93%	Better
S13	Gap-graded, Asphalt-rubber	12.5	15%	0%	93%	Better

[a] Nominal maximum aggregate size; [b] asphalt content; [c] highly modified asphalt

The mixtures were constructed as 1.5-inch surface lifts over highly polymer-modified intermediate and base layers of asphalt. The target thickness for both the intermediate and base layers was 2.25 inches per layer. The asphalt pavement cross-section was relatively thin for the heavy loading on the Test Track so that the surface layers would experience significant stress and strains but avoid bottom-up fatigue cracking by using the highly modified mix for intermediate and base layers.

2.3 Construction and Interim Performance

The as-constructed cross-sections of the experimental test sections are illustrated in Figure 1. Some variations in thicknesses of the layers were identified from construction surveys.

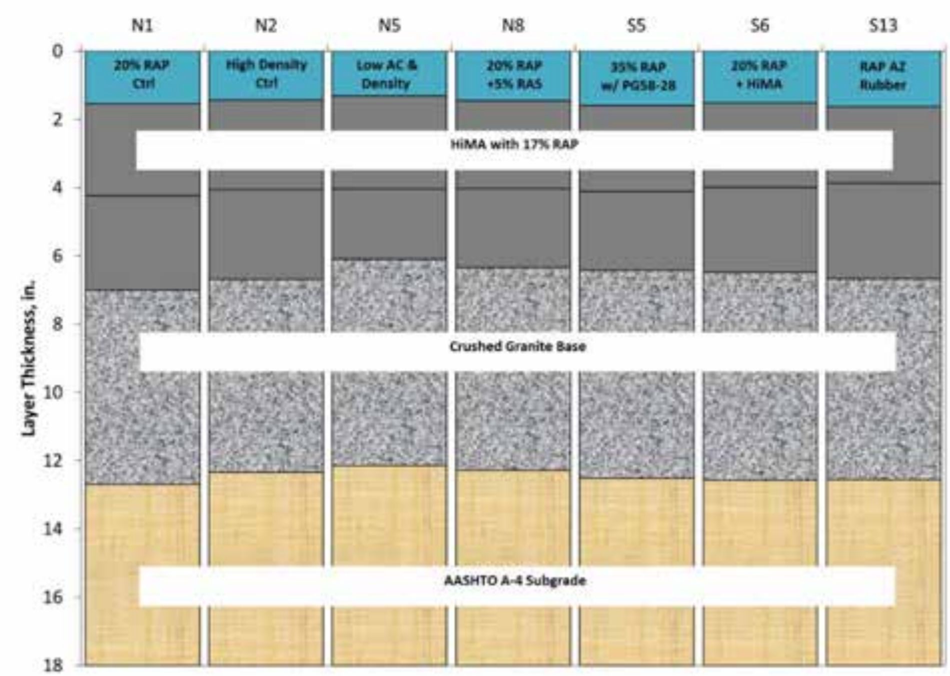

Figure 1 Cross-section of Cracking Group Test Sections on the NCAT Test Track

As-Constructed Mixture Properties

Table 2 provides a summary of the mix designs for each of the surface layers as well as the results from traditional quality control testing and construction of the mixtures. The control section mix constructed in Section N1 was close to most of its mix design targets, with the quality control (QC) asphalt content about 0.3% lower than design. The voids in mineral aggregate (VMA) drop from design to production was more than usual, which was attributed to the lower production aggregate bulk specific gravity (G_{sb}), which is estimated from production calculated aggregate effective specific gravity (G_{se}) and the difference between mix design G_{se} and G_{sb}. A similar VMA drop was evident for other test sections. Section N2 used the same 9.5 mm NMAS mix and was constructed consecutively, the difference being a higher target in-place density. The as-constructed density for N1 was 93.6% of the mix theoretical maximum specific gravity (G_{mm}), slightly below its target of 94%. The as-constructed density for N2 was 96.1%; 0.9% below its target density of 97%. However, the as-constructed density for N2 was 2.5% higher than N1, which is expected to have an impact on its performance. Section N5 was built to help assess the impact of a lower asphalt content and a lower in place density compared to the control section. The as-constructed asphalt content of N5 was 0.3% lower than the N1 control mix. The average in-place density of N5 was 90.3% of G_{mm}, which was 2.3% lower than the control section. Section N8 included 5% post-consumer RAS in its mix design. The mix also contained 20% RAP. The virgin binder in N8 was a PG 67-22, the same binder used in N1, N2, and N5. This mix is expected to have diminished cracking resistance. Section S5 contains 35% RAP but uses a softer virgin binder, a PG 58-28. The as-constructed total asphalt content and effective asphalt content of S5 was slightly higher than the control mix. Section S6 was similar to the control mix but used a highly-modified binder instead of the PG 67-22. The binder in S6 contains approximately 7% styrene-butadiene-styrene (SBS) polymer and graded as a PG 94-22.

The as-constructed total asphalt content and effective asphalt content of Section S6 was slightly higher than the control. The last test section in the experiment, S13, was substantially different than the other mixes. S13 is a gap-graded 12.5 mm NMAS mix containing an asphalt-rubber binder. The as-constructed total binder content of S13 was at least 1.5% higher than all of the other mixtures. S13 was designed following the Arizona Department of Transportation approach to asphalt-rubber mixes using the Marshall method. S13 contained 15% coarse fractionated RAP, which contributed 0.55% of binder to the mix. For each of the test sections, the as-constructed in-place densities were a little lower than the targets, but were within a reasonable tolerance of the expectations.

Table 2 Traditional Mix Design and Quality Control Properties of the NCAT Top-Down Cracking Group Test Sections

	N1 Control		N2 Control w/ High Density		N5 Control w/ Low Dens. & AC		N8 Control w/ 5% RAS		S5 35% RAP PG 58-28		S6 Control w/ HiMA		S13 Gap-Graded Asphalt-Rubber*	
Sieve Size	Design	QC	Design	QC	Design	QC	Design	QC	Design	QC	Design	QC	Design	QC
12.5 mm (1/2")	100	99	100	100	100	100	100	99	100	99	100	100	95	96
9.5 mm (3/8")	99	97	99	98	98	99	99	98	98	96	99	98	77	85
4.75 mm (#4)	74	67	74	70	74	73	70	66	74	73	74	67	38	35
2.36 mm (#8)	51	52	51	54	52	54	45	51	52	56	51	52	23	22
1.18 mm (#16)	39	41	39	43	41	42	35	41	41	44	39	42	18	19
0.60 mm (#30)	26	28	26	28	27	28	23	30	27	29	26	28	12	14
0.30 mm (#50)	15	15	15	15	15	15	14	17	15	16	15	15	7	8
0.15 mm (#100)	9	9	9	9	10	9	9	11	10	10	9	9	5	5
0.075 mm (#200)	6.2	5.4	6.2	5.6	6.3	5.7	6.1	7.1	6.3	6.3	6.2	5.4	3.3	3.6
Total Binder Content (P_b)	5.7	5.4	5.7	5.4	5.2	5.1	5.5	5.3	5.7	5.8	5.9	5.8	7.4	7.4
Eff. Binder Content (P_{be})	5.0	4.7	5.0	4.7	5.0	4.4	5.0	4.8	5.0	5.1	5.2	5.0	6.6	6.6
RAP Binder Ratio	0.19	0.20	0.19	0.20	0.21	0.21	0.20	0.20	0.33	0.33	0.18	0.19	0.07	0.07
RAS Binder Ratio	--	--	--	--	--	--	0.14	0.14	--	--	--	--	--	--
Dust/Binder Ratio	1.2	1.1	1.2	1.2	1.4	1.3	1.2	1.5	1.2	1.2	1.2	1.1	0.6	0.5
Rice Sp. Gravity (G_{mm})	2.474	2.469	2.474	2.468	2.493	2.478	2.483	2.492	2.481	2.472	2.470	2.459	2.418	2.402
Avg. Bulk Sp. Gravity (G_{mb})	2.375	2.375	2.375	2.372	2.355	2.348	2.383	2.415	2.382	2.393	2.371	2.384	2.273	2.319
Air Voids (V_a)	4.0	3.8	4.0	3.9	5.5	5.3	4.0	3.1	4.0	3.2	4.0	3.1	6.0	3.4
Agg. Bulk Gravity (G_{sb})	2.654	2.634	2.654	2.631	2.665	2.633	2.668	2.672	2.665	2.656	2.654	2.634	2.647	2.631
Avg. VMA	15.6	14.7	15.6	14.7	15.9	15.4	15.5	14.4	15.7	15.1	16.0	14.7	19.9	18.4
Avg. VFA	75	74	75	73	65	66	74	79	75	79	75	79	71	81
Compacted thickness (in.)	1.5	1.6	1.5	1.5	1.5	1.3	1.5	1.5	1.5	1.6	1.5	1.5	1.5	1.6
Mat Density (%G_{mm})	94.0	93.6	97.0	96.1	91.0	90.3	94.0	91.5	94.0	92.2	94.0	91.8	94.0	92.7

*50-blow Marshall hammer compaction used for mix design and QC.

Table 3 summarizes the properties of the virgin binders used in the experiment as well as the extracted and recovered binders from mixes obtained at the time of construction. Recovered binder for Section S13 (asphalt-rubber) was not obtained because most of the rubber remains as a solid during an extraction. Each of the virgin binders graded out higher than their designated grades.

Table 3 Properties of Virgin and Recovered Binders (°C)

Sample Type	Material	T_{cont} High Original	T_{cont} High RTFO	T_{cont} Int.	T_{cont} Low S	T_{cont} Low m	Continuous Grade	20 hr. Delta T_c
Virgin	PG 67-22	70.2	71.8	21.4	-27.6	-24.0	70.2 -24.0	-3.6
Virgin	PG 58-28	67.5	67.5	11.0	-35.6	-33.2	67.5 -33.2	-2.4
Virgin	PG 88-22 (HiMA)	97.1	94.8	15.6	-33.7	-31.9	94.8 -31.9	-1.9
Extracted	RAP stockpile	115.4	112.0	30.5	-22.9	-13.8	112.0 -13.8	-9.1
Extracted	N1: Control (20% RAP)	90.1	88.6	25.6	-26.0	-16.6	88.6 -16.6	-9.4
Extracted	N2: Control, High Density	91.0	89.9	26.8	-31.6	-15.9	89.9 -15.9	-15.7
Extracted	N5: Ctrl, Low Dens. & AC	89.2	88.0	25.1	-26.5	-18.5	88.0 -18.5	-8.0
Extracted	N8: Control+5% RAS	111.9	107.3	28.3	-25.4	-5.4	107.3 -5.4	-20.0
Extracted	S5: 35% RAP, PG 58-28	84.9	82.8	18.8	-32.3	-23.0	82.8 -23.0	-9.3
Extracted	S6: Control, HiMA	106.6	101.4	17.9	-33.6	-21.5	101.4 -21.5	-12.1
Extracted	CG Base/Intermediate	109.7	102.3	16.4	-33.3	-28.8	102.3 -28.8	-4.5

Interim Performance Results

From October 2015 to November 2017, trafficking of the test sections accumulated just over 10 million equivalent single-axle loads (ESALs). Field performance of the test sections at the end of the cycle is summarized in Table 4. All of the Cracking Group sections performed well over the two years of heavy loading with a few sections exhibiting rapid growth in the amount of cracking in the last few months, as shown in Figure 2. However, most of the measured cracking was very fine hairline cracks that are only visible to the trained eye. All of the sections have demonstrated excellent rutting resistance. There were some differences in the changes in the international roughness index (IRI) among the test sections, but the differences are not considered meaningful at this time. Likewise, the changes in surface texture through the cycle were similar except for S13, which has a different gradation and much higher binder content that the other surface mixtures in the experiment.

Table 4 Performance of NCAT Cracking Group Test Sections After 10 Million ESALs

NCAT Test Track Section	Mixture Description	Rutting (mm)	Change in IRI (in./mi)	Change in Mean Texture Depth (mm)	Cracking (% of lane area)
N1	Control	1.7	3	0.4	21.5
N2	Control, Higher Density	2.2	8	0.6	6.2
N5	Control, Low Density, Low AC	1.2	15	0.5	5.0
N8	Control+5% RAS	1.2	17	0.7	16.9
S5	35% RAP, PG 58-28	1.5	4	0.5	0
S6	Control, HiMA binder	1.4	11	0.6	0
S13	Gap-graded, asphalt-rubber	2.8	6	0.1	0

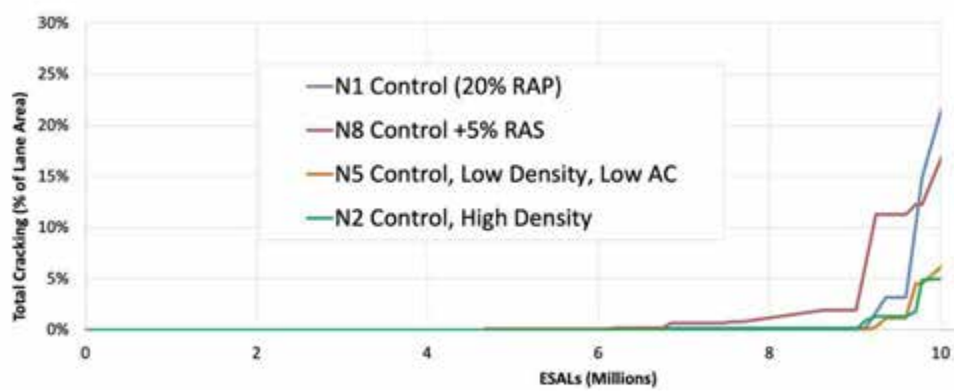

Figure 2 Plot of Cracking in the Cracking Group Test Sections Through the 2015-2017 Cycle

Figures 3 through 6 show photographs of cores taken on cracks at the end of the cycle from each of the test sections with observed cracking. The observed cracking was marked prior to coring with white spray paint. The direction of traffic was marked with an arrow using a silver paint pen. From these photographs, the cracking marked on Sections N1, N2, and N5 was so minor that they are hardly visible on the surface of the cores. In fact, there was no evidence of cracks from the sides of these cores, even on very close inspection. However, the cracking in Section N8, shown in the photographs of Figure 6, was much more evident and had even deteriorated by minor spalling. These cracks can also be seen to have propagated all the way through the surface layer but had not propagated through the interface with the intermediate layer. For all cores, the layers were well bonded and there was no evidence of moisture damage or segregation.

Although some cracking has begun to appear in several of the Cracking Group test sections, only N8 has progressed sufficiently for it to be meaningful. Therefore, all of these test sections will be left in place during the next cycle of the Test Track to allow for better separation of field cracking performance. This will help establish better relationships with the laboratory cracking test results.

Figure 3 Cores Taken from Section N1 (Control, 20% RAP)

Figure 4 Cores Taken from Section N2 (Control, Higher Density)

Figure 5 Cores Taken from Section N5 (Control, Low Density, Low AC)

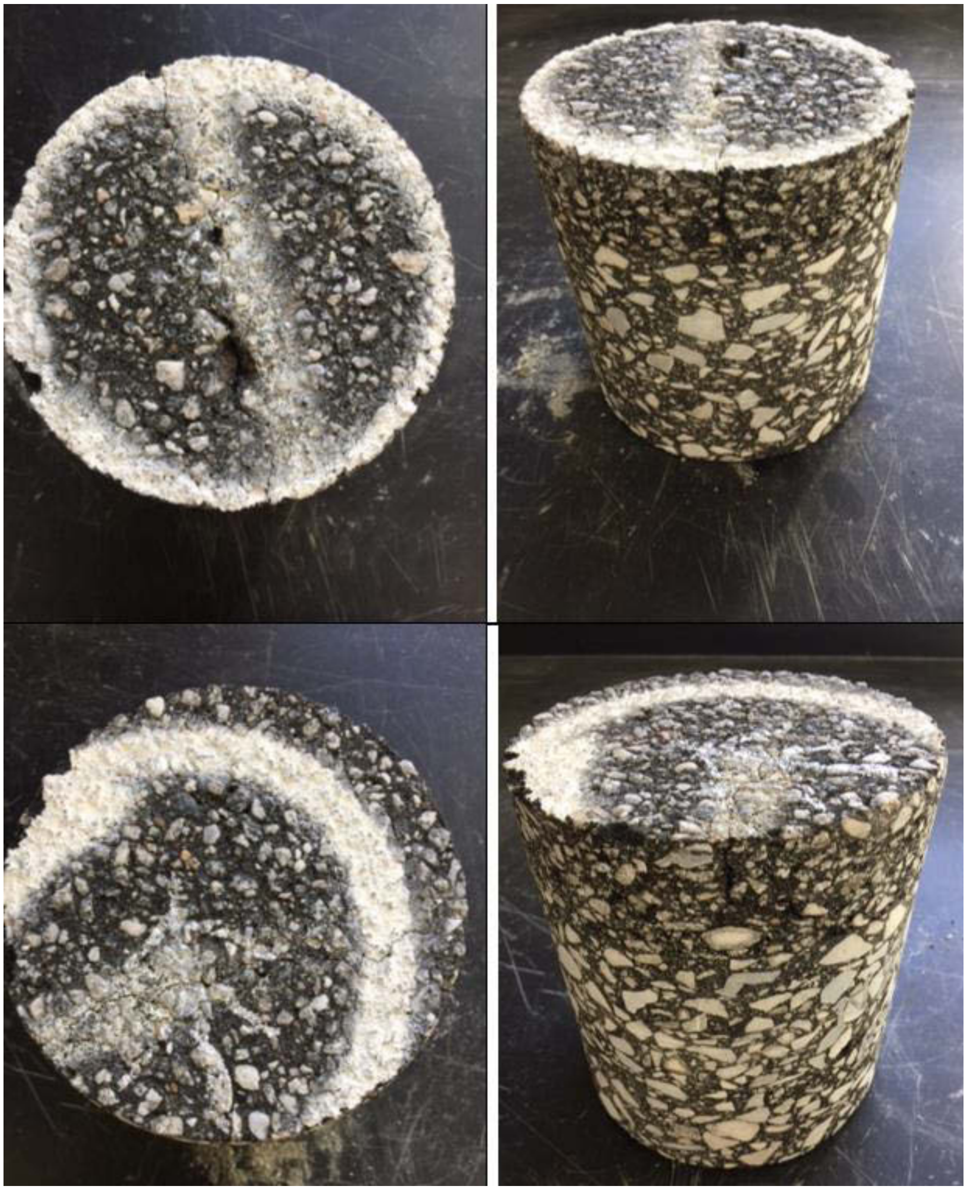

Figure 6 Cores Taken from Section N8 (Control+5% RAS)

2.4 Pavement Response Analysis

Part of the field investigation of the Cracking Group sections included measurements made from embedded instrumentation under truck loading along with back-calculation of in situ material properties from falling weight deflectometer (FWD) testing. The goals with each of these monitoring programs included the following:

- Characterize seasonal temperature effects on pavement responses;
- Evaluate differences in pavement responses driven primarily by surface lift mixture differences; and
- Quantify effects of pavement cracking on measured pavement responses through non-destructive testing.

To that end, each section in the experiment was instrumented with strain gauges (ASGs), earth pressure cells (EPCs), and temperature probes during the construction process. The ASGs were placed to measure bending tensile strain at the bottom of the asphalt concrete in the direction of traffic while the EPCs were placed at the asphalt concrete/granular base interface and granular base/subgrade interface to measure vertical pressures at those depths, respectively. While 100% of the EPCs survived installation and the entire two-year traffic cycle, the ASG survivability was extremely low and did not produce sufficient data to include in the following analyses. Data were collected from the embedded gauges twice per week (morning and afternoon) to establish both daily and seasonal measurement trends as discussed below.

The FWD testing program consisted of testing approximately three times per month along the inside, outside, and between wheel paths at four random locations in each section. Each set of deflection data at a given location consisting of three replicate drops at three drop heights representing 6, 9, and 12-kip loadings. The measured deflection data were used in EVERCALC 5.0 to back-calculate the layer properties to establish a time history of in situ properties during the two-year test cycle. Only data from the 9-kip load level resulting in backcalculated properties with a root-mean-square-error (RMSE) of less than 3% are presented below.

Measured Pavement Responses

Figures 7 and 8 show the base and subgrade pressure measurements, respectively, taken during the entire two-year test cycle for each section. As expected, the seasonal trends are evident where pressures increase during the summer months and decrease in the cooler months. These pressure changes result primarily from the effect temperature has on the asphalt modulus during the course of the year. Warmer temperatures decrease the asphalt concrete modulus, which leads to high pressures in the lower layers. The opposite occurs in the cooler months. It is difficult to discern appreciable differences in the subgrade measurements (Figure 7) between test sections as they all cover a very similar range. However, it appears that the S6 (Ctrl w/ HiMA) base pressure measurements (Figure 8) are substantially higher than the other sections. When examining these two data sets, none of the sections exhibit significant trends due to aging or damage that may occur over longer time periods.

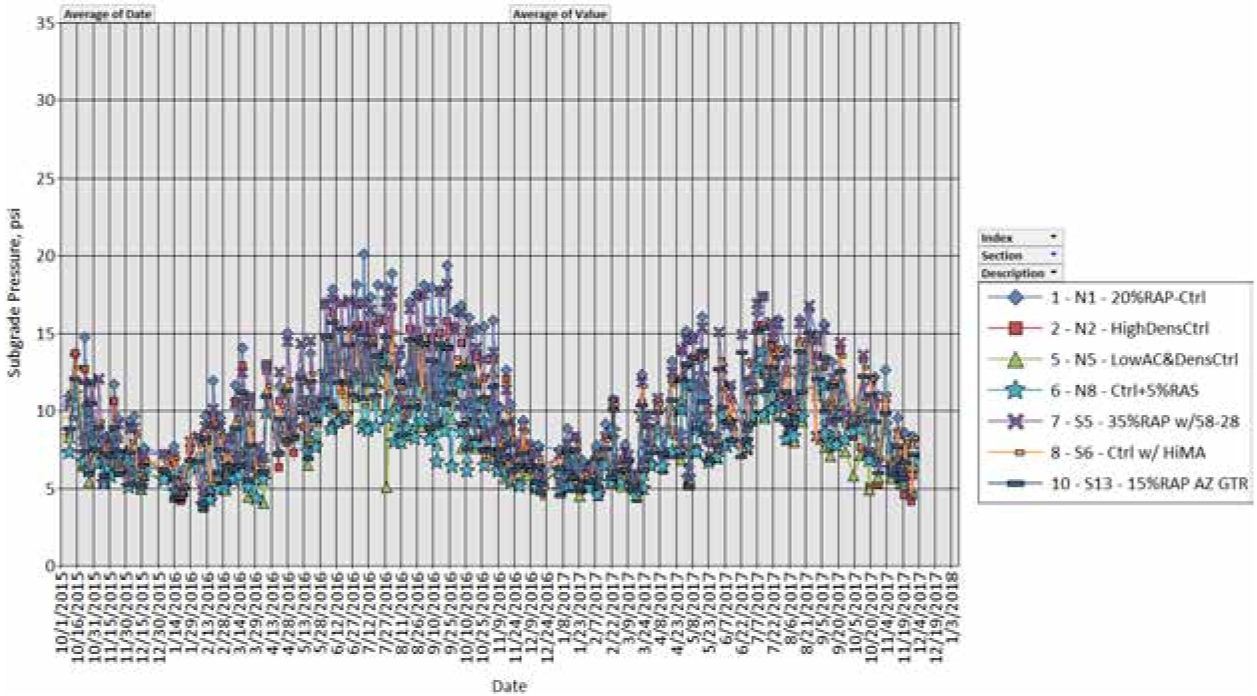

Figure 7 Subgrade Pressure Measurements

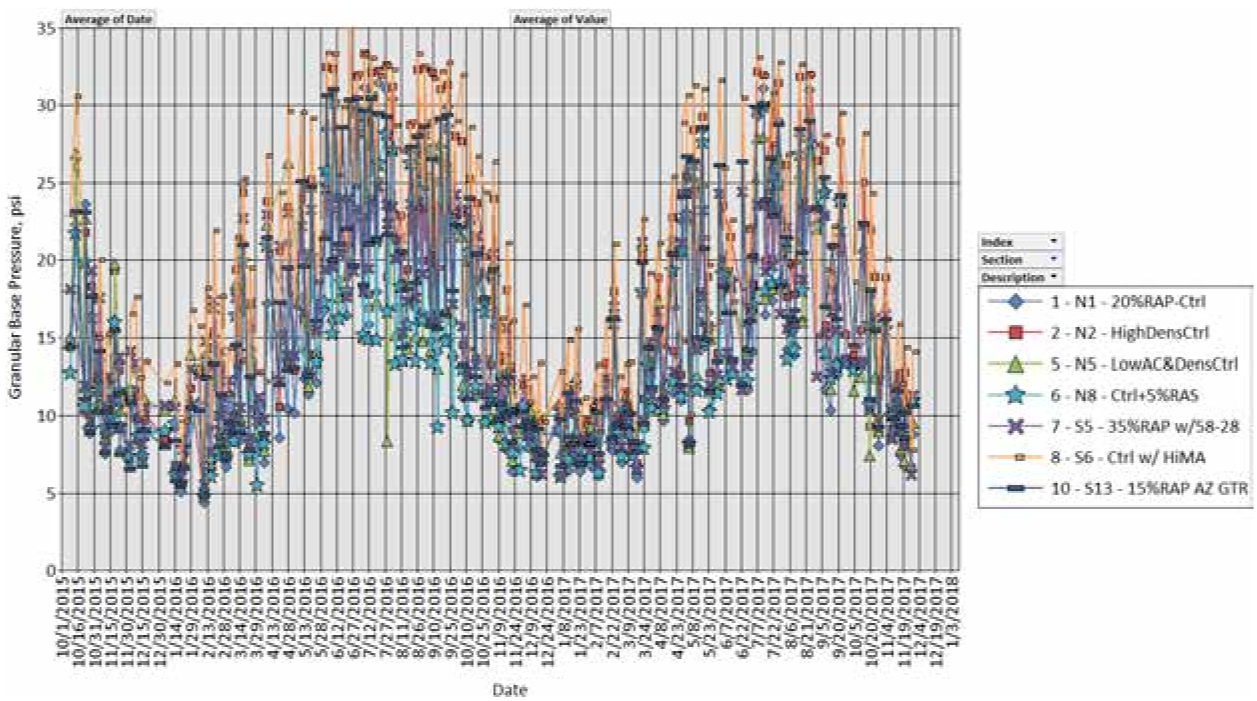

Figure 8 Base Pressure Measurements

As mentioned above, the change in base and subgrade pressures over time are primarily a function of the asphalt pavement temperature. Figures 9 and 10 demonstrate the strong influence of mid-depth asphalt temperature on the subgrade and base pressure measurements, respectively. Again, it is difficult to discern meaningful differences between the sections in the subgrade measurements, as the S6 (Ctrl w/ HiMA) base pressures appear higher than the rest.

For the most part, the sections experienced very similar stress levels, and by proxy, one can infer very similar strain levels with the exception of Section S6.

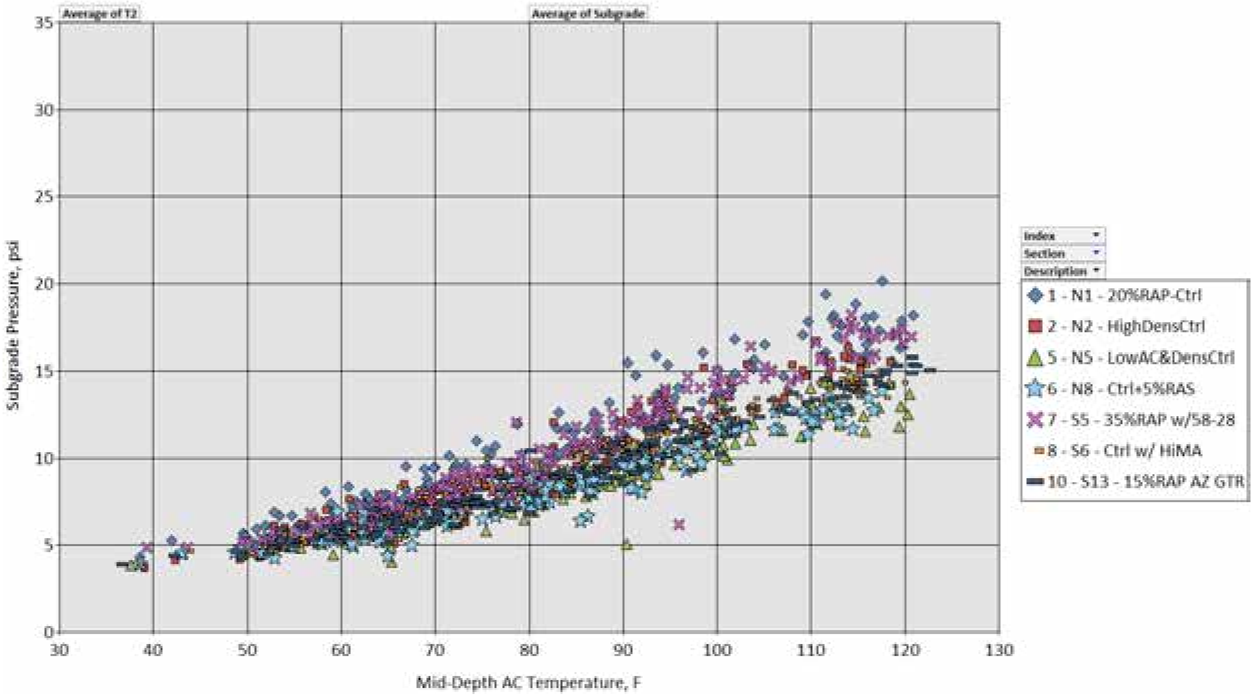

Figure 9 Subgrade Pressure Versus Mid-Depth Asphalt Temperature

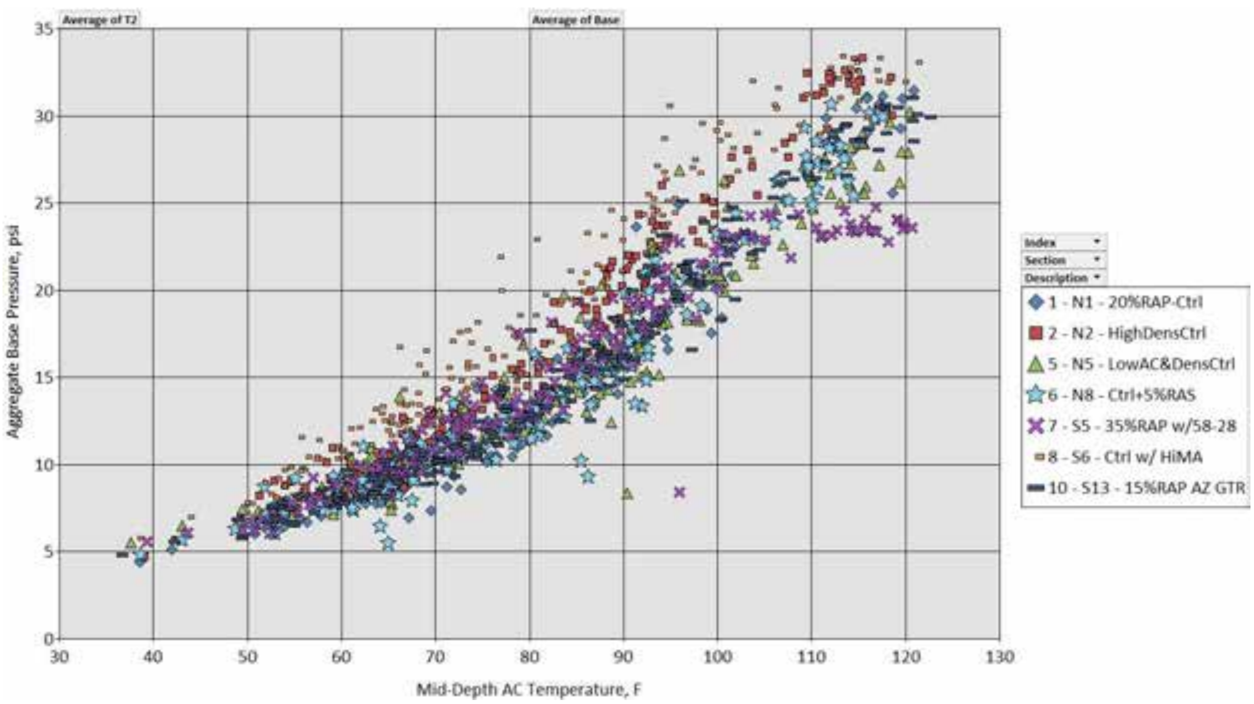

Figure 10 Base Pressure Versus Mid-Depth Asphalt Temperature

To factor out the temperature effects, the data were normalized to 68°F using section-specific regression functions derived from the data shown in Figures 9 and 10 and replotted against date as shown in Figures 11 and 12. The temperature normalization process followed a previously-established procedure (*1*). These plots, where the data are relatively constant over time, indicate that the sections are not experiencing sufficient damage at the surface of the pavement to clearly detect its impact deeper in the pavement structure. Presumably, with more traffic application over time, the effects will be measurable by these gauges.

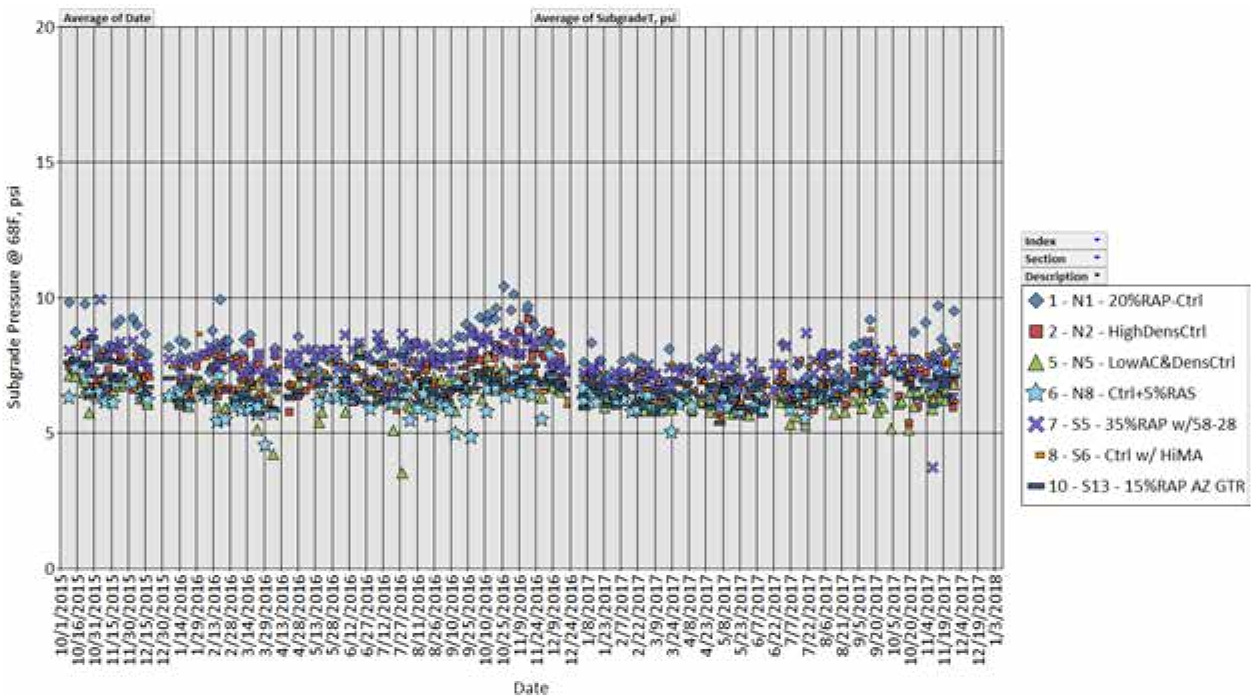

Figure 11 Subgrade Pressure at 68°F Versus Time

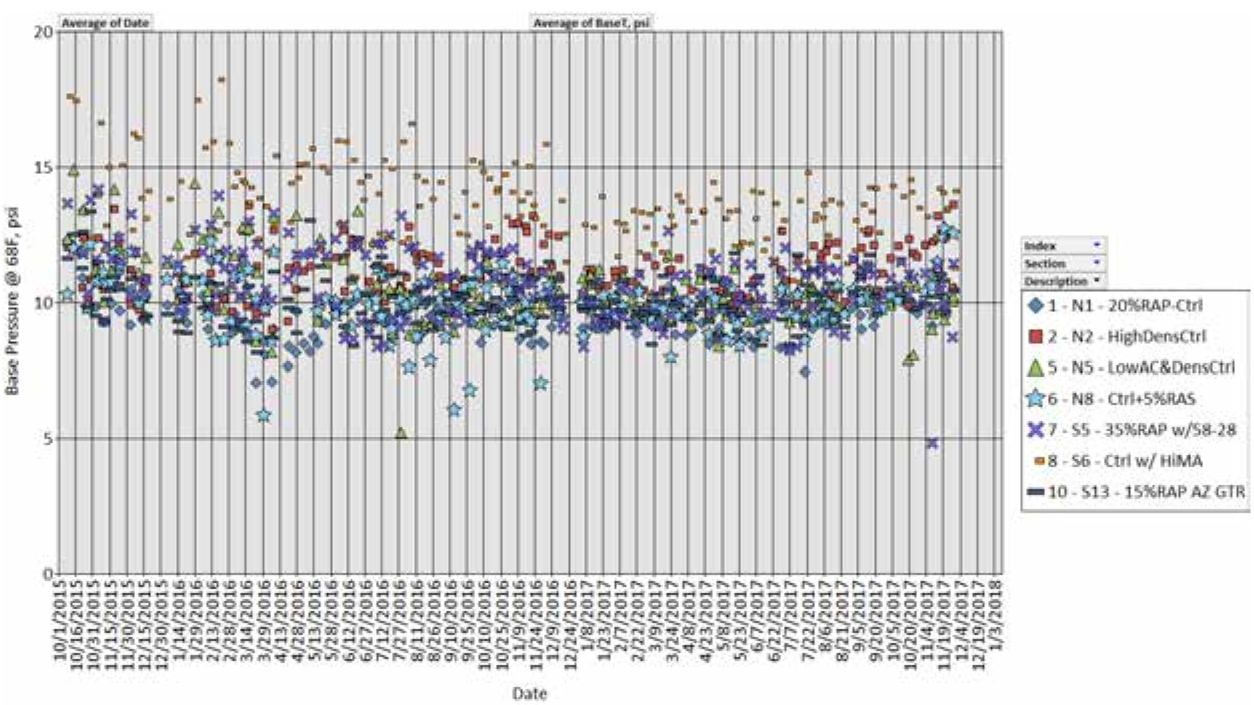

Figure 12 Base Pressure at 68°F Versus Time

The data in Figures 11 and 12 were used to create Figure 13, which shows the average and standard deviation of the measured pressures at 68°F. Tukey-Kramer statistical testing was conducted (α=0.05) to group sections within each subset (base and subgrade pressure, respectively). Symbols were overlaid on sections having statistically-similar values. For example, the stars in N5 and N8 indicate that their measured subgrade pressures were statistically equivalent. Sections with no symbol on the bar are statistically different than all other sections. Interestingly, the subgrade pressures in the control section (N1) were statistically higher than all the other sections, though one could argue the practical significance of less than 2 psi difference. Conversely, the base pressures were statistically lower than all the other sections. Again, the practical significance of less than 2 psi could be argued, but S6 (control w/ HiMA) stood out beyond this threshold and seemed to experience much higher base stress levels.

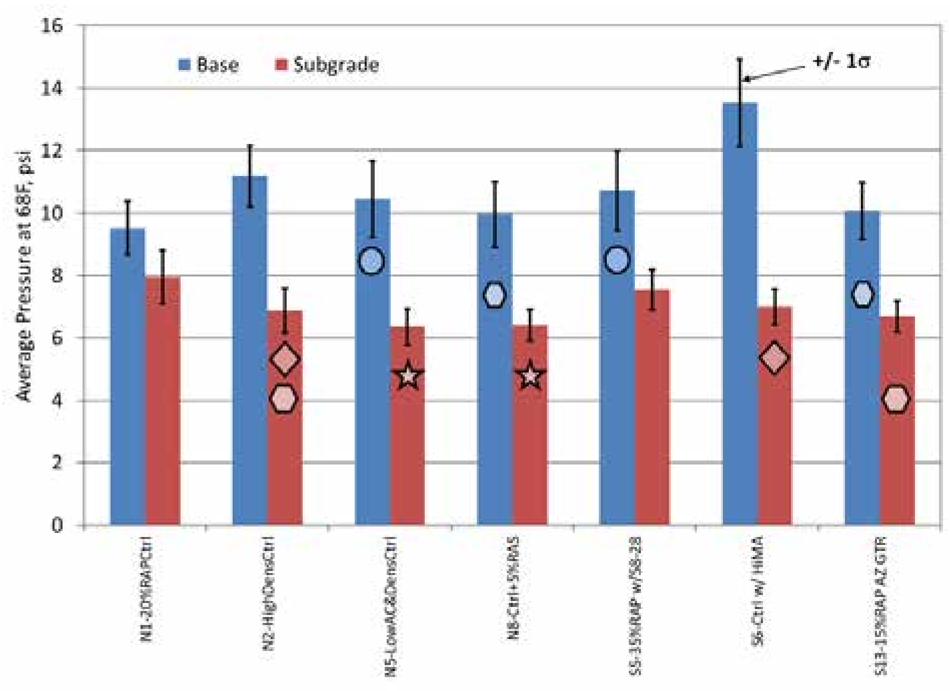

Figure 13 Average Pressure Measurements at 68°F

Backcalculated Asphalt Concrete Moduli

Backcalculated asphalt concrete moduli are plotted for each section versus date in Figure 14. These backcalculated asphalt moduli are, in essence, composite moduli for the three asphalt layers in each section. Similar to the measured pressure responses, the seasonal trends are clearly evident, and it should be noted that these values are section-wide averages based on the date of testing. Due to the temperature variation covering approximately an order of magnitude, it is difficult to discern any long-term potential aging or effects of pavement damage on this data set.

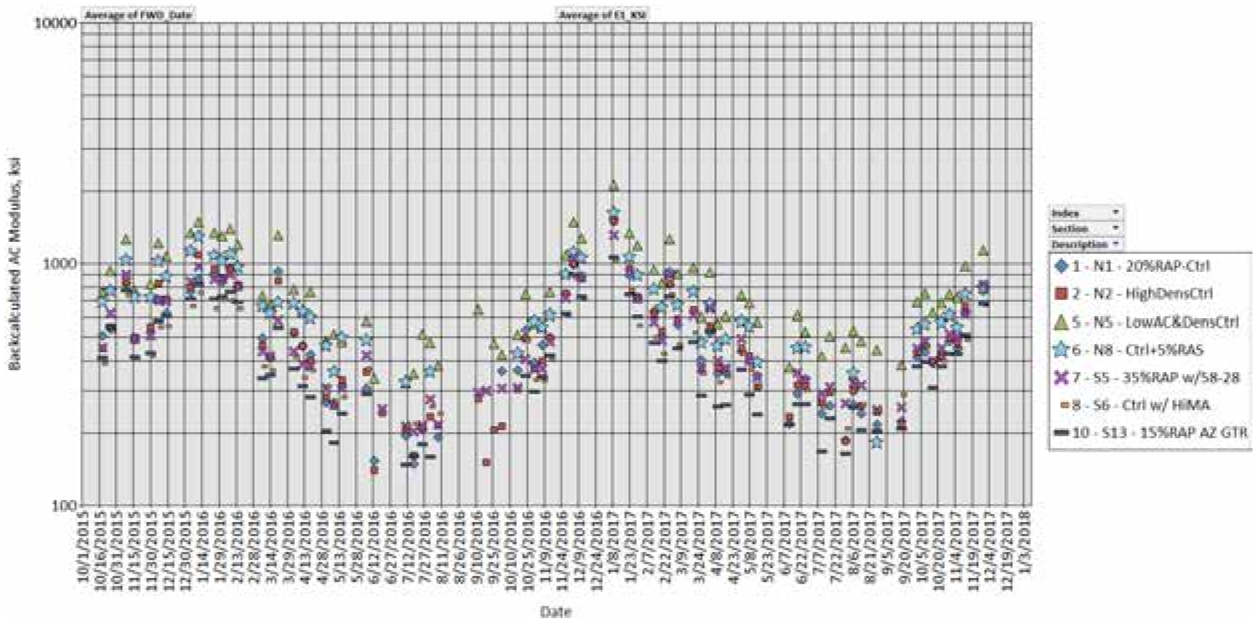

Figure 14 Asphalt Modulus Versus Date

A similar approach to that described above with the pressure data was taken to eliminate temperature as a factor in the backcalculated asphalt modulus data. Figure 15 shows the asphalt moduli versus temperature from each section where N5 (low AC & density control) appears significantly higher than the other sections. Again, the influence of temperature is clearly evident.

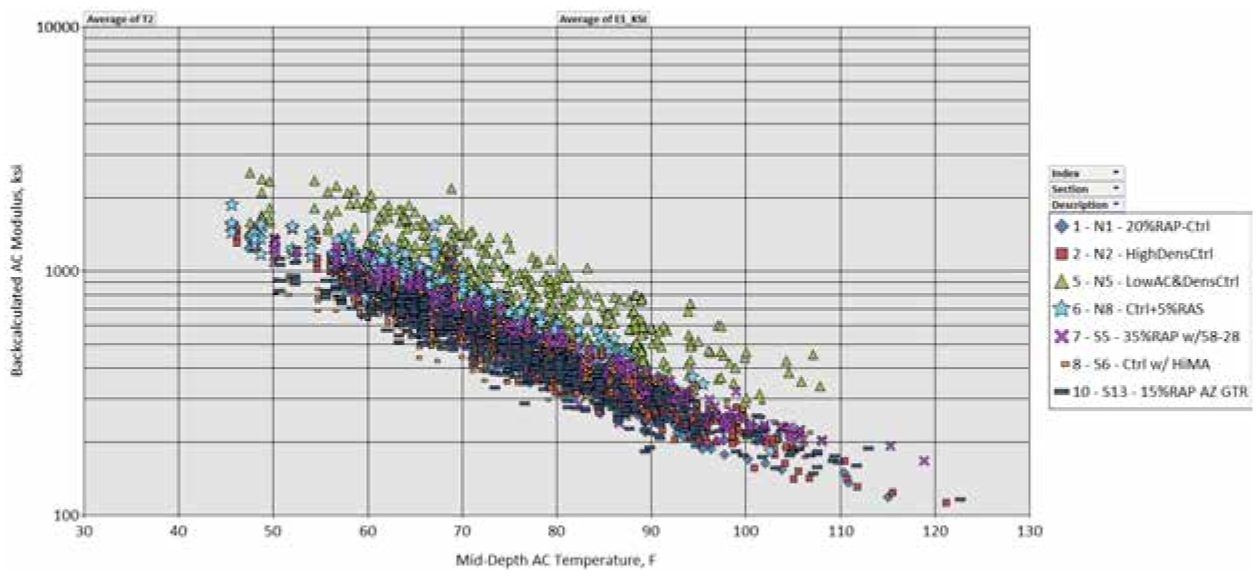

Figure 15 Asphalt Modulus Versus Mid-Depth Asphalt Temperature

Following previously-established procedures (*1*), each section's data in Figure 15 was normalized to 68°F and plotted in Figure 16. Tukey-Kramer statistical testing was again conducted, and all sections were found to be statistically different from each other. Section N5 had the highest asphalt modulus and S6 the lowest. The S6 finding was consistent with the base pressure measurements in that the highest pressure readings came from the section with the lowest modulus. The N5 finding was somewhat unexpected as the N8 (Control+5%RAS) was expected to be the highest.

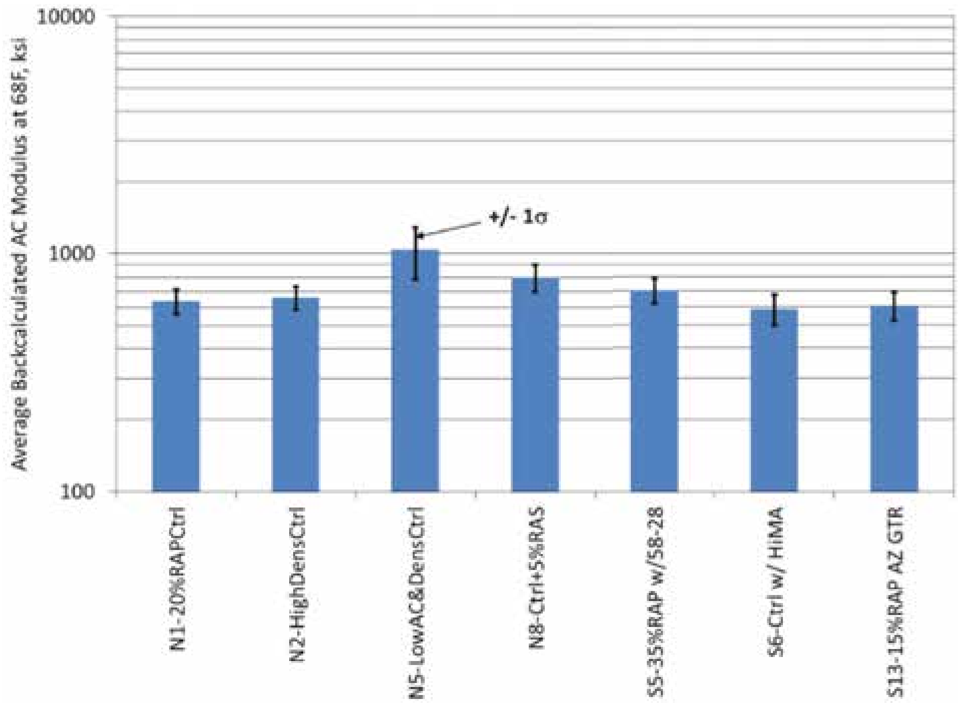

Figure 16 Asphalt Modulus at 68°F

Using the temperature normalization procedure, the data in Figure 15 were replotted at 68°F versus date in Figure 17 to look for signs of aging (modulus increase over time) or damage (modulus decrease over time). The data do not clearly indicate aging or damaging effects in any of the sections when shown in this fashion. However, unlike the pressure measurements which are obtained at one location in the section, FWD testing is conducted at 12 locations in each section (four random locations with three offsets at each location). Therefore, it could be that the data in Figure 17 did not have sufficient resolution to show damage at specific locations. The data were therefore subdivided into wheel paths since some sections had cracking in the wheel paths by the end of the trafficking. Figures 18 and 19 highlight the findings for Sections N1 and N8, respectively.

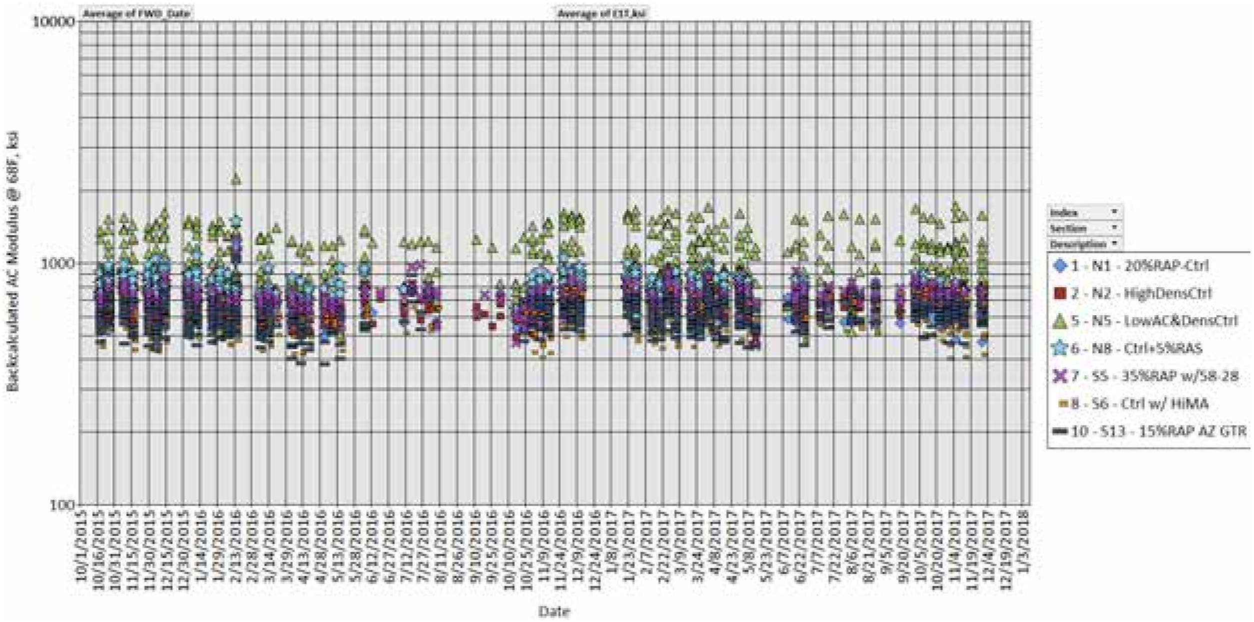

Figure 17 Asphalt Modulus at 68°F Versus Date

Figure 18 shows modulus subdivided by wheel path (B = between, I = inside and O = outside wheel path) for Section N1. Trendlines were fit to each data set, and although the R^2 were relatively low, they each had a slightly positive slope indicative of a very minor amount of aging. Had the cracking in this section been more severe, one would expect the modulus to decrease at some point, so the conclusion that the section is still structurally healthy is reasonable. The N1 data was representative of most of the other sections except for Section N8 shown in Figure 19.

Section N8 did have a significant amount of higher severity cracking at the end of the test cycle. This is evident in the data in Figure 19 where the modulus decreases over time in the inside and outside wheel paths while remaining relatively constant over time between wheel paths where no cracking was evident. Using the slope and R^2 of the trendlines, it is reasonable to conclude that the inside wheel path has experienced more pavement damage than the outside since the slope and R^2 are greater. No other section experienced this level of modulus change during the two-year testing period.

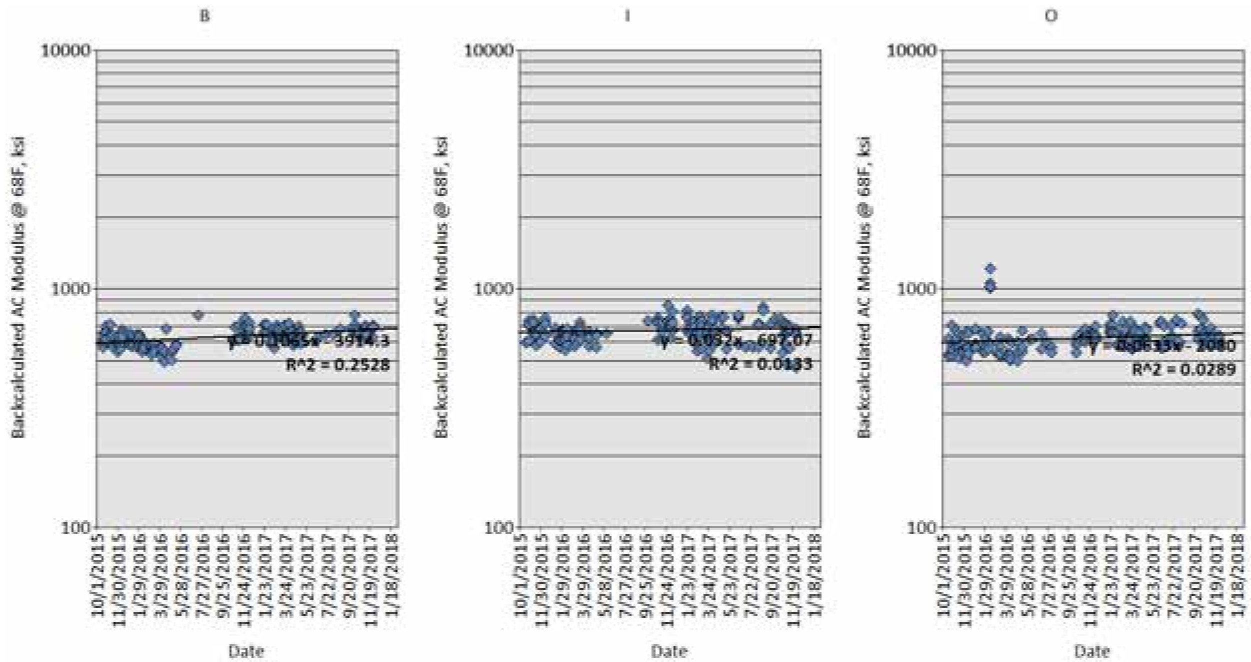

Figure 18 N1 (20% RAP Control) Asphalt Modulus at 68°F by Wheel Path Versus Date

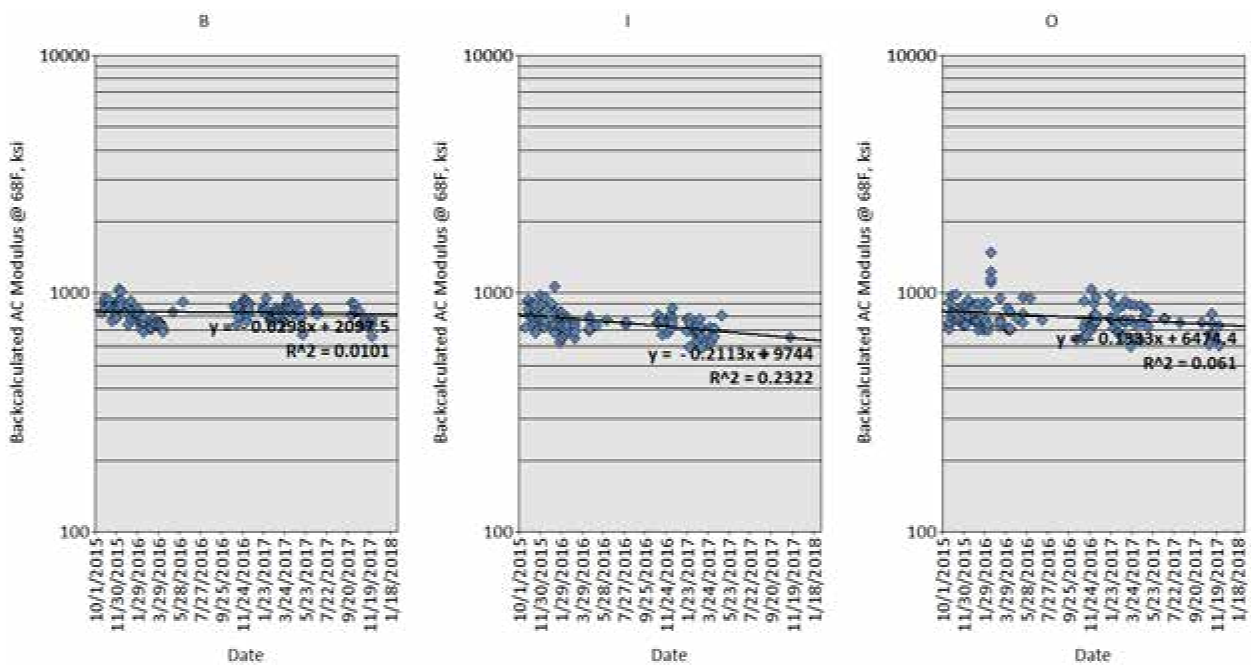

Figure 19 N8 (Contol+5% RAS) Asphalt Modulus at 68°F by Wheel Path Versus Date

2.5 Laboratory Testing Plan

Five laboratory cracking tests were initially selected by the sponsors as the preferred candidates for evaluating top-down cracking: Energy Ratio, Texas Overlay (TX-OT) test, NCAT modified Overlay Test (NCAT-OT), semi-circular bend test (SCB) (Louisiana method), and the Illinois Flexibility Index (I-FIT) test. The IDEAL Cracking Test (IDEAL-CT) was added to the experimental plan after the experiment was under way following discussions with the sponsors.

Cantabro tests were also conducted on the N_{design} specimens used for determining volumetric properties during mix design and construction quality control except for Section S13, as indicated in Table 2. This section presents the results of the cracking tests performed on specimens fabricated from the mixtures sampled during construction. In addition to testing the plant-produced mix, the initial laboratory testing plan also included testing of lab-prepared mixtures to represent samples that would be prepared as part of mix design. However, that work started much later in the research cycle and has not been completed at the time of this report. Soon after the project started, the NCAT team also decided that it was important to consider the effect of aging in the evaluation of surface layer cracking. Therefore, the testing plan was expanded to include laboratory aging of both lab-prepared and plant-produced mixtures. NCAT researchers first evaluated several options for aging of the mixtures and established a loose mix aging protocol of eight hours at 135°C (275°F) to simulate in-situ aging of a surface layer to point where cracking typically begins (*2*). Results of the cracking tests on the aged specimens will be reported at a later date.

Energy Ratio

The Energy Ratio (ER) protocol was developed as an indicator of top-down cracking by researchers at the University of Florida (*3*) and is based on an asphalt mix fracture mechanics model (*4*). A national standard for Energy Ratio has not been established. Three indirect tension (IDT) tests are conducted on the same specimens: resilient modulus (ASTM D7369), creep compliance (AASHTO T322), and indirect tensile strength (ASTM D6931). Figure 20 illustrates the properties determined from the three IDT tests in the ER protocol. Dissipated creep strain energy (DCSE) is an intrinsic property of an asphalt mixture. It is the energy required beyond the elastic region to initiate cracking. Energy Ratio is the dissipated creep strain energy threshold of the mixture, $DCSE_{HMA}$, divided by the minimum dissipated creep strain energy required to resist top-down cracking. A mixture with an ER of greater than 1.0 would be acceptable.

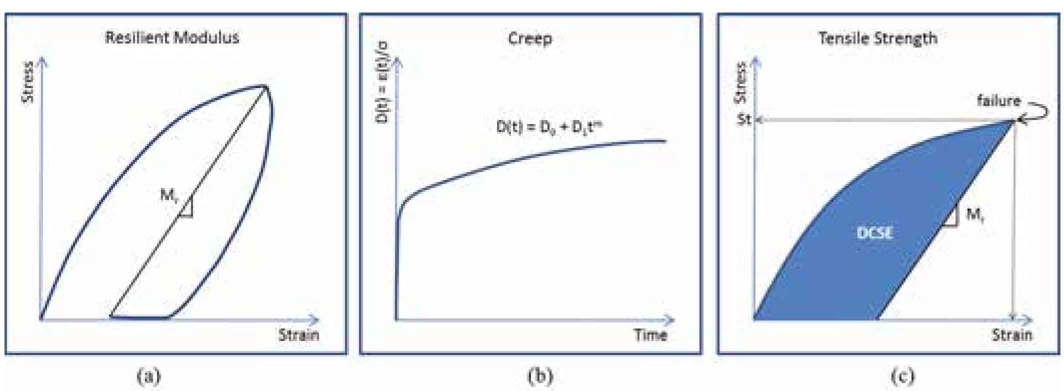

Figure 20 Parameters Determined by (a) Resilient Modulus, (b) Creep, and (c) Strength Tests

Florida researchers found that the ER criteria distinguished cracked and uncracked sections in 19 of the 22 pavements studied (*3*). 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 uncracked 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 5 shows the ER criteria by Roque, et al. (*3*) for mixtures with different traffic ranges and the supplemental criteria based on the $DCSE_{HMA}$.

Table 5 Energy Ratio Criteria

Mix Property	Criterion	
Energy Ratio	Traffic: Million-ESALs <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	

The resilient modulus, creep compliance, and indirect tensile strength tests were performed at 10°C using a servo-hydraulic testing device. For each mixture, three specimens 150-mm in diameter by 38 to 50-mm thick were cut from Superpave gyratory compactor (SGC) specimens. The air voids for these samples corresponded to the target densities listed in Table 1 with a tolerance of ± 0.5 percent. Horizontal and vertical extensometers were glued to both faces of the specimen and deformations were recorded during the tests. A trimmed mean of the deformations is used where the highest and lowest values are discarded. The analysis procedure does not allow for any characterization of variability, as the final values from each test are reported and used to calculate a single ER value. Data analysis was performed using a software package developed at the University of Florida, the details of which are documented elsewhere (*5*).

Texas Overlay Test

The Texas Overlay Test (TX-OT) was developed in the late 1970s to simulate reflective cracking of asphalt overlays on concrete pavements. The method was refined by Zhou and Scullion (*6*) and established as Texas DOT method Tex-248-F. The test was adapted for the Asphalt Mixture Performance Tester (AMPT) by one equipment manufacturer. OT specimens are cut from an SGC sample or a field core and glued to two metal plates. One plate remains fixed while the other plate moves back and forth in a saw-tooth cycle with a 0.635 mm (0.025”) maximum opening displacement. Each cycle is ten seconds long (five seconds of loading, five seconds unloading). Tests are performed at 25°C. The displacement simulates the expansion and contraction that occurs at the joints of concrete pavements due to temperature fluctuations. The peak load on the first cycle is measured and specimen failure is taken as the cycle where the load drops to 93% of the initial peak load.

Zhou and Scullion initially demonstrated that the OT is sensitive to testing temperature, maximum opening displacement, and asphalt binder content and type (*6*). A more in-depth sensitivity study was performed by Walubita et al. reviewing the current state of the OT in a number of labs in the United States (*7*). High variability (coefficient of variation > 30%) was reported by many of the labs. The researchers assumed that a significant proportion of the variability was due to either poor provisional OT specifications or poor adherence to Tex-248-F. Even when Tex-248-F was strictly followed, inherent variability associated with cyclic testing was evident. It was recommended that four or five specimens be tested and that the best three results reported (*7*).

Zhou and Scullion recommended a pass/fail criterion of >300 number of cycles to failure (N_f) for reflective cracking for Texas and >750 N_f for reflective cracking with the presence of a rich bottom layer (*6*). The >300 N_f criterion was intended only for reflective cracking, but the value worked well for predicting fatigue cracking as well (*8*). The OT is also used in New Jersey where 45% of the roads are asphalt overlays on Portland cement concrete (*9*). In 2013, the New Jersey DOT recommended >150 N_f for surface mixtures with high RAP contents and PG 64-22 binder and >175 N_f for surface mixtures with high RAP contents and PG 76-22 binder (*10*).

For this study, TX-OT testing was performed using an AMPT in accordance with Tex-248-F. TX-OT specimens were compacted in an SGC to a target height of 125 mm. Tex-248-F requires that one specimen per gyratory pill be used for testing; however, a sensitivity analysis performed by Walubita recommended using two specimens per SGC specimen to improve efficiency (*7*). Therefore, two specimens were cut from SGC specimens to the following dimensions: 150 mm x 76 mm x 38 mm. Four specimens per mixture were tested at 25°C in controlled displacement mode.

NCAT Modified Overlay Test

NCAT developed a modified version of the Overlay Test (*11*) that differs from the Tex-248-F method in three ways: 1) cycle frequency, 2) maximum opening displacement, and 3) the failure definition. The NCAT-OT method is performed with a one second cycle rather than ten seconds. The NCAT-OT method uses a 0.381 mm maximum displacement rather than 0.635 mm required in Tex-248-F. The smaller maximum displacement better simulates reflection cracking strain levels in flexible pavements. The NCAT-OT method uses the peak of the “normalized load x cycle” curve to identify failure as this approach closely matched when cracks propagated completely through the specimen as evident in video analysis. A comparison between the failure definitions of the two methods is shown in Figure 21. The cycle that corresponds to the maximum product of the load and the cycle is reported as the N_f for the test. The NCAT-OT method was shown to have lower variability than Tex-248-F (*11*).

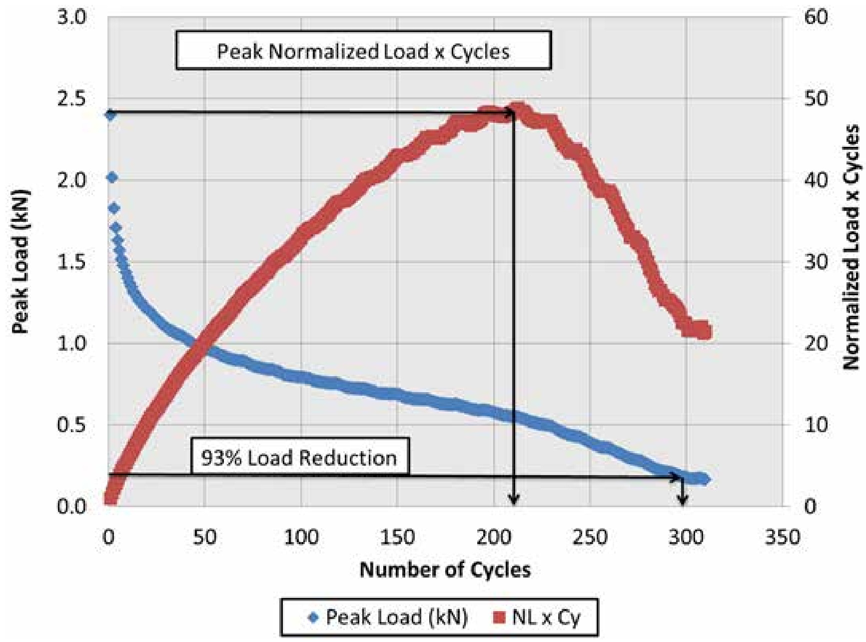

Figure 21 Determination of Failure Point for NCAT-OT vs. TX-OT

Semi-Circular Bend Test

The semi-circular bend test (SCB) has been used by numerous researchers to evaluate properties of asphalt mixtures for over a decade. Variations of the SCB test have been used to assess low-temperature fracture resistance, estimate tensile strength, and determine fatigue resistance (*12, 13, 14, 15, 16*). One approach to evaluate the cracking resistance of asphalt mixtures using notched specimens has been based on the critical strain energy release rate, or the J-integral (J_c). The J_c concept was first introduced to asphalt mixtures by Mull et al. to determine fracture characterization of mixtures with crumb rubber (*17*). The J-integral has been used extensively in Louisiana to determine fatigue resistance in asphalt mixtures (*16*). They reported that J_c correlated well with field cracking data despite an average coefficient of variation (COV) of 20% from 86 test mixtures. Wu et al. analyzed the sensitivity of J_c to a range of mixture variables and found significant effects from NMAS, binder type, and N_{design} (*18*).

The Louisiana DOTD SCB method uses SCB specimens with notch depths of 25.4, 31.8, and 38.1 mm (*19*). SCB specimens are loaded at a rate of 0.5 mm/min. As shown in Figure 22, the area under the load-displacement curve to the peak load is the strain energy to failure. The strain energy values are plotted against notch depths for each of the specimen replicates to create a linear regression line with a slope, dU/da. The slope of the regression line multiplied by -1 and divided by the average specimen thickness is the J-Integral (J_c) value. Larger J_c values indicate higher fracture resistance. Louisiana DOTD currently specifies a minimum 0.5 kJ/m^2 for asphalt mixtures (*19*).

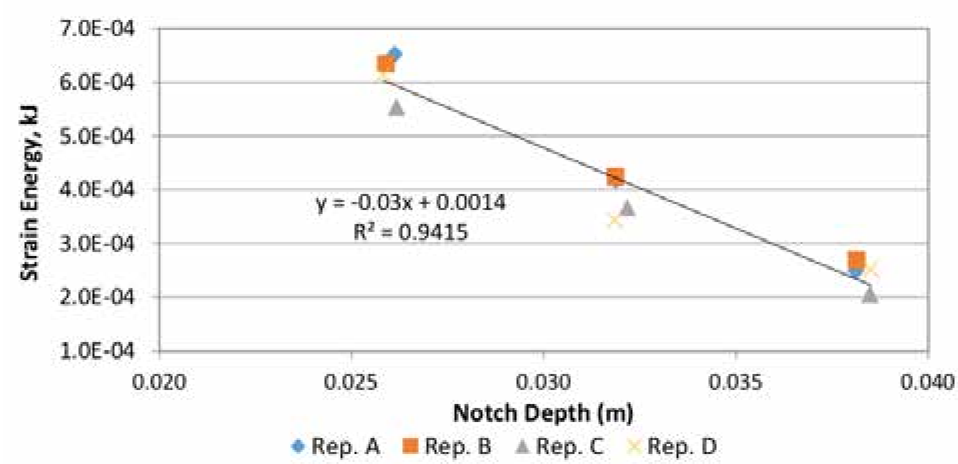

Figure 22 Typical Strain Energy Versus Notch Depth Results for SCB-LA DOTD Method

Specimens were conditioned in an environmental chamber at 25 ± 0.5°C for two hours prior to being tested for this study. Specimens were loaded monotonically using an AMPT in a three-point bending device at a rate of 0.5 mm/minute. From each specimen, a plot of load versus displacement (measured with an external LVDT) was collected. For each data file, numerical integration was used to determine the area under the load-displacement curve to the peak load. Twelve SCB specimens were tested for each mixture (four specimens at each notch depth). The slope from the least-squares linear regression of strain energy to failure versus notch depths was divided by the thickness to determine a single J_c value for each mix. Therefore, testing of twelve SCB specimens results in a single J_c value. Without replicates of J_c it is not possible to establish a true measure of the variability of this test.

Illinois Flexibility Index Test

Researchers at the University of Illinois developed another SCB test to identify mixtures with poor cracking resistance (*20*). The Illinois Flexibility Index Test (I-FIT) also uses a fracture energy approach to determine cracking resistance. However, the researchers recognized that fracture energy alone could not accurately distinguish between good and poor performing mixtures (*21*). Figure 23 shows a typical load-displacement data for the I-FIT test. The slope of the post-peak portion of the curve, *m*, was found to be sensitive to changes in testing conditions and material properties (*20*). Steep slopes indicate more brittle mixtures while more gradual slopes indicate a more ductile failure after crack initiation. I-FIT tests are conducted at 25°C and at a loading rate of 50 mm/min.

Currently, the Illinois Department of Transportation uses a Flexibility Index (FI) of 8 as the preliminary minimum criterion for all asphalt mixtures (*22*). Research is ongoing to further validate the criteria for different traffic levels, climates, and mixture types and applications (*21*). Typical COV values for FI range between 10-20% (*23*).

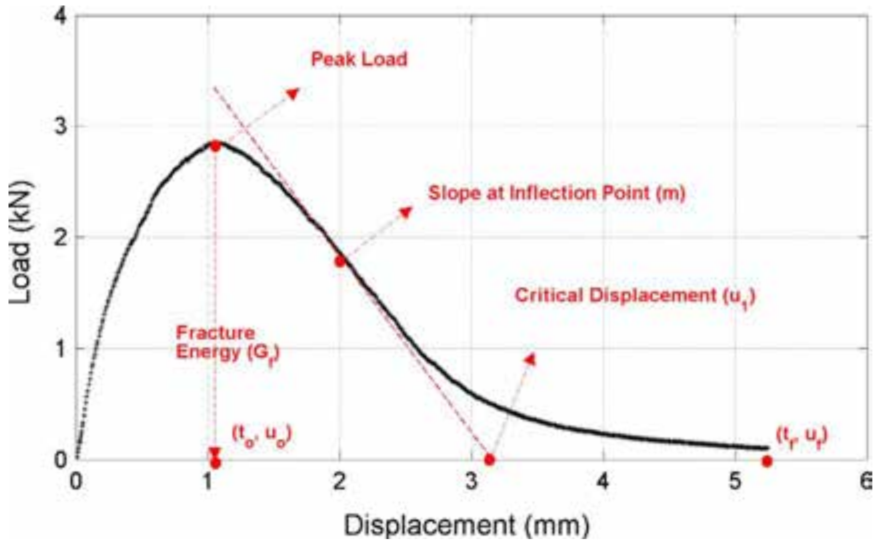

Figure 23 Typical Result of I-FIT Test (*21*)

In this study, the I-FIT was performed in accordance with a draft specification using a standalone I-FIT testing device (*22*). SCB specimens were prepared to the target air void level ± 0.5 percent after trimming. A minimum of six replicates were prepared for this study. Each specimen was trimmed from a larger 160-mm tall by 150-mm diameter gyratory specimen. Four replicates were obtained per specimen. A notch was cut 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 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 displacements were recorded at a rate of 50 Hz by the testing system.

The load and displacement data for each specimen were used to calculate the Fracture Energy (FE) and the Flexibility Index. The FE is the area under the load-displacement curve normalized for the specimen dimensions. FE is calculated by integrating the area under the raw load displacement curve and dividing it by the ligament area (the area of the semi-circular specimen through which the crack will propagate). The slope of the post-peak portion of the curve is determined at the inflection point after the peak load. The Flexibility Index was then calculated by dividing the FE by the post-peak slope and then multiplying that quotient by a scaling factor. In general, a higher FE 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 analyses for this project were performed using a tool developed by the Illinois Center for Transportation (ICT) at the University of Illinois at Urbana-Champaign (UIUC).

IDEAL-CT

The IDEAL cracking test (IDEAL-CT) is a test that has been recently developed by researchers at the Texas Transportation Institute (TTI) (*24*). The goal behind the development of this test was to develop a practical cracking test that could be routinely used in asphalt mix design as well as for QC/QA (*24*). The test itself is conducted on gyratory specimens that are compacted to a target height and air void level. A major benefit of this test is that no additional cutting or notching is required. Specimens are temperature conditioned for two hours at 25°C and then tested using an indirect tension load frame. Testing was performed using a stand-alone servo-hydraulic machine capable of sampling load and displacement data at a rapid rate (40 Hz). Specimens are loaded monotonically at a rate of 50 mm/min in load line displacement (LLD) until failure. The plot of load versus LLD is then analyzed to determine the CT Index. The type of analysis used to calculate the CT Index is a post-peak analysis similar to that used to calculate the I-FIT Flexibility Index. The major difference is that the slope of the post-peak curve where the load is reduced to 75% of its peak value is used rather than the exact inflection point. Figure 24 illustrates how this slope is determined from the load-displacement curve. For this study, a minimum of six replicates were prepared for each mixture.

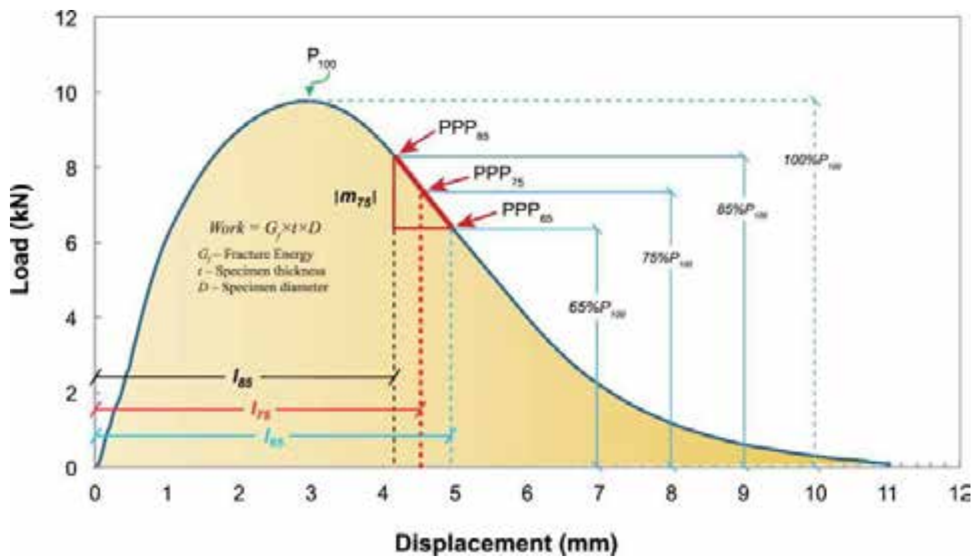

Figure 24 Calculation of Post-Peak Slope for IDEAL-CT (*24*)

Summary of Cracking Tests

Table 6 summarizes the load type, temperature, result, and typical variability of the five cracking tests. As noted in previous sections, it is not possible to determine a measure of variability for the Energy Ratio or the SCB-J_c test because there are no replicates for the calculated ER or J_c values. The lack of a measure of test result variability is a significant disadvantage of these methods because it precludes standardized statistical comparisons of results and would make it extremely challenging to establish precision and bias information that are critical to a test's use in specifications for mix design or quality assurance. Of the remaining three tests conducted in this study, the I-FIT test has the lowest variability. The Texas Overlay Test has high variability, which has been one of its primary disadvantages.

Table 6 Summary of Properties of Cracking Tests in Experiment

Cracking Test	Loading Type	Test Temp.	Result	Typical Variability
Energy Ratio	Cyclic haversine waveform (Resilient Modulus) Monotonic vertical axis loading (Tensile Strength) Static loading (Creep Compliance)	10°C	ER (ratio of $DCSE_{HMA}$ to $DCSE_{MIN}$)	N/A
Texas Overlay Test	Displacement-controlled cyclic loading to 0.635 mm.	25°C	N_f (number of cycles to 93% load reduction)	30-50% COV
NCAT Overlay Test	Displacement-controlled cyclic loading to 0.381 mm.	25°C	N_f (number of cycles to peak of normalized load x cycle curve)	20-30% COV (limited number of studies)
SCB-J_c	Monotonic three-point bending at 0.5 mm/min.	25°C	J_c (Critical J-Integral)	N/A
I-FIT	Monotonic three-point bending at 50 mm/min.	25°C	FI (Flexibility Index)	10-20 % COV (limited number of studies)
IDEAL-CT	Monotonic Indirect Tension at 50 mm/min	25°C	CT Index	Most less than 20% (*24*)

Table 7 includes NCAT's estimates of equipment costs, preparation and testing time, and complexity of the five cracking tests. All of these tests require saws for preparing specimens from SGC specimens. It should be noted that although some of the tests can be performed on alternative equipment, the values listed in Table 7 are representative of the equipment most often used. As previously noted, NCAT used an AMPT for the OT tests and the SCB-J_c tests for this study. Current prices for AMPTs start at approximately $80,000. The ER tests are conducted at 10°C, which requires an environmental chamber and a more sophisticated servo-hydraulic test system to conduct repeated load, static load, and monotonic testing. Sample preparation and testing times are based on starting with reheating plant mix in a bucket and ending with the final test result. Specimens for each of these tests are typically cut from larger SGC specimens. The ER and both OT methods require gluing; I-FIT and SCB require notching of the specimens. Complexity refers to data analysis and interpretation and is rated on a scale of 1 to 5 by NCAT researchers with 1 being very simple and 5 being very complex. Data analysis for each test is somewhat automated by the test controls and/or worksheets, but some test results are more readily interpreted by engineers and technicians in the current workforce.

Table 7 Comparison of Cracking Test Equipment Cost, Testing Time, and Complexity

Cracking Test	Approximate Equipment Cost (Thousand $)	Sample Preparation, Testing, and Analysis Time	Number of Specimens per Test	Complexity
ER	150-250	4 – 5 Days	3	4.8
TX-OT	55-85	3 – 4 Days	4	2.2
NCAT-OT	55-85	3 – 4 Days	4	2.4
SCB-J_c	10-30	2 – 3 Days	12	3.4
I-FIT	10-20	2 – 3 Days	6	2.4
IDEAL-CT	10-20	1 – 2 Days	3	1.2

2.6 Statistical Results and Analysis

All test results were checked for outliers in accordance to ASTM E178-08 except the Energy Ratio. All results that failed ASTM E178-08 at a significance level of 0.10 were eliminated. Results of the OT methods and the I-FIT were analyzed using an ANOVA with a Games-Howell post hoc test. Statistical groupings were determined using the Games-Howell method, which does not require that samples have equal variance. Letters are used to designate statistical groups in the results section. Mixtures that share a letter were not 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.

Results of the SCB tests following the Louisiana method were also analyzed by alternate method. Estimates of the error of the dU/da slopes were determined from the regression analyses. The standard deviation of the dU/da slope was estimated by dividing the estimate of the total model deviation (se) by the sum of squared differences between the x values (Sxx), in this case, notch lengths, in each model. This provided an isolated estimate for the error in the model pertaining only to the slope (*25*). A more detailed explanation of this analysis is available elsewhere (*26*). The slope of the strain energy vs. notch depth line was the only variable in J_c calculation because specimen thicknesses were constant. Therefore, the standard deviation of dU/da was multiplied by the same constant as the slope to determine an estimation of the variability in the J_c results. 95% confidence intervals were calculated for each mixture. Mixtures were considered statistically different if the intervals did not overlap.

Energy Ratio Results

The Energy Ratio intermediate properties and ER values are summarized in Table 8. Three mixtures (N2, N8, and S6) had mean $DCSE_{HMA}$ results below the recommended range of 0.75 – 2.5 kJ/m^3, indicating that these mixtures are susceptible to top-down cracking. As noted previously, at the end of the cycle, N2 had about 6% of the lane area cracked and N8 had about 17% of the lane area cracked; no cracking has been observed to date in S6. All mixtures easily passed the minimum ER criterion of 1.95. The other two sections that have cracking (N1 and N5) had acceptable $DSCE_{HMA}$ and ER results. Therefore, there are several inconsistencies between the ER results and field performance. It can also be seen that the ER results for S13 are the lowest of all the mixtures. This seems to be another inconsistency since this mix has a much higher binder content than the other mixtures and is expected to be much more resistant to top-down cracking than the other mixtures. However, recall that the ER criteria were based on field cores from pavements that were at least 10 years old (*3*) whereas the results analyzed here were from tests on reheated plant mix. Although the ER values and intermediate properties fail to consistently match the performance to date, additional tests on aged mix samples will be conducted and field performance will continue to be monitored through another cycle of trafficking.

Table 8 Results of Energy Ratio Tests on Reheated Plant Mix Samples

Test Section and Mixture Description	Resilient Modulus (GPa)	Creep Compliance Rate	IDT Fracture Energy (kJ/m³)	$DCSE_{HMA}$ (kJ/m³)	Energy Ratio
	Trimmed Means				
N1: Control	9.94	3.79E-09	4.8	0.82	5.5
N2: Control, Higher Density	12.41	1.98E-09	3.9	0.48	7.4
N5: Control, Low Dens. & AC	7.93	4.31E-09	3.4	0.89	3.6
N8: Control+5% RAS	12.75	4.98E-10	1.8	0.12	12.8
S5: 35% RAP, PG 58-28	7.38	3.46E-09	6.0	0.78	7.4
S6: Control, HiMA Binder	7.28	2.44E-09	5.4	0.56	9.2
S13: Gap-gr., Asphalt-rubber	7.40	5.17E-09	2.7	1.13	2.2

Texas Overlay Results

The Texas Overlay results are summarized in Table 9 from highest to lowest cycles to failure. It can be seen that TX-OT results for mixture S13 with asphalt-rubber greatly exceeded the other six mixtures. S13 is obviously different than the other six mixtures from a practical and statistical viewpoint. All of the other mixes would fail Texas DOT and New Jersey DOT criteria. Mixture N8 containing RAP and RAS failed at two cycles for each of the four replicates. This is common for mixes containing RAS and often occurs with high RAP content mixtures. The result of mix S6 with polymer-modified asphalt (HiMA) was, however, very surprising. This mixture had a mean N_f that was lower than even the mix with low density and low asphalt content.

The Games-Howell statistical groupings of the mixtures are also presented in Table 9. The average COV for these seven sets of data was 45 percent. This value is in agreement with literature and past experience from other research studies using the OT conducted at NCAT. The mixture from S13 had the lowest COV (excluding N8), although its standard deviation was eight times greater than the other mixtures. The high variability of the TX-OT could create problems in lab-to-lab comparisons of results such as mix design verifications and quality assurance testing. The high variability also greatly diminishes the power of the test to statistically distinguish mixtures with major differences in composition as evident with six of the seven mixtures having the same Games-Howell groupings.

Table 9 Results of Texas Overlay Tests on Reheated Plant Mix Samples

Test Section and Mixture Description	Replicates	Average N_f	Standard Deviation	COV (%)	Statistical Groups
S13: Gap-gr., Asphalt-rubber	4	1725	360	21	A
S5: 35% RAP, PG 58-28	4	61	39	64	B
N2: Control, Higher Density	4	59	46	78	B
N1: Control	4	25	19	79	B
N5: Control, Low Dens. & AC	4	17	4	25	B
S6: Control, HiMA Binder	3	13	6	48	B
N8: Control+5% RAS	4	2	0	--	B

NCAT Overlay Test Results

Table 10 shows the results of the NCAT modified overlay test. NCAT has not recommended criteria for this method. As with the TX-OT results, mixture S13 with asphalt-rubber was superior by a large margin and mixture N8 with RAP/RAS had the worst results. The results of the Texas method and NCAT method produced very similar rankings. The only differences were the order of mixtures N5 with low density and S6 with polymer-modified asphalt. Again, these results followed expected trends except for S6, which had lower than expected results.

The variability of the NCAT-OT method compared to the TX-OT method is generally lower. The average COV for these seven mixes was approximately 35 percent. However, it should be noted that four of the seven mixtures had one specimen that failed the ASTM E178-08 outlier check. The lower COV's of the NCAT-OT method versus the TX-OT method might be misleading. Further research is needed to validate the NCAT-OT and to calibrate the results to field performance.

Table 10 Results of the NCAT-OT Tests on Reheated Plant Mix Samples

Test Section and Mixture Description	Replicates	Average N_f	Standard Deviation	COV (%)	Statistical Groups
S13: Gap-gr., Asphalt-rubber	3	3054	951	31	A B
S5: 35% RAP, PG 58-28	3	773	235	30	A B
N2: Control, Higher Density	4	697	330	47	A B
N1: Control	3	516	146	28	A B
S6: Control, HiMA Binder	4	411	278	68	A B
N5: Control, Low Dens. & AC	4	189	189	27	A B
N8: Control+5% RAS	3	12	2	17	B

Semi-circular Bend Test (Louisiana Method) Results

Table 11 shows results of the average strain energy to failure of each notch depth for every mixture along with the corresponding COV. With one exception (S13 – 31.8 mm notch), all were below 20% and the overall average COV was approximately 10%. This COV was consistent with values found in literature. However, the strain energy to failure is not the test result used as a mix criteria. The strain energy release rate, J_c, is the test parameter for the Louisiana version of the SCB test.

Table 11 SCB Strain Energy Results for Reheated Plant Mix Samples

Test Section and Mixture Description	Notch Length (mm)	Avg. U (kN-mm)	Replicates	COV (%)
N1: Control	25.4	0.53	4	10%
	31.8	0.40	4	13%
	38.1	0.26	3	8%
N2: Control, Higher Density	25.4	0.76	3	2%
	31.8	0.49	3	5%
	38.1	0.32	4	9%
N5: Control, Low Dens. & AC	25.4	0.46	4	15%
	31.8	0.28	4	11%
	38.1	0.22	4	19%
N8: Control+5% RAS	25.4	0.47	4	13%
	31.8	0.28	3	1%
	38.1	0.19	4	7%
S5: 35% RAP, PG 58-28	25.4	0.47	4	12%
	31.8	0.33	4	8%
	38.1	0.23	4	17%
S6: Control, HiMA Binder	25.4	0.49	4	5%
	31.8	0.34	4	9%
	38.1	0.23	4	6%
S13: Gap-gr., Asphalt-rubber	25.4	0.81	3	3%
	31.8	0.61	4	21%
	38.1	0.46	4	12%

Table 12 lists the J_c results, confidence intervals for dU/da, and statistical groupings based on the analysis approach described in Section 2.6. The mixture with the highest J_c value was N2 with high density. S13 with GTR was the second highest result and was just above the Louisiana criterion. However, recall that the SCB-J_c criterion were based on long-term oven aged mixtures. The two mixtures with the most cracking observed in the field, N1 and N8, had J_c results below the Louisiana criteria, but so did S6 and S5, which have had no cracking. In fact, S5 was one of the mixtures with the lowest J_c result. The statistical analysis also showed that the J_c results for this data set did not clearly distinguish between most of the mixtures. The inability of the test to statistically distinguish between many of the mixtures with very different properties is a cause for concern.

Table 12 J_c Results and Statistical Groupings for Reheated Plant Mix Samples

Test Section and Mixture Description	J_c (kJ/m^2)	dU/da	Std. Dev. of dU/da	95% Confidence Interval for dU/da	Statistical Groups
N2: Control, Higher Density	0.61	-0.0348	0.0021	-0.0300, -0.0395	A
S13: Gap-gr., asphalt-rubber	0.51	-0.0293	0.0050	-0.0179, -0.0407	A B
N8: Control+5% RAS	0.39	-0.022	0.0022	-0.0170, -0.0269	B
S6: Control, HiMA binder	0.37	-0.021	0.0012	-0.0182, -0.0238	B
N1: Control	0.36	-0.021	0.0029	-0.0142, -0.0272	B
N5: Control, Low Dens. & AC	0.34	-0.019	0.0029	-0.0128, -0.0258	B
S5: 35% RAP, PG 58-28	0.34	-0.019	0.0028	-0.0130, -0.0254	B

I-FIT

Table 13 shows the results of the I-FIT sorted from highest to lowest FI results. For each mixture, a minimum of six replicates were tested before the outlier analysis was performed. The trend of S13 having the best result and N8 the worst result in this study continued for the I-FIT test. These two mixes also had the highest and lowest variance, respectively. Counter to the expected trend, the low density mixture, N5, had a higher mean FI than the high density mixture, N2. Table 13 lists the FI values of each mix and the corresponding variability and Games-Howell statistical grouping. The FI results have much lower variability than the OT methods, which bodes well for its potential usage as a specification criterion. Again, this trend was largely expected for the monotonic tests in comparison with the cyclic OT. The average COV for these seven sets of I-FIT data was approximately 18 percent. However, it should be noted that every mixture in this experiment except S13 would have failed the preliminary FI pass/fail criterion of 8 set forth by IDOT. This may suggest the need of development and validation of regional criteria for this test that will help identify mixes that will perform well in the field.

Table 13 IFIT Results and Statistical Analysis

Test Section and Mixture Description	Replicates	Avg. FI	Std. Dev.	COV (%)	Statistical Groups
S13: Gap-gr., asphalt-rubber	6	10.4	4.4	42	A B C D
S5: 35% RAP, PG 58-28	6	6.3	0.6	10	A
S6: Control, HiMA binder	5	4.5	0.3	6	B
N1: Control	7	3.6	0.3	8	C
N5: Control, Low Dens. & AC	6	2.7	0.8	29	C D E
N2: Control, Higher Density	5	1.9	0.2	13	E
N8: Control+5% RAS	9	0.4	0.1	18	F

IDEAL-CT

Table 14 shows the IDEAL-CT test results sorted from highest to lowest CT Index. For each mixture, a minimum of six replicates were tested before the outlier analysis was performed. For the re-heated plant-produced mix, the IDEAL-CT ranked the mixes in the same way as the I-FIT with the exception of S5 and S6 having switched places. Again, S13 was in the highest statistical grouping and N8 was in the lowest statistical grouping. The density sections also showed similar trends to the I-FIT, with N5 outperforming N2 with respect to CT Index. While this runs counter to the expected trend (higher density having better cracking resistance), it is not surprising that the I-FIT and IDEAL-CT rank mixes similarly due to both having a post-peak style analysis methodology. The average CV for these seven mixes for the re-heated plant-produced mix was approximately 16 percent, which was very comparable with that of the I-FIT on the same set of mixes.

Table 14 Summary of IDEAL-CT Results and Statistical Analysis

Test Section and Mixture Description	Replicates	Avg. FI	Std. Dev.	COV (%)	Statistical Groups
S13: Gap-gr., asphalt-rubber	3	275.5	60.4	22	A
S6: Control, HiMA binder	3	34.7	2.4	7	B
S5: 35% RAP, PG 58-28	3	30.5	4.3	14	B
N1: Control	3	28.2	7.0	25	B
N5: Control, Low Dens. & AC	3	26.4	4.6	17	B
N2: Control, Higher Density	3	13.9	2.5	18	C
N8: Control+5% RAS	3	5.3	0.6	10	D

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 14. 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 the seven mixtures in this study. This shows that Energy Ratio is highly correlated to Creep Rate, the Texas Overlay Test is highly correlated to the NCAT Modified OT, the Flexibility Index is strongly correlated with both the TX-OT and the NCAT Modified OT, and the IDEAL-CT is strongly correlated with both OT methods as well as the I-FIT Flexibility Index. Other studies at NCAT have also shown strong correlations between I-FIT results and Overlay Tester results. Interestingly, the SCB-Louisiana method and the Energy Ratio results were not strongly correlated with any of the other cracking tests.

Table 14 Pearson Correlation Coefficients Among Cracking Test Results

	Resilient Modulus	Creep Rate	$DCSE_{HMA}$	Energy Ratio	TX-OT	NCAT-OT	SCB (Louisiana)	I-FIT
Creep Rate	-0.742	1.000						
$DCSE_{HMA}$	-0.519	0.212	1.000					
Energy Ratio	0.563	-0.956	-0.071	1.000				
TX-OT	-0.347	0.590	-0.349	-0.585	1.000			
NCAT-OT	-0.390	0.635	-0.166	-0.627	0.973	1.000		
SCB (Louisiana)	0.415	-0.062	-0.333	-0.158	0.426	0.480	1.000	
IFIT	-0.732	0.760	0.207	-0.641	0.830	0.891	0.117	1.000
IDEAL-CT	-0.461	0.656	-0.248	-0.624	0.991	0.973	0.343	0.887

2.7 Summary of Preliminary Observations

After two years of trafficking with ten million accumulated ESALs, only one of the seven test sections in the top-down Cracking Group experiment on the NCAT Test Track had a substantial amount of cracking. Section N8 containing 20% RAP and 5% RAS with a PG 67-22 binder has cracking in nearly seventeen percent of the lane area. Limited coring showed that the cracking in N8 was confined to the surface layer with no evidence of debonding between layers. Structural analysis based on backcalculated asphalt concrete moduli also indicated that the wheel path cracking in this section has resulted in damage to the pavement structure.

The other three test sections that had evidence of very low-severity cracking on their surfaces were also cored at the end of the cycle. There was no visible evidence that the observed hair-line surface cracking in N1, N2, and N5 had propagated into the surface layer. Structural analysis provided no indication of damage to these pavement structures.

More time and traffic are necessary to increase the amount and severity of cracking in several test sections in order to accomplish the experiment's objectives. Sponsors have agreed to support the continued traffic and monitoring of the experiment in the 2018 cycle of the Test Track.

The laboratory test results presented in this interim report represent only about one fourth of the total testing plan associated with this study. The results presented herein were for plant mix samples that were reheated just enough to fabricate the specimens. Additional work is underway to test plant mix samples that have been laboratory aged to simulate approximately four years of field aging in Auburn, Alabama as well as testing of laboratory prepared mixtures that are aged to represent mix production aging and four years of in-service aging of surface mixtures. The laboratory aging protocol used for this study to simulate in-service aging is eight hours at 135°C in a loose mix state. NCAT refers to this protocol as "critically aged" and it represents 70,000 cumulative degree days (CDD) of in-situ aging, which is when top down cracking typically occurs in surface layers (*26*).

Based on the results of this project, the Energy Ratio method has several significant shortcomings. In its current procedure, it is not possible to properly characterize the variability of the ER parameter. The equipment cost and test complexity also render it impractical for routine use. The intermediate parameter, $DSCE_{HMA}$, did indicate that the mixture from Section N8 (Control+5% RAS) was susceptible to brittle failure. However, the mixture from S6 (Control, HiMA) also failed the $DSCE_{HMA}$ criteria and this mixture has had no signs of cracking. Also, the mixture with the lowest Energy Ratio was S13 (15% RAP, gap-graded mix with asphalt-rubber), but this section has no signs of cracking and seems likely to have excellent long-term cracking performance based on its much higher asphalt content. Although the field results are limited, and results of aged mixtures are yet to be reported, the Energy Ratio method does not seem suitable for specification use in routine practice.

The OT results (both the Texas method and the NCAT-modified method) ranked the mixtures largely in accordance with their anticipated level of field cracking. Results of the two test methods were highly correlated. Both methods predicted that N8 would be the most susceptible to cracking, as was confirmed in the field. Both methods also indicate that S8 will have much superior cracking resistance than all of the other mixtures. However, one of the disadvantages of the OT methods is their relatively high variability. For the results of this study, the pooled coefficient of variation for the Texas method was approximately 45% and for the NCAT-modified test it was approximately 35%. These are similar to COVs reported in the literature for these methods. This diminishes the power of the test to distinguish mixtures with significant differences in composition. Furthermore, higher equipment costs and longer time to complete the tests are substantial disadvantages. On the other hand, both OT tests appear to appropriately rank the mixtures with different density levels. The mixture with the higher density level (N2) had higher cycles to failure than the control mixture (N1).

The SCB (Louisiana method) and J_c criteria was able to able to identify the mixture from N8 as susceptible to cracking, but it also indicated very similar results for four of the other mixtures, two of which (S5 and S6) have no signs of cracking to date. More field performance data are needed to judge the validity of the J_c criteria. Despite the relatively large number of specimens needed to obtain the J_c parameter, the test can be completed within a few days. As noted previously, a disadvantage of the SCB method is the inability to assess variability of the J_c parameter with traditional statistical analyses.

The I-FIT method yielded a relatively large spread of Flexibility Index results for the seven mixtures. This kind of statistical spread in results for different mixtures would allow users to better assess how to improve mix designs and adjust field mixtures. The FI results indicated that the mixture from N8 was the most susceptible to cracking, as was confirmed on the Test Track. The IDEAL-CT data showed the same trends as the I-FIT data in most respects. More field performance data are needed to better judge the validity of the test and potentially set criteria for specification use. One concern with both the I-FIT test and the IDEAL-CT is the impact of specimen density. Counter to the expected outcome, higher density specimens have lower FI and CT Index results than lower density specimens. The results of the I-FIT, IDEAL-CT and the two OT methods were strongly correlated. The I-FIT and IDEAL-CT have the lowest equipment cost and fastest testing time of the six cracking tests in the experiment, but the IDEAL-CT offers faster specimen fabrication than the I-FIT since no specimen saw cutting is required.

2.8 References

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CHAPTER 3 ALABAMA DEPARTMENT OF TRANSPORTATION EVALUATION OF OPEN-GRADED FRICTION COURSE MIXTURES

3.1 Background

The Alabama Department of Transportation (ALDOT) has used open-graded friction course (OGFC) mixtures for many years. However, ALDOT has limited its use due to premature raveling issues occurring after approximately six or seven years in service (or between 10 to 20 million equivalent single axle loads). A typical OGFC mix in Alabama consists of a 12.5-mm nominal maximum aggregate size (NMAS), 0.3 percent cellulose fiber, and 6 percent PG 76-22 asphalt modified with styrene-butadiene-styrene (SBS).

In 2012, ALDOT sponsored three test sections (E9A, E9B, and E10) on the NCAT Test Track to evaluate potential changes in its mix design procedure to improve the durability of OGFC mixtures in Alabama. The following changes for an OGFC mixture were evaluated.

1. A finer (9.5-mm NMAS) gradation (instead of a typical 12.5-mm NMAS gradation) was designed with a cellulose fiber and SBS-modified asphalt binder for the OGFC mixture in Section E9A.
2. A synthetic fiber (instead of a cellulose fiber) was utilized in the OGFC mix design for Section E9B with a typical 12.5-mm NMAS gradation and SBS-modified asphalt binder.
3. A ground tire rubber (GTR) modified binder was used in place of SBS-modified binder for the OGFC mixture in Section E10 with a typical 12.5-mm NMAS gradation but without cellulose fiber.

Sections E9A, E9B, and E10 were milled and inlaid with the OGFC mixtures in 2012. Except for the changes made in the mix designs, the sections were paved following common construction practices for OGFC mixtures in Alabama. While a state-approved OGFC mix design was referenced when designing the three mixtures in Sections E9A, E9B, and E10, it was not paved on the NCAT Test Track for this experiment as previous in-service pavements on Interstate 85, which are a few miles away from the Test Track, were considered the control for this experiment. After these mixtures were paved in 2012, 10 million equivalent single axle loads (ESALs) were applied on these test sections in the 2012 research cycle (from 2012 through 2015). Field performance data of these test sections in the 2012 research cycle were presented in a previous report (*1*) and are briefly discussed below.

At the end of the 2012 research cycle, the three sections showed no cracking and less than 5 mm of rutting in all sections. Other surface characteristics of these sections are shown in Figure 1. The mean texture depth (MTD) of the 9.5-mm OGFC mixture in Section E9A was approximately the same as that of the 12.5-mm OGFC mixtures in Sections E9B and E10. The 9.5-mm OGFC mix in Section E9A showed slightly higher international roughness index (IRI) measurements in the first part of the test cycle and increased more in the summer of 2014, whereas roughness in Sections E9B and E10 did not change over the research cycle (*1*).

Since the three OGFC mixtures still performed well and showed no signs of raveling or cracking at the end of the 2012 research cycle, ALDOT decided to continue trafficking these sections in the 2015 research cycle (from 2015 through 2017) to evaluate the long-term durability of the three OGFC mixtures.

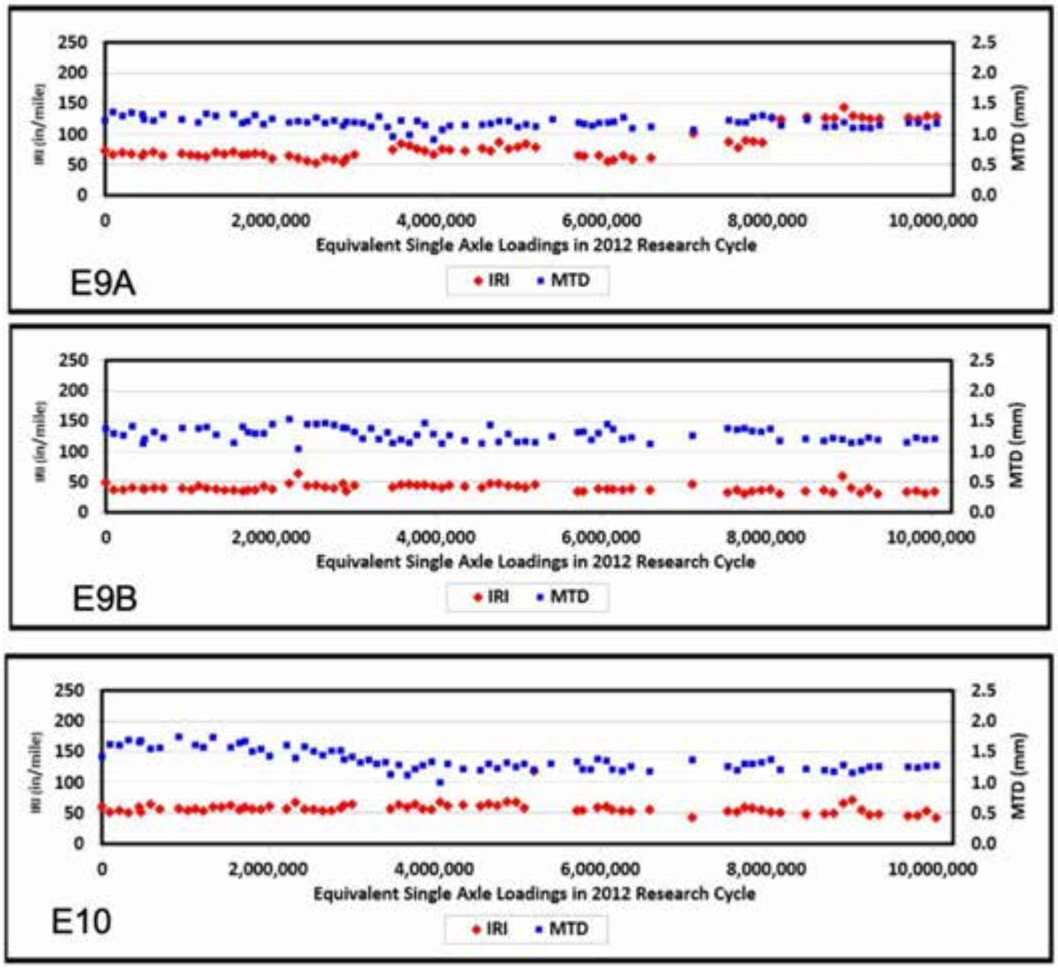

Figure 1 Smoothness and Mid Depth Texture Comparison (*1*)

This chapter provides key findings of the experiment (with three test sections) to evaluate the potential changes in ALDOT's OGFC mix design procedure to improve the durability of OGFC mixtures in the field. The findings are drawn based on results from both 2012 and 2015 research cycles of the NCAT Test Track (from 2012 through 2017).

3.2 Mix Design and Performance Testing

Prior to the construction of the three test sections at the NCAT Test Track, five OGFC mixtures were designed in 2012 based on a 12.5-mm OGFC mix design previously approved by ALDOT. The approved OGFC mix was designed based on ALDOT's OGFC mix design procedure and consisted of 91 percent #78 granite aggregate, 8 percent M10 granite aggregate, 1 percent baghouse fines, 0.3 percent cellulose fiber, and 6 percent PG 76-22 asphalt modified with polymer. The approved mix design did not include an antistrip agent, but the tensile strength ratio determined in accordance with AASHTO T283 without a freeze-thaw cycle was 0.87, which is higher than the commonly used tensile strength ratio criterion of 0.80 in Alabama. A description of each OGFC mix design follows.

- The first OGFC mixture was a 12.5-mm mix with cellulose fiber deigned similar to the state approved mix design except that this mix design had a lower air void content than a typical OGFC mixture approved by the state.
- The second OGFC mixture, which was later selected for Section E9A, was designed with a 9.5-mm NMAS gradation instead of a 12.5-mm NMAS gradation. This mix design consisted of 44 percent #78 granite aggregate, 50 percent #89 granite aggregate, 6 percent M10 granite, 0.3 percent cellulose fiber, and 6 percent PG 76-22 asphalt modified with polymer.
- The third OGFC mixture, which was later selected for Section E9B, was designed with a 12.5-mm NMAS gradation similar to the one utilized in the state approved OGFC mix design. It had 91 percent #7 granite aggregate, 8 percent M10 granite aggregate, 1 percent baghouse fines, and 6 percent PG 76-22 asphalt modified with polymer. However, this mix design had 0.5 percent synthetic fiber instead of the cellulose fiber used in the state approved mix design to prevent draindown of the thick asphalt binder film from aggregate particles.
- The fourth OGFC mixture was designed similar to the third OGFC mix, except that this mix had both 0.3 percent synthetic fiber and 0.3 percent cellulose fiber.
- The same 12.5-mm NMAS gradation was used in the fifth OGFC mix design, which was later selected for Section E10. However, the mix design for Section E10 had 6.5 percent asphalt modified with GTR, which consisted of 5.8 percent base asphalt modified by adding 12 percent GTR (by weight of asphalt binder). The GTR was a minus No. 30 mesh size, and the base asphalt was a PG 67-22. No fiber was added to the mix in order to determine whether GTR alone could prevent drain-down and provide resistance to raveling.

Table 1 summarizes the design gradations for the 9.5-mm and 12.5-mm OGFC mixtures. All the binders were pre-blended with an antistrip agent at a dosage of 0.5 percent by weight of the base binder. During the mix design, samples were prepared in the NCAT laboratory and compacted to 50 gyrations to measure air void content, Cantabro stone loss, and splitting tensile strengths.

Table 1 Design Aggregate Gradations for OGFC Mixtures

Sieve Size (mm)	Sieve Size (in)	Percent Passing	
		9.5-mm OGFC (E9A)	12.5-mm OGFC (E9B and E10)
19	¾	100.0	100.0
12.5	½	98.5	95.7
9.5	3/8	87.2	56.1
4.75	#4	32.4	15.7
2.36	#8	9.8	9.5
1.18	#16	5.7	7.1
0.6	#30	4.2	5.7
0.3	#50	3.1	4.5
0.15	#100	2.1	3.4
0.075	0.075	1.4	2.6

Figure 2 shows the sample air voids and Cantabro loss results for the five OGFC mixtures evaluated during mix design. These two mixture properties were found to be important for the performance of OGFC mixtures in the field. An OGFC mixture with higher air voids is expected to drain water off the pavement surface quicker, and the minimum air void content for OGFC mixtures was estimated to be approximately 15 percent in previous research cycles of the NCAT Test Track. In addition, an OGFC mixture with lower Cantabro stone loss results is anticipated to have better resistance to raveling, and the maximum Cantabro loss for OGFC mixtures compacted to 50 gyrations is 15 percent. A further discussion of the results shown in Figure 2 follows.

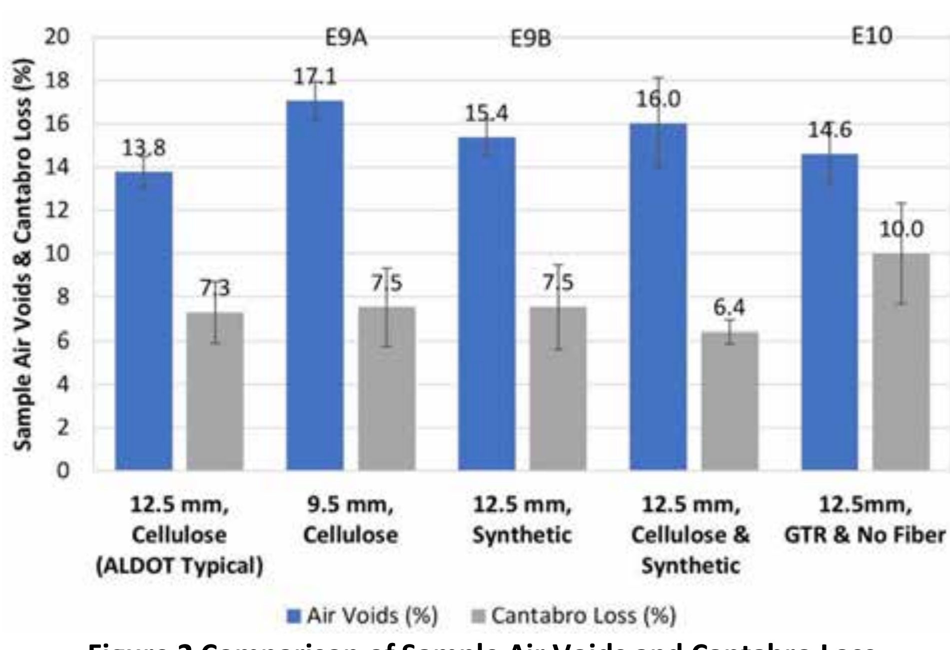

Figure 2 Comparison of Sample Air Voids and Cantabro Loss

- Except for the 9.5-mm mix, the other mixes shown in Figure 2 used the same 12.5-mm design gradation. Also, except for the GTR mix, the other mixes had a polymer modified PG 76-22 binder and cellulose and/or synthetic fiber. The three mixtures selected for evaluation at the NCAT Test Track included the 9.5-mm mix with cellulose fiber (Section E9A), 12.5-mm mix with synthetic fiber (Section E9B), and 12.5-mm mix with GTR modified binder (Section E10). The air voids determined for the three mixtures selected were close to or above the minimum air void content of 15 percent, and the Cantabro loss results were all lower than the maximum Cantabro loss threshold of 15 percent.

- The 12.5-mm mix with cellulose fiber (the first mix in Figure 2) was similar to the ALDOT-approved mix design with known field performance from previous pavements on Interstate 85, which are a few miles away from the NCAT Test Track. Compared to the three OGFC mixes selected for the experiment, this mix had lower air voids and a similar Cantabro stone loss. The 12.5-mm mix design with both cellulose and synthetic fibers (the fourth mix in Figure 2) was not selected because it had air voids and Cantabro loss results similar to the mix that had only the synthetic fiber. Thus, adding cellulose fiber into an OGFC mixture with synthetic fiber did not provide added benefits.

Figure 3 compares splitting tensile strength test results obtained during mix design for four OGFC mixes. While the ALDOT procedure for OGFC mix design requires the tensile strength ratio to be determined in accordance with AASHTO T 283 without a freeze-thaw cycle, the splitting tensile strength test (results shown in Figure 3) was conducted with a freeze-thaw cycle as the conditioned splitting tensile strength determined with a freeze-thaw cycle was found to be a good indicator of OGFC mixture performance in the field. In addition, a minimum splitting tensile strength of 50 psi was proposed for a performance-based mix design in a previous study (*2*). This threshold is shown as the dash line in Figure 3.

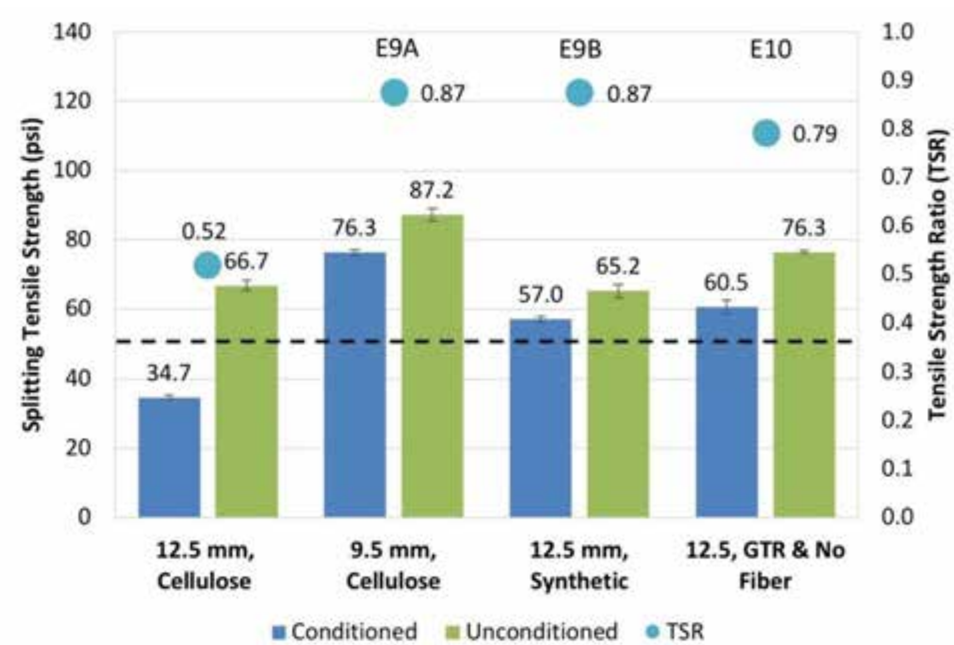

Figure 3 Comparison of Tensile Strength Test Results for OGFC Mixtures

In Figure 3, the 9.5-mm mixture with cellulose fiber provided the highest tensile strengths. Among the 12.5-mm OGFC mixes, the GTR mix without fiber had the highest tensile strengths but marginally failed the commonly accepted tensile strength ratio requirement of 0.8, while the 12.5-mm mix with cellulose fiber failed this requirement significantly. In addition, the three selected OGFC mixtures passed the conditioned splitting tensile strength threshold of 50 psi while the 12.5-mm mix with cellulose fiber failed this proposed requirement. The moisture conditioning and freeze-thaw cycle in AASHTO T 283 had a significant effect on the conditioned splitting tensile strength of this mix.

3.3 Field Performance

All the mixes were placed 0.75 inches thick on the NCAT Test Track, and in-place air voids immediately after construction were approximately 20 percent. After 20 million ESALs were applied in the 2012 and 2015 research cycles, none of the test sections showed cracking. Rutting in all sections was very low at approximately 0.05 inches as shown in Figure 4.

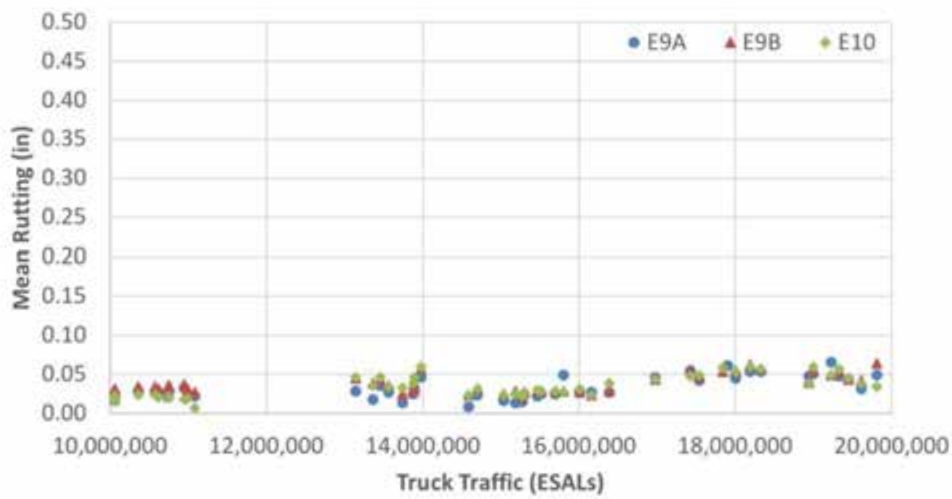

Figure 4 Rutting Performance Comparison

As previously shown in Figure 1, the 9.5-mm mix in Section E9A experienced an increase in roughness measured by IRI toward the end of the 2012 cycle, but the roughness level was consistent throughout the 2015 cycle as shown in Figure 5. Roughness in the 12.5-mm sections did not change over the two test cycles from 2012 through 2017. The mean texture depth of the 9.5-mm mix in Section E9A was approximately the same as the 12.5-mm mix in Section E9B, and they were both higher the GTR modified OGFC mix in Section E10 (Figure 6).

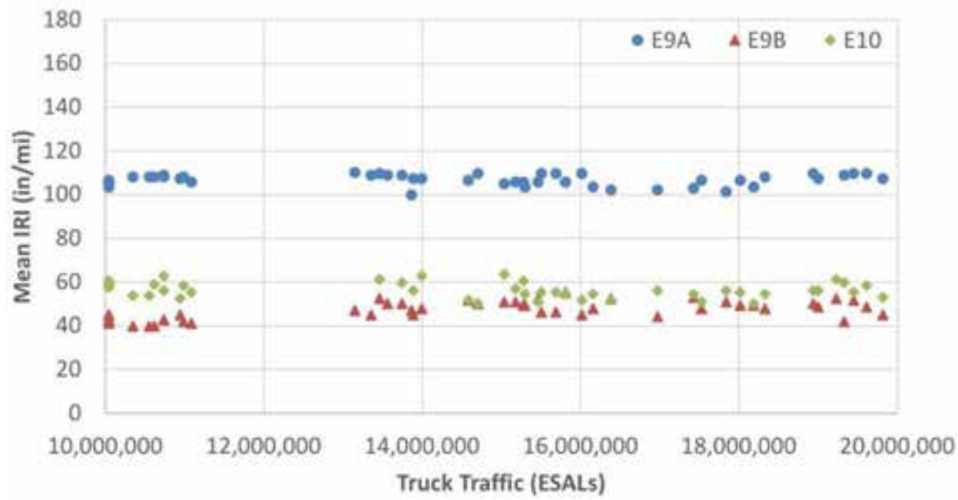

Figure 5 International Roughness Index Comparison

Figure 6 Mean Texture Depth Comparison

3.4 Conclusions and Recommendations

Based on the results of the laboratory testing and six-year field performance evaluation at the NCAT Test Track from 2012 through 2017, the following conclusions can be offered.

- The three OGFC mixtures selected for evaluation on the NCAT Test Track showed similar Cantabro loss and improved splitting tensile strength test results with higher air voids compared to the 12.5 mm OGFC mix design based on the current ALDOT mix design procedure. The OGFC mixes placed on the NCAT Test Track were designed with a design compaction effort of 50 gyrations to have minimum air voids of 15 percent, a maximum Cantabro loss of 15 percent, and a minimum conditioned splitting tensile strength of 50 psi.
- The 9.5-mm mixture in Section E9A had higher tensile strengths after moisture and freeze-thaw conditioning than the 12.5-mm mixes.

- While all the OGFC mix designs met the Cantabro loss requirement of maximum 15%, the 12.5 mm OGFC mix design based on the current ALDOT mix design procedure had much lower tensile strength after moisture and freeze-thaw conditioning especially when compared with the 9.5-mm OGFC mixture.
- All the mixes were placed 0.75 inches thick on the NCAT Test Track with in-place air voids immediately after construction at approximately 20 percent. The 9.5-mm mixture in Section E9A experienced an increase in roughness toward the end of the 2012 research cycle, but this increased roughness level stayed the same throughout the 2015 cycle. The roughness of the 12.5-mm mixes was consistent throughout the two research cycles. The three mixtures performed well without cracking and experienced minimum field rutting of approximately 0.05 inches after 20 million ESALs from 2012 through 2017. There was no sign of raveling nor significant difference in the performance between the three OGFC mixes on the NCAT Test Track.

Since the three OGFC test sections still performed well on the NCAT Test Track, ALDOT has decided to continue trafficking these test sections for another research cycle until 2021. Based on the laboratory and field evaluation performance, finer OGFC mixtures designed at 50 gyrations with a minimum air void content of 15 percent, a maximum Cantabro loss of 15 percent, and a minimum conditioned splitting tensile strength of 50 psi may be further evaluated for potential implementation in Alabama.

3.5 References

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CHAPTER 4 COLLABORATIVE AGGREGATES DELTA S REJUVENATOR STUDY

4.1 Background

Reclaimed asphalt pavement (RAP) materials have been widely used in asphalt mixtures because of their economic, environmental, and social benefits. Using RAP helps lower production costs, conserve natural resources, reduces the energy consumption associated with material extraction, and saves landfill and stockpiling areas (*1*). It is a common practice among producers to use RAP as a component in new asphalt mixtures. It was reported that 74.2 million tons of RAP were used in new pavements in the United States in 2015, with an estimated 85.1 million tons of RAP produced nationwide by the end of that year. In 2015, the average RAP content in asphalt mixtures was 20.3%, with most state highway agencies allowing the use of up to 25% RAP in their asphalt mixtures (*2*).

Due to their previous exposure to the environment, aged binders in RAP can be stiffer and more susceptible to cracking distresses. For this reason, there have been concerns that using higher proportions of RAP in asphalt mixtures could result in stiffer mixtures that are likely prone to cracking and would result in higher maintenance and rehabilitation costs (*3*). Several methods have been proposed to reduce the potential effects of recycled binders on the field performance of asphalt mixtures. One method is to use rejuvenators to restore some rheological properties of oxidized asphalt binders in recycled mixtures. These rejuvenators can be petroleum-based or bio-based materials that have been formulated to restore the balance of maltenes that were lost or transformed to asphaltenes in the oxidized asphalt binders (*4*).

Delta S is a bio-based, commercially available rejuvenator developed by Collaborative Aggregates for use in recycled asphalt mixtures. This rejuvenator was used in the surface mixture of Section N7 for evaluation on the NCAT Test Track.

4.2 Objective and Scope

The overall purpose of this study was to evaluate the effects of Delta S on the field performance of a surface asphalt mixture with a high RAP content. This study included several tasks with the following specific activities:

- Evaluate laboratory properties of the plant-produced mix with Delta S;
- Monitor field performance of the Delta S mixture in Section N7 of the Test Track; and
- Compare laboratory and field performance properties of the Delta S mixture with those of selected mixtures evaluated on the Test Track since 2015.

4.3 Original Construction of Section N7

The surface layer of Section N7 was built in July 2015 using a 9.5-mm nominal maximum aggregate size (NMAS) mixture with 20% RAP and 5% recycled asphalt shingles (RAS) with the addition of Delta S. The rejuvenator was added to the virgin binder at a dosage of 10% by weight of the recycled binders available in the RAP and RAS materials based on an initial binder testing experiment at NCAT. The virgin binder used for this mixture was PG 67-22.

Truck trafficking started in October 2015. Cracking was first noticed in Section N7 in late January 2016, at which time the section had received 1.4 million equivalent single axle loads (ESALs). In March 2016, when the section had endured 1.8 million ESALs, full depth cores of the 6-inch asphalt pavement structure were extracted from an uncracked area for analysis to determine the reasons for the premature cracking.

Results from the analysis of the extracted cores confirmed that the failure originated from delamination between the surface and intermediate asphalt layers, which was causing the cracking problems observed on the pavement surface. When cracking began developing more quickly, traffic was diverted from this section and additional cores were obtained from areas that showed transverse cracking. These cores showed that the existing de-bonding problems had weakened the pavement structure, causing the intermediate and base layers to crack.

4.4 First Repave of Section N7

Due to delamination and cracking occurring in all of the asphalt layers, it was decided that the best course of action would be to replace the entire asphalt structure. For this repave, the base and intermediate asphalt layers were constructed using highly polymer-modified asphalt (HiMA) mixes. These base and intermediate HiMA mixes matched the ones used in the seven test sections in the Cracking Group experiment on the Test Track. The thickness and mixture for the surface layer was the same as the original N7 design.

Repaving work started with the removal of the original pavement structure by milling on April 12, 2016, as shown in Figure 1. After compacting the aggregate base and verifying density requirements were met, paving of the base and intermediate asphalt courses started on April 13, 2016 (Figure 2). The surface layer of Section N7 was completed on April 15, 2016.

Figure 1 Milling of the Original Surface of Section N7


Figure 2 First Repave of Section N7

Within one day after the first repave of Section N7 was complete, slippage failures were observed. Figure 3 shows one of the slippage failures seen at this stage. Observation of the asphalt mixture after peeling from its surface with a skid steer revealed an oily and wet interface between the surface and binder layers with an asphalt mixture that was softer than usual and was easily broken, as shown in Figure 4.


Figure 3 Slippage Failures of Section N7 After Repaving

Figure 4 Interface Between Layers in Failure Area

Truck traffic was diverted to avoid further damage in this section and cores were extracted from the section outside of the edge line. Bond strength and splitting tensile strength tests were performed on these cores in accordance with ALDOT-430 and AASHTO T283. Bond strength results were lower than the minimum value (100 psi) recommended for interface bond strength. Section N8, with a comparable asphalt mixture (without Delta S) and location within the Test Track, showed considerably higher bond strength results. Results from bond strength tests on Sections N7 and N8 cores are shown in Figure 5 with the recommended minimum value of 100 psi marked.

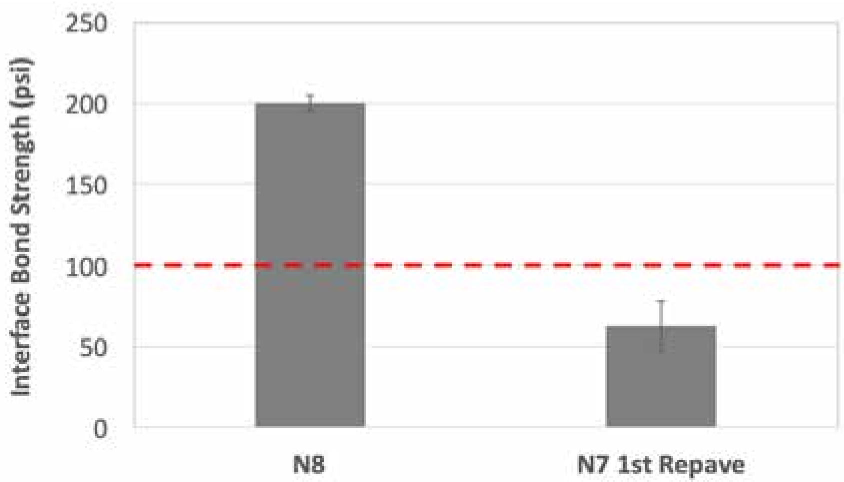

Figure 5 Bond Strength Between Surface and Binder Layers for Sections N8 and N7

The splitting tensile strength test results for field cores from Section N7 were considerably lower than those obtained from laboratory-compacted, reheated plant mix specimens, as shown in Figure 6. The main difference between these two sets of specimens was the reheating of plant mix for compacting laboratory specimens, which may have improved the interaction of the aged binder and the Delta S rejuvenator in the mixture. During the original construction and the first repave of Section N7, the asphalt mixture was laid and compacted without any silo storage as it was produced and hauled to the Test Track (approximately 10 minutes away) for immediate paving. Because of this, the interaction between Delta S (blended with the virgin binder) and the recycled binder, especially in the RAS, may not have been complete, leaving a higher proportion of Delta S in the virgin binder than originally intended. This caused reduced stiffness and splitting tensile strength, leading to bond failures and slippage problems.

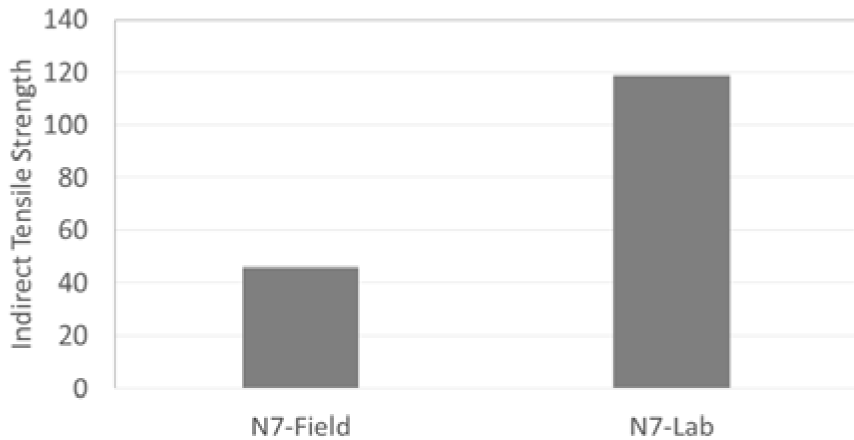

Figure 6 Splitting Tensile Strength Results of Field Cores and Reheated Plant Mix Specimens for Section N7

4.5 Second Repave of Section N7

After discussions with the sponsor, it was decided that the interaction between Delta S and the aged binder in the RAS should be further studied and that the wearing course of Section N7 would be replaced with a mixture containing only RAP materials. This mix would have 35% RAP with a recycled binder ratio similar to those of the original N7 and N8 surface mixes. Because this new design did not include RAS (even though it had a similar recycled binder ratio), the Delta S dosage was reduced to 5% by weight of the aged RAP binder, which was 5% lower than the dosage used in the original N7 mix with 20% RAP and 5% RAS. The N7 surface mixture was designed to compare directly with the N1 surface mix, which is the control mix for the Cracking Group experiment. Finally, it was determined that the mixture would be kept in a silo for two hours before paving. The second repave of Section N7 was started the week of May 9th, 2016. By May 12th, work was finished and trafficking was resumed. Figure 7 shows Section N7 after the second repave.

Figure 7 Section N7 After the Second Repave

The corrective actions taken for the second repave of Section N7 addressed some problems identified in the original construction and the first repave. To date, approximately 10 million ESALs have been applied for the 2015 research cycle sections, with the second repave of Section N7 having endured around 7.5 million ESALs. Performance of Section N7 has been satisfactory, and the slippage failures that appeared in previous paving from the delamination of the asphalt layers have not been observed.

4.6 Experimental Plan

The Delta S evaluation on the Test Track focused on the assessment of laboratory and field performance of the surface mixture of Section N7 and how it compared to the surface mixtures in Section N1 and N8 of the Cracking Group experiment. The cross-sections of the pavements analyzed in this laboratory and field evaluation are shown in Figure 8, where AC represents asphalt concrete.

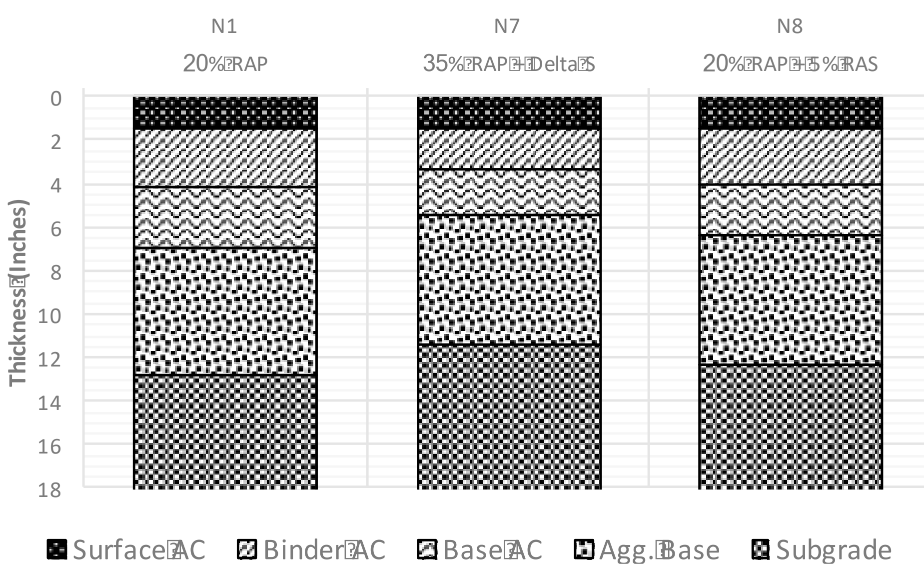

Figure 8 Pavement Structures of Sections N1, N7, and N8

Rutting and moisture susceptibility were analyzed for the surface mixture of Section N7 as part of the mix design process. Binders were extracted from all of the mixtures and analyzed to determine the performance grade of the binder blends. The stiffness and cracking performance were compared with the surface mixtures of Sections N1 and N8 using various laboratory performance tests, as shown in Table 1. A modification of the Texas Overlay Test was recently developed at NCAT and is referred to as the NCAT Overlay Test in Table 1. This test setup includes the following changes: (1) a higher test frequency of 1 Hz, up from 0.1 Hz in the original Texas OT; (2) a lower maximum opening displacement of 0.381 mm, changed from 0.635 mm in the original Texas OT; and (3) a redefined failure point at the number of cycles corresponding to the peak of the “load x cycle” curve instead of the point where the applied loads are reduced by 93% in comparison with the initial loading cycle.

Table 1 Laboratory Testing Plan

Property	Test Conducted	Standard Method/Practice
Binder properties	Performance grading of tank and extracted binders	AASHTO T164 Method A, ASTM D5404/D5404M, AASHTO M320
Moisture susceptibility	Hamburg Wheel-Tracking Test	AASHTO T324-14
Rutting resistance		
Mixture stiffness	Dynamic Modulus Test	AASHTO T 378-17 and AASHTO PP61-13
Cracking performance	Energy Ratio Test	ASTM D7369-11, AASHTO T322-07 and ASTM D6931-12
	Texas Overlay Test	Tex-248F
	NCAT Overlay Test	Modified from Tex-248F
	Illinois Flexibility Index Test	AASHTO TP124-16

A summary of properties for the three surface mixtures is shown in Table 2, followed by a brief description of each mixture. The base and binder asphalt layers in all of the sections consisted of the same highly polymer-modified asphalt mixture, which was designed to be resistant to fatigue cracking so that all cracking would occur in the surface layer. The aggregate base layer consisted of a crushed granite base. The subgrade at the Test Track is classified as an A-4 soil according to the AASHTO soil classification system.

Table 2 Surface Mixture Quality Control (QC) Properties

Mixture	Virgin Binder PG	As-built Thickness (in)	In Place %G_{mm}	QC P_{be} (%)	QC V_a (%)	QC VMA (%)	Recycled Binder Ratio
N1 20% RAP	67-22	1.6	93.6	4.7	7.0	14.7	0.177
N7 35% RAP+Delta S	67-22	1.5	92.1	5.2	7.0	16.0	0.282
N8 20% RAP+5% RAS	67-22	1.5	91.5	4.8	7.0	14.4	0.372

Section N1 (20% RAP)

Section N1 was built using a 9.5 mm NMAS mixture with 20% RAP and PG 67-22 virgin binder. It represents a typical mixture in the industry, where according to NAPA, an average of 20.3% RAP is used in asphalt mixtures (*2*). This mixture was selected as the control mixture for the Cracking Group experiment carried out at the Test Track as part of this research cycle.

Section N7 (35% RAP+Delta S)

Section N7 was built with a 35% RAP surface mixture. The construction of this section was part of a study sponsored by Collaborative Aggregates to test the effectiveness of Delta S on the field performance. To produce the N7 surface mixture, Delta S was in-line injected into the PG 67-22 binder supply at a target rate of 5% by weight of RAP binder. To give the Delta S time to interact with the aged binder in the RAP and to avoid issues related to slippage and de-bonding caused by mix softness, the mixture was stored in a silo for two hours before being transported to the Test Track for paving.

Section N8 (20% RAP+5% RAS)

Section N8, also part of the Cracking Group experiment at the Test Track, had a surface mixture designed to include 5% RAS in addition to the 20% RAP and PG 67-22 virgin binder that the control mixture had. The virgin aggregates used were the same, but the gradation was slightly modified to accommodate the added RAS. The result was a stiffer mixture that was not expected to have adequate cracking performance.

4.7 Field Performance

The Test Track produces an accelerated accumulation of traffic-produced damage over a reduced time. For each research cycle, 10 million ESALs are applied and pavement responses are constantly monitored. Traffic is suspended each Monday and the pavement sections are measured for ride quality, rutting, and cracking performance. Ride quality and rutting are measured with a Dynatest inertial profiler, shown in Figure 9, which determines the international roughness index (IRI) of each section. For cracking performance, sections are inspected visually, and when observed, cracks are mapped and measured (Figure 10). Cracks are considered to have an area of influence that is 6 inches to each side. Linear measurements of the cracks are performed to determine the cracked area of the sections. With these measurements, cracking is then calculated based on the percent lane area of each section.

Figure 9 Inertial Profiler Used to Assess Ride Quality and Rutting Measurement

Figure 10. Mapped Cracks on the Test Track

The measured distresses were used to rate the condition of the pavement sections based on the ratings scale recommended by the Federal Highway Administration (FHWA) in the Code of Federal Regulations 23 490.313 “Calculation of Performance Management Measures.” This method rates the pavement condition in each of the distress categories. Table 3 shows the condition thresholds for each of the distresses.

Table 3 Pavement Condition Thresholds

Distress	Condition		
	Good	Fair	Poor
Ride quality (IRI – m/km)	< 1.5	1.5 – 2.7	> 2.7
Rut depth (mm)	< 5	5 – 10	> 10
Cracking area (%)	< 5	5 – 20	> 20

4.8 Results and Discussion

Binder Test Results

Table 4 shows the test results of tank and extracted binders. In addition to the performance grade of each binder, the difference between the critical temperatures at which the S value is 300 MPa and the temperature at which the m-value is 0.3, obtained from the bending beam rheometer (BBR) binder test and referred to as ΔT_c, are also reported in Table 4. This parameter has been found to be an indicator of non-load related cracking potential, and AASHTO PP78 “Design Considerations When Using Reclaimed Asphalt Shingles (RAS) in Asphalt Mixtures” recommends a ΔT_c threshold of -5.0 °C for recovered binders.

The virgin binder used in all three surface mixtures was a PG 67-22 but had a continuous performance grade of 70.2-24.0 and a ΔT_c value of -3.6°C. The binder blend extracted from the N1 mixture (control) had a continuous performance grade of 88.6-16.6 and a ΔT_c value of

-9.4°C. During the mix design, the dosage of Delta S was selected at 5% by weight of RAP binder so that the lower temperature grade and ΔT_c of the binder blend in the N7 mixture would match the N1 mixture. The binder blend extracted from the N7 mixture had a high-temperature performance grade of 94°C, which was higher than the binder blend extracted from the N1 mixture. However, the low-temperature grade (-16.4°C) and ΔT_c value (-10.1°C) of the binder blend extracted from the N7 mixture were very close to the N1 mixture. The binder blend extracted from the N8 mixture showed that this mix would be the stiffest with a continuous performance grade of 107.3-5.4 and a ΔT_c value of -20°C. From these results, the surface mixtures in Sections N1 and N7 were expected to have similar resistance to thermal cracking while the surface mixture in Section N8 was anticipated to be the most susceptible to block cracking or weathering distresses.

Table 4 Extracted Binder Analysis Results

Mixture	Virgin Continuous PG	Virgin ΔT_c	Extracted Continuous PG	Extracted ΔT_c
N1 20% RAP	70.2-24.0	-3.6	88.6-16.6	-9.4
N7 35% RAP+Delta S	70.2-24.0	-3.6	94.5-16.4	-10.1
N8 20% RAP+5% RAS	70.2-24.0	-3.6	107.3-5.4	-20.0

Hamburg Wheel-Tracking Test Results

Table 5 shows the Hamburg test results of the N7 surface mixture. Since the binder blend extracted from the plant mix was PG 94-10 (Table 4), the Hamburg test required a minimum of 20,000 passes to reach the critical 12.5-mm rut depth for rutting resistant mixtures. The N7 mixture showed an average of 19,200 passes to reach 12.5 mm rut depth, slightly lower than the rutting threshold of 20,000 passes. Also, the mixture had an average stripping inflection point of 15,000 passes, higher than the 10,000 passes previously proposed for stripping resistant mixtures. Based on the Hamburg test results, the N7 mixture was expected to have good resistance to moisture-induced damage and satisfactory resistance to rutting.

Table 5 Summary of Hamburg Results for Section N7 Mixture

Parameter	Result	Criteria	Pass/Fail
Rut depth at 10,000 passes (mm)	3.29	N/A	N/A
Rut depth at 20,000 passes (mm)	13.41	N/A	N/A
Passes to 12.5 mm rut	19,200	20,000	Fail
Stripping inflection point	15,000	10,000	Pass

Dynamic Modulus Test Results

The dynamic modulus (E*) master curves for the three mixtures are shown in Figure 11. The fitting statistics for the generated master curves are shown in Table 6, and Table 7 gives a summary of the regression coefficients. The fitting statistics Se/Sy and R^2 indicated an excellent fit between measured and predicted data for all of the mixtures. Based on the Gamma factor, which describes the steepness of the curves, the N8 mixture had the lowest slope, meaning that this mixture was less susceptible to changes in temperature/frequency. The N1 mixture had the highest slope, suggesting that it was the most susceptible to temperature/frequency changes.

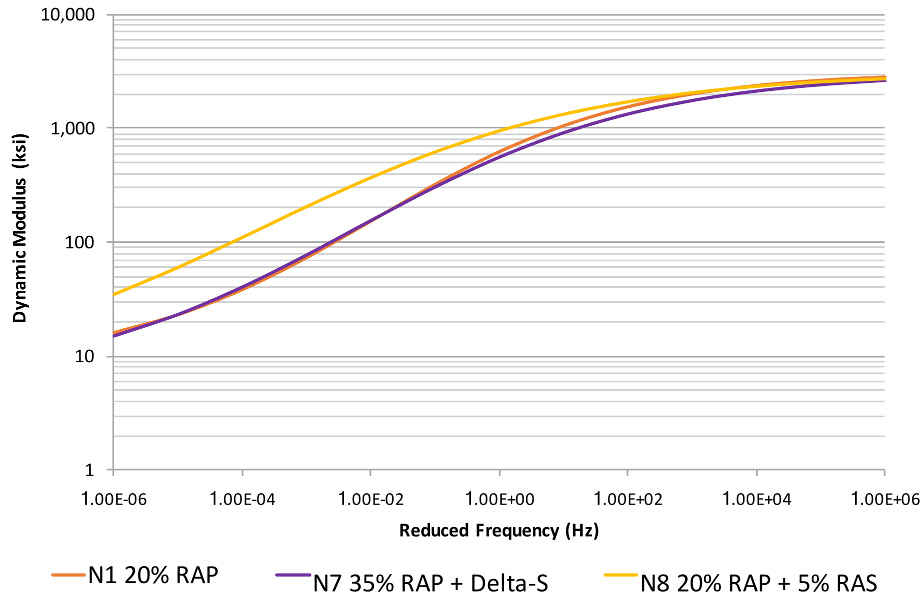

Figure 11 Master Curves of Analyzed Mixtures

Table 6 Master Curves Goodness of Fit Parameters

Mixture	R^2	Se/Sy
N1 20% RAP	0.997	0.036
N7 35% RAP+Delta S	0.997	0.036
N8 20% RAP+5% RAS	0.999	0.023

Table 7 Master Curves Coefficients

Mixture	Max E* (ksi)	Min E* (ksi)	Beta	Gamma	EA
N1 20% RAP	3159.19	8.36	-0.989	-0.510	201423.2
N7 35% RAP+Delta S	3125.02	5.33	-0.997	-0.441	219584.8
N8 20% RAP+5% RAS	3148.88	5.60	-1.463	-0.395	216839.5

The generated master curves divided the mixtures into two distinct groups. The N7 mixture with Delta S and 35% RAP had a master curve close to that of the N1 mixture, which had 20% RAP. The N8 mixture, with 20% RAP and 5% RAS, was stiffer. These results indicate that the addition of Delta S lowered the stiffness of the N7 mixture closer to that of N1, whose RAP content was 15% lower.

Energy Ratio

The dissipated creep strain energy at failure ($DCSE_{HMA}$), the minimum dissipated creep strain energy ($DCSE_{Min}$), and Energy Ratio (ER) of the mixtures obtained from the ER tests are shown in Figures 12 through 14. The N1 mixture had the highest $DCSE_{HMA}$ and $DCSE_{Min}$ results, followed by the N7 and N8 mixtures. The only mixture meeting the recommended $DCSE_{HMA}$ range of 0.75 – 2.5 kJ/m^3 was Section N8, while the others exceeded that range. The ER results showed a different trend. The mixture with the highest ER was N8, which was the worst ranked in the other tests. This can be explained by the low $DCSE_{Min}$ result obtained from this mixture, which is inversely proportional to the ER value. The $DCSE_{Min}$ is affected by creep compliance parameters, and the N8 results for this test were very low due to its high stiffness. All the mixtures exceeded the minimum ER criteria established in Table A.5 for any traffic level.

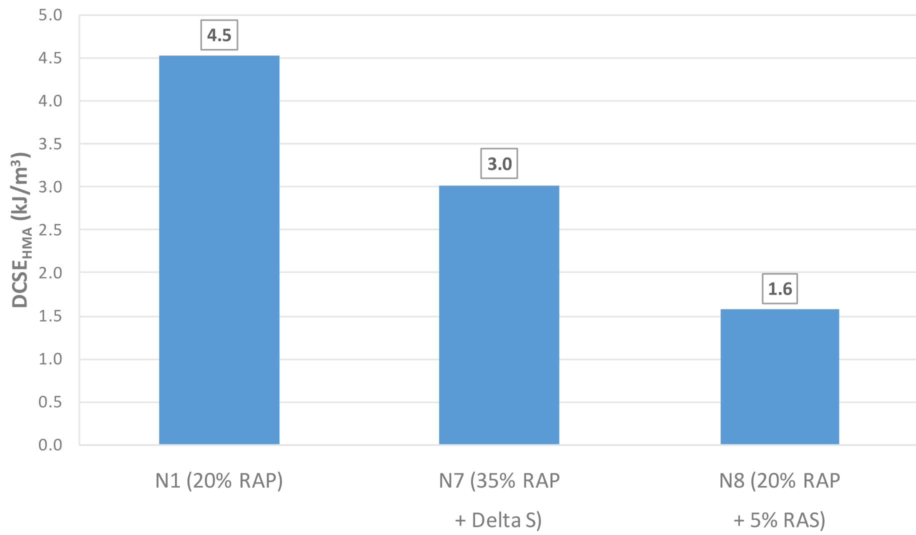

Figure 12 Dissipated Creep Strain Energy at Failure

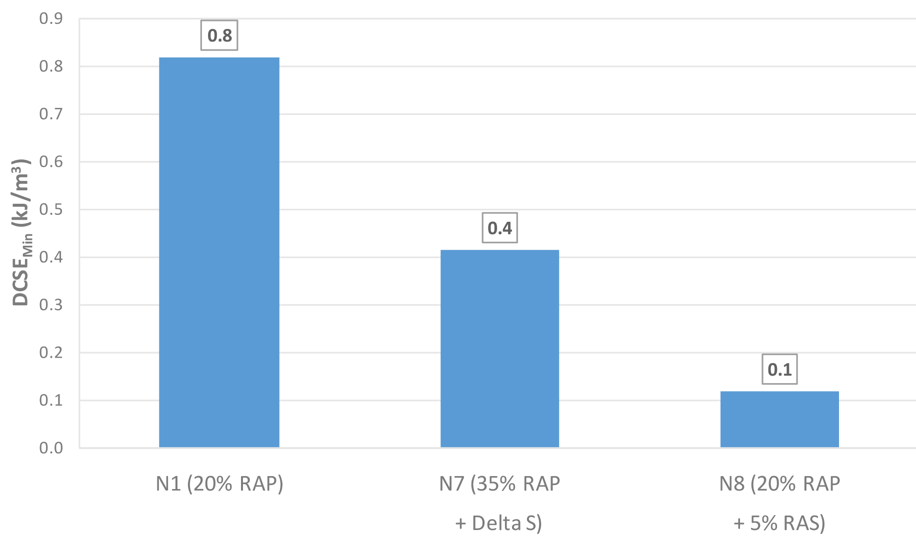

Figure 13 Minimum Dissipated Creep Strain Energy

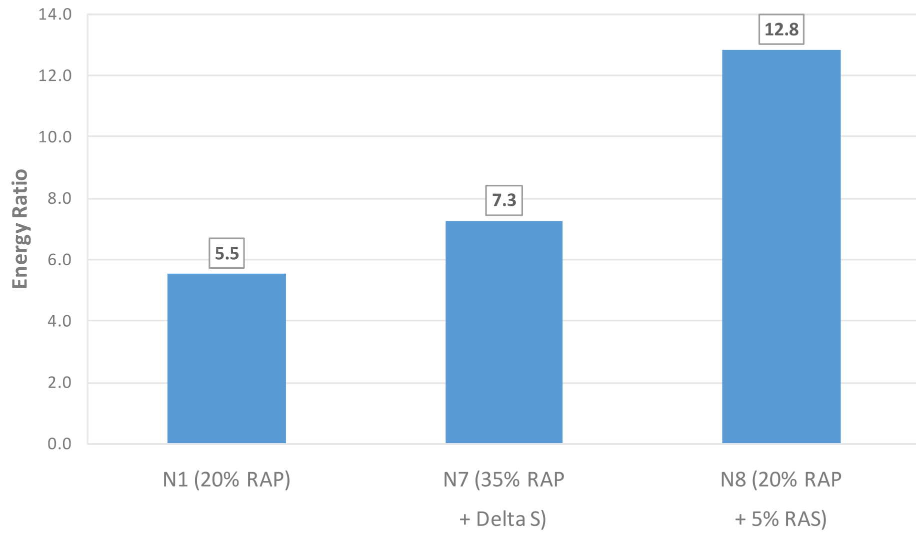

Figure 14 Energy Ratio

Texas Overlay Test

The Texas Overlay Test (OT) results are shown in Figure 15 with the error bars representing the standard deviation of test results for each mixture. The N1 mixture performed best, which failed at an average number of cycles to failure of 25. The N7 mixture with 35% RAP and Delta S failed at an average number of cycles to failure of 10. Finally, the N8 mixture was the one that failed quickest in this test. On average, the variability for all the data sets was very high.

A Tukey-Kramer statistical comparison with a 95% confidence interval was used to rank and group the Texas OT results. Table 8 shows the results ranked from best to worst performance according to the statistical analysis with their standard deviation and coefficient of variability (COV). From this analysis, no significant differences were found in the OT results obtained.

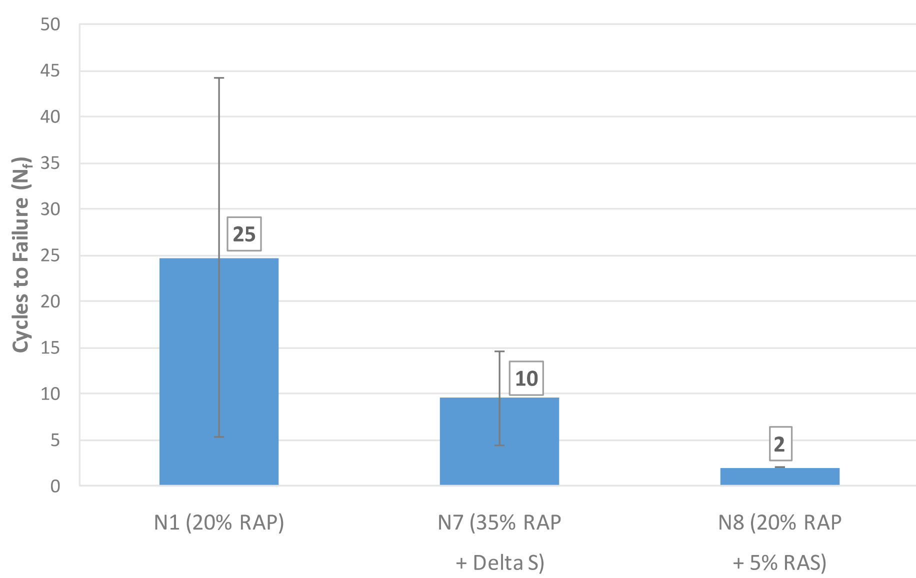

Figure 15 Texas OT Results

Table 8 Texas OT Results and Ranking of Performance

Mixture	N_f	Standard Deviation	COV (%)	Grouping
N1 20% RAP	25	19.5	79	A
N7 35% RAP+Delta S	10	5.1	53	A
N8 20% RAP+5% RAS	2	0	0	A

* Means that do not share a letter are statistically different.

NCAT-Modified Overlay Test

The NCAT-Modified overlay test (NCAT-OT) results are shown in Figure 16 with the error bars representing the standard deviation of the NCAT-OT test results for each mixture. The trends and rankings of the mix performance were similar to the Texas OT results. The N1 mixture had the highest number of cycles to failure. It was followed by the N7 mixture with Delta S. Finally, the N8 mixture had the lowest cycles to failure. A Tukey-Kramer statistical analysis at a 95% confidence interval was performed to rank and group the NCAT-OT results, which are shown in Table 9. The variabilities of these results were considerably lower than those of the Texas OT results. The statistical groupings classified N1 as the mixture with the best performance with N7 and N8 grouped together with no significant difference and lower cracking resistance.

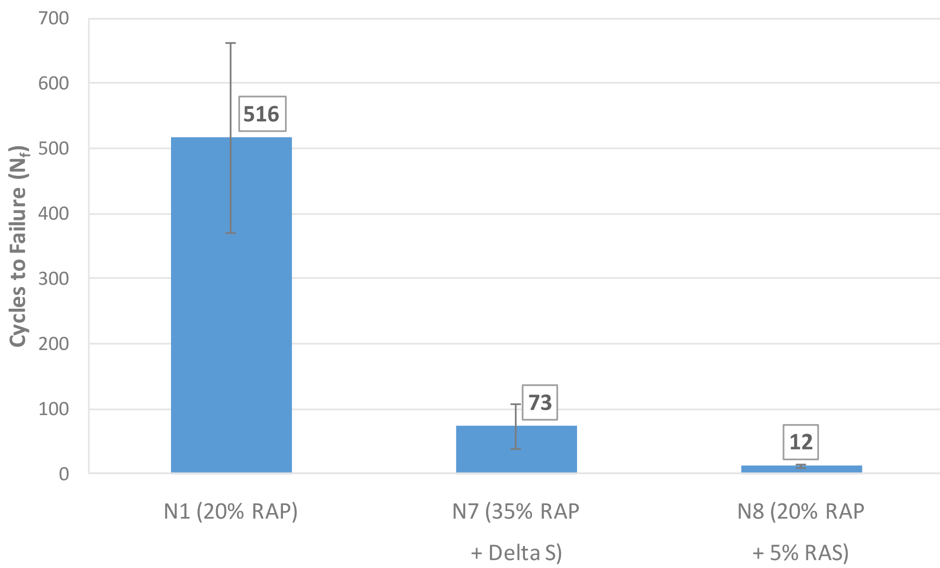

Figure 16 NCAT-OT Results

Table 9 NCAT-OT Results and Ranking of Performance

Mixture	N_f	Standard Deviation	COV (%)	Grouping
N1 20% RAP	516	146.0	28	A
N7 35% RAP+Delta S	73	34.1	47	B
N8 20% RAP+5% RAS	12	2.1	17	B

* Means that do not share a letter are statistically different.

Illinois Flexibility Index Test

The Illinois flexibility index test (I-FIT) results are shown in Figure 17 with the error bars representing the standard deviations. Similar trends were observed as in Texas and NCAT-modified overlay tests; the mixtures ranked in the same order, although the variabilities were less significant. The N7 mixture with Delta S had results that were close to N1. The mixture with the lowest flexibility index (FI) was the one in Section N8. From a Tukey-Kramer statistical analysis at a 95% confidence interval, shown in Table 10, the differences between the N1 and N7 mixtures were found to be statistically insignificant and their results were statistically higher than the results of the N8 mixture.

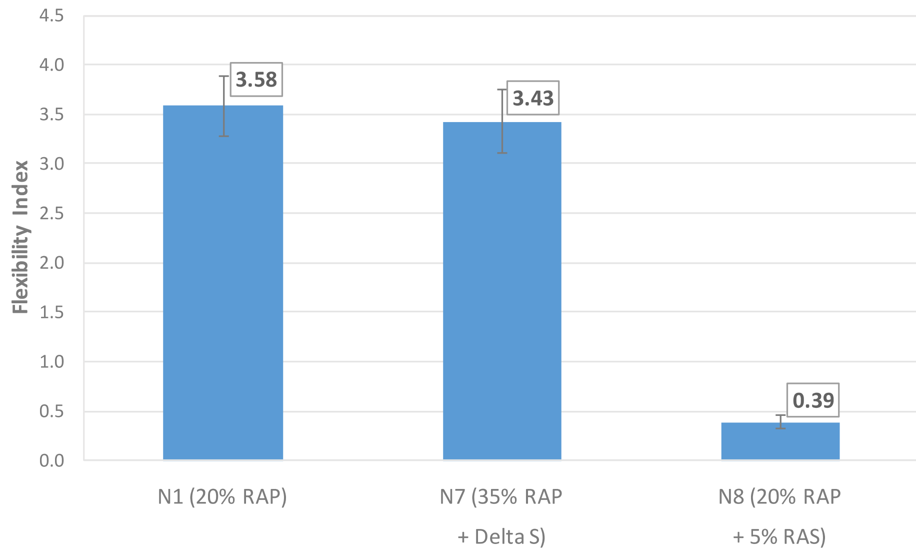

Figure 17 I-FIT Results

Table 10 I-FIT Results and Performance Ranking

Mixture	FI	Standard Deviation	COV (%)	Grouping
N1 20% RAP	3.58	0.30	8	A
N7 35% RAP+Delta S	3.43	0.32	9	A
N8 20% RAP+5% RAS	0.39	0.07	18	B

* Means that do not share a letter are statistically different.

Field Performance

The field performance of the three test sections was measured weekly at the Test Track. The final measurements were taken after 10 million ESALs had been applied on Sections N1 and N8 and 7.5 million ESALs had been applied on Section N7 since the second repave.

Table 11 shows the ride quality and rut depth results. All sections had relatively low IRI readings, indicating a good ride quality. The section with the highest IRI was N1 with 1.1 m/km (70 in/mile), which would be considered in good condition based on the FHWA performance measures. All of the sections had rutting measurements less than 5 mm, also considered good condition based on the FHWA performance measures.

Table 11 Field Performance

Section	IRI (m/km)	Rut Depth (mm)
N1 20% RAP	1.1	1.8
N7 35% RAP+Delta S	0.9	3.3
N8 20% RAP+5% RAS	0.8	1.3

Based on the field cracking measurements shown in Figure 18, the first appearance of hairline cracks (less than 1 mm wide) as shown in Figure 19 was noticed in Section N1 shortly after 6 million ESALs, in Section N8 after 5 million ESALs, and in Section N7 after 3.5 million ESALs (from the second repave). These cracks were manually surveyed, as they were not detectable by an automated survey system. This information was reported for research purposes but should not be used to rank the performance of these test sections based on the FHWA performance measures, as these sections would show no cracks if they were surveyed using an automated cracking survey system. Sections N1 and N8 had a low percentage of hairline cracks up to 9 million ESALs, and Section N7 showed a low percentage of hairline cracks up to 6.5 million ESALs (from the second repave). After that, hairline cracks grew quickly in these sections because of colder weather at the Test Track. At the end of the 2015 research cycle, the hairline cracks were observed in approximately 20 percent of the lane area in each section, and these cracks were still very tight (less than 1 mm) and undetectable by an automated cracking survey system.

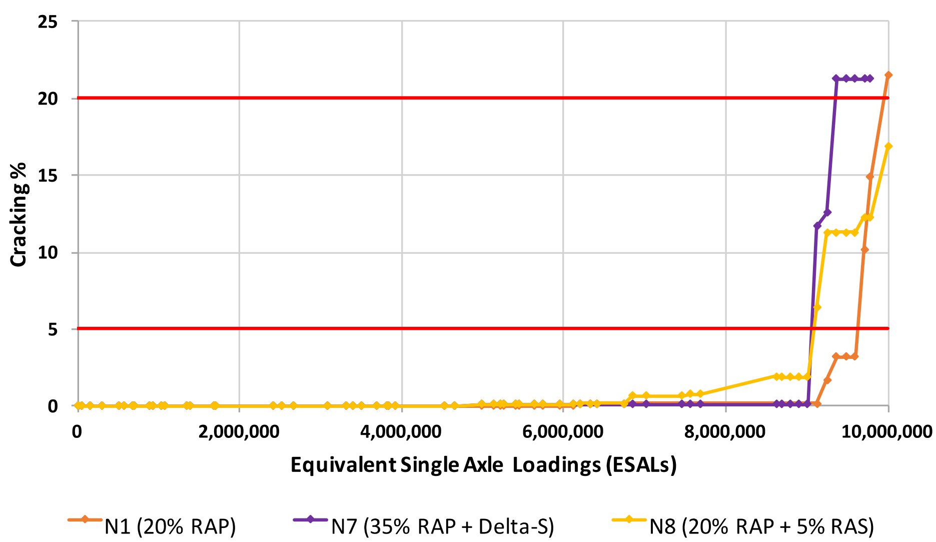

Figure 18 Field Cracking Measurements

Figure 19 Close-up of Hairline Cracks in Sections N7 and N1

Summary of Results

Results from the extracted binder tests and the field measurements of IRI and rut depth are shown in Table 12. Results from the laboratory cracking tests performed on these mixtures and the field cracking measurements are shown in Table 13. Also shown are the ΔT_c data from the extracted binders.

Since all of the mixtures had high high-temperature performance grades, they showed good rutting performance on the Test Track. The ΔT_c results seemed to agree with the I-FIT test results. Due to variable results, it was difficult to distinguish the performance of the three mixtures using the overlay test. Also, the three test sections had similar field cracking performance, making it difficult to differentiate their performance for comparison with the laboratory test results. These sections will be kept in place for another research cycle to allow for a thorough field performance evaluation.

Table 12 Extracted Binder, Field Measured IRI, and Rutting Results

Mixture	Extracted PG	IRI (m/km)	Rutting (mm)
N1 20% RAP	88-16	1.1	3.4
N7 35% RAP+Delta S	94-10	0.8	3.2
N8 20% RAP+5% RAS	106-4	0.8	2.0

Table 13 Laboratory and Field Cracking Performance

Mixture	ΔT_c	ER	Texas OT (Nf)	NCAT- OT (Nf)	I-FIT (FI)	Crack (width < 1mm) (%)
N1 20% RAP	-9.4	5.5	25 (A)	556 (A)	3.58 (A)	21.5
N7 35% RAP+Delta S	-10.1	7.3	10 (A)	73 (B)	3.43 (A)	21.3
N8 20% RAP+5% RAS	-20.0	12.8	2 (A)	12 (B)	0.39 (B)	16.9

*Letters next to Texas OT, NCAT-OT, and I-FIT results represent groupings from statistical analysis.

4.9 Conclusions and Recommendations

This study consisted of evaluating the laboratory and field performance of the surface mixture in Section N7 of the NCAT Test Track. This section was designed with 35% RAP with a bio-based rejuvenator called Delta S. The performance of this mixture was compared with two other surface mixtures paved in 2015 as part of the Cracking Group experiment, including Section N1 with 20% RAP and Section N8 with 20% RAP and 5% RAS. The three mixtures used the same PG 67-22 virgin binder and had similar gradations and volumetric properties.

Samples of plant mix were taken during construction and tested to determine the stiffness, rutting, and cracking resistance of these mixtures. These results were compared with the ride quality, rutting, and cracking measurements at the Test Track. Based on the results obtained, the following conclusions can be made:

- Section N7 with Delta S had a ride quality that would be classified as good condition in the pavement performance measures recommended by the FHWA, and it was similar to those of the other two sections.
- Based on the field rutting measurements, Delta S did not produce any negative effects on rutting performance, which agreed with the Hamburg test results. Low severity rutting has been recorded for Section N7 (second repave) after 7.5 million ESALs and for Sections N1 and N8 after 10 million ESALs due to the stiff binders in the three mixtures with the lowest high-temperature performance grade being 88°C. The dynamic modulus test results showed that the N7 mixture was as stiff as the N1 mixture, and they were not as stiff as the N8 mixture, which agreed with the high-temperature performance grades of the binders extracted from the plant mixtures.
- Based on the ΔT_c data obtained from the extracted binders, the N7 mixture with Delta S had the same resistance to non-load related cracking as N1, which had lower RAP contents. The surface mixture in Section N8 was anticipated to be the most susceptible to block cracking or weathering distresses. The ΔT_c data ranked the three mixtures similar to the I-FIT test results. With high variabilities, the Texas OT results were not able to distinguish the cracking performance of the three mixtures.
- The field cracking performance of Section N7 with Delta S could be considered equal to Section N1. The percent lane area of hairline cracks (crack width of less than 1 mm) was reported for research purposes and should not be used to rank the cracking performance of these sections based on the FHWA performance measures, as these cracks were not detectable using an automated cracking survey system.
- Higher RAP contents would normally make the N7 mixture more susceptible to cracking than the N1 mixture, but this was not shown in the field, which can be attributed to the effect of Delta S. Section N7 showed acceptable performance toward the end of the 2015 research cycle.

Overall, Delta S did not produce any negative effects on the ride quality and rutting performance of the mixture. The cracking performance of the mixture was acceptable at the end of the 2015 research cycle. Delta S could be considered an alternative in the design and production of asphalt mixtures with high RAP contents. Section N7, as well as Sections N1 and

N8, will be kept in place for continuing traffic for another research cycle to allow for a thorough field performance evaluation.

4.10 References

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CHAPTER 5 FEDERAL HIGHWAY ADMINISTRATION DEVELOPMENT OF ASPHALT BOUND SURFACES WITH ENHANCED FRICTION PROPERTIES

5.1 Objective and Background

The objective of this study was to compare friction performance of asphalt bound friction surfaces to the standard high friction surface treatment (HFST). The FHWA Every Day Counts program focused on the polymer resin bound standard HFST for addressing high crash rate locations, such as horizontal curves, deceleration ramps, and intersection approaches. FHWA sponsored the 2015 study on the NCAT Test Track to examine the friction performance of asphalt bound surfaces to better quantify the role of asphalt bound friction surfaces for roadway safety. The primary measures for friction performance were the locked-wheel skid trailer, dynamic friction tester (DFT), high speed laser texture, and circular texture meter (CTM).

The standard HFST is most often a polymer resin-bound layer of calcined bauxite aggregate as specified in AASHTO PP 79-14 *Standard Practice for High Friction Surface Treatment for Asphalt and Concrete Pavements*. HFST with calcined bauxite has demonstrated the highest friction and high macro-texture characteristics for addressing high crash rate locations; however, the HFST surface treatment system (both polymer resin and calcined bauxite) is expensive, the aggregate must be imported, and there are a limited number of contractors that have the equipment and expertise to place the surface. A previous FHWA-sponsored study of HFST conducted at the NCAT Test Track found that polymer resin bound surfaces with other regionally available friction aggregate sources did not provide the same level of friction performance as the calcined bauxite (*1*).

This 2015 Test Track study explored the friction performance of asphalt bound surfaces with high cost calcined bauxite as the primary friction aggregate using conventional asphalt-based construction technologies to place micro-surfacing and thin asphalt overlays as opposed to the specialized application equipment required to place standard HFST. The cost of the calcined bauxite will increase the cost of the traditional asphalt bound surface but that cost will be substantially lower than a standard HFST. If successful, agency safety engineers will have other treatment options available for addressing crash locations.

The study consisted of the following steps:

1. Examine candidate asphalt surfaces and select the surface(s) with the highest potential to provide high friction performance.
2. Acquire a supply of calcined bauxite.
3. Develop a mix design incorporating the calcined bauxite as the primary coarse aggregate.
4. Place the asphalt surface on the Test Track in the west curve so the traffic polishing would be comparable to the previous HFST study.
5. Obtain a field produced mixture for accelerated friction testing in the NCAT lab.
6. Monitor and report friction and texture performance.

5.2 Surface Selection

The key factors for selecting the best asphalt surfaces included working with a calcined bauxite aggregate processed to a narrow gradation band and selecting a surface with high macro-texture. Calcined bauxite is processed to a narrow gradation passing the #6 sieve and retained on the #16 sieve. Compared to conventional asphalt surfaces, this is a relatively small size aggregate. High macro-texture can only be achieved by certain asphalt surface types, such as open-graded friction course (OGFC), stone matrix asphalt SMA, ultra-thin bonded wearing course (UTBWC), chip seals, and micro-surfacing. Both OGFC and UTBWC were removed from consideration because they would require a high percentage of calcined bauxite and the small size of the aggregate is not practical for these mixtures. The chip seal was removed because of concern about achieving the proper amount of asphalt bond on the small aggregate particles. The remaining surface types, SMA and micro-surfacing, were selected for the study. Simply based on the volume of material required for each surface layer, the SMA requires significantly more calcined bauxite per unit area, which translates into a higher material cost.

The SMA was placed on Test Track Section W3. The micro-surfacing test section was placed on W7, and after further discussions with FHWA, W7 was subdivided to include placement of a micro-surfacing with a Texas sandstone in addition to the micro-surfacing with calcined bauxite.

5.3 Material Sources

Funding for this study did not include the purchase of the calcined bauxite. NCAT researchers approached two materials suppliers and were able to obtain the required calcined bauxite quantities for the study. Ashapura, supplying calcined bauxite from a source in India, provided the quantity for designing and placing the micro-surfacing. Great Lakes Minerals, supplying calcined bauxite from a source in China, provided the quantity needed for developing an SMA mix design. Ashapura provided the quantity needed for the placement of the SMA. The Texas sandstone was on hand for other Test Track pavement preservation sections, so there was no additional cost for this material.

Calcined bauxite is a manufactured aggregate, not a naturally occurring aggregate. The quality and consistency of conventional aggregate for use as a friction aggregate is directly tied to the geology at the aggregate source. The quality of calcined bauxite for use as a friction aggregate is a function of the alumina-oxide (Al_2O_3) content of the mined ore and the manufacturing process. The same raw bauxite mineral source can be processed into different quality levels of calcined bauxite product by changing the manufacturing process. Therefore, it is not appropriate to conclude that calcined bauxite quality from a particular source is all the same. The quality of the calcined bauxite is described for each test section below.

5.4 Materials and Mix Design

Test Track Section W7 – Micro-Surfacing. The selected friction aggregates were a calcined bauxite and Texas sandstone. Ashapura provided the quantity of calcined bauxite needed for the Test Track. This material had an 84% Al_2O_3 (aluminum oxide) content. There were no laboratory or field studies in the United States with the 84% Al_2O_3 calcined bauxite prior to this study, so the friction performance was unknown. Great Lakes Minerals maintains a supply of calcined bauxite for HFST meeting AASHTO PP 79-14 (87% minimum Al_2O_3) but was not able to donate the required quantity. Photos of both samples are shown in Figure 1. The photo of the 84% Al_2O_3 calcined bauxite shows both crushed particles and rounded particles, which is atypical of the manufactured calcined bauxite used for HFST shown in the right photo.

Figure 1 Calcined Bauxite Samples

As shown in Figure 2, micro-deval tests on the 84% Al_2O_3 calcined bauxite had 8% loss compared to 2% loss for the 87% Al_2O_3 calcined bauxite used in the previous FHWA HFST study. The mass loss results for the 84% Al_2O_3 calcined bauxite sample were similar to the best regional friction aggregate (taconite) from the earlier FHWA HFST study (*1*). The graph also shows two other aggregates with lower mass loss, flint, and slag. These aggregates had lower friction performance compared to taconite in the field, therefore, the 8% mass loss for the 84% Al_2O_3 calcined bauxite stockpile may or may not correlate to lower friction performance in the field. Accelerated laboratory friction testing using the NCAT Three Wheel Polishing Device (TWPD) and DFT on a polymer resin-bound 84% Al_2O_3 calcined bauxite slab specimen was determined to be invalid due to the DFT malfunctioning.

A previous NCAT laboratory evaluation of a sandstone source from Oklahoma showed very good friction performance, so the Texas sandstone was expected to perform well (*2*). The Texas sandstone source was a preferred source used by Vance Brothers who worked in conjunction with NCAT for the pavement preservation test sections.

The binder used in the micro-surfacing was a highly polymer modified asphalt (HiMA) processed into a CSS-1HP emulsion for use in micro-surfacing applications and was supplied by Ergon Asphalt & Emulsions in Vicksburg, Mississippi. The HiMA base asphalt was selected to improve surface durability and aggregate particle retention compared to conventional emulsion.

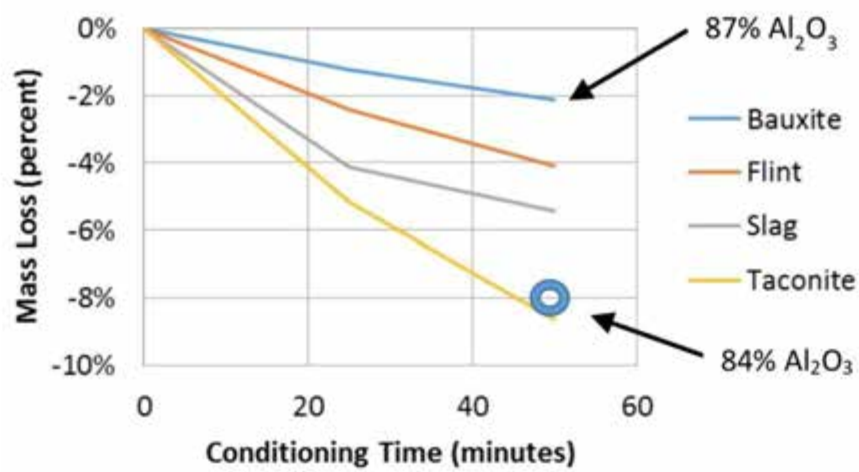

Figure 2 Comparison of Micro-Deval Test Results for Two Calcined Bauxite Samples (1)

The mix design of the calcined bauxite micro-surfacing and sandstone micro-surfacing were performed by Paragon Technical Services, Inc. The gradations for both are shown in Table 1. The calcined bauxite gradation was predominately 26% retained on the #8 sieve and 70% retained on the #16 sieve. This was blended with a Calera, Alabama limestone sand with 51% passing the #16 sieve. The mix was a 50:50 aggregate blend to satisfy micro-surfacing gradation targets. The coarse fraction of the blend (aggregate retained above the #16 sieve) was 67% calcined bauxite. The emulsion content was 12.5% by weight of dry aggregate (8.1% residual asphalt by aggregate weight).

Table 1 Gradations by Percent Passing of Section W7A&B Micro-surfacing

Sieve Size	Bauxite (50%)	Limestone (50%)	JMF Blend W7A	Sandstone W7B
3/8”	100	100	100	100
#4	100	99	100	90
#8	74	76	75	53
#16	4	51	28	32
#30	1	33	18	21
#50	1	20	11	15
#100	1	13	8	12
#200	0.7	10.8	6.7	9.7

The mix design gradation for the sandstone micro-surfacing had 47% retained above the #8 sieve compared to only 25% retained above the #8 sieve for the calcined bauxite gradation. The sandstone gradation had a coarse fraction of 68% retained above the #16 sieve, which was similar to the calcined bauxite gradation. The CSS-1HP emulsion content for the sandstone micro-surfacing was 12% by weight of dry aggregate (7.6% residual asphalt by aggregate weight).

Test Track Section W3 – SMA. To accomplish a surface with a dominant exposure of calcined bauxite, the SMA was designed as a 4.75 mm nominal maximum aggregate size (NMAS) mixture. The mix design process started with a previous 2003 Test Track Section (N7) granite SMA job mix formula (JMF) using a Marshall 50-blow design procedure. The initial W3 calcined bauxite SMA mix design used the Great Lakes Minerals calcined bauxite stockpile and was later adjusted (increased binder content) for the Ashapura calcined bauxite stockpile used for field production and placement. A PG 76-22 binder was selected for the W3 mix design and the final binder content was 8.3%. The final combined gradation was 40% calcined bauxite, 59% granite, and 1% filler as shown in Table 2. The coarse fraction of the gradation (retained on the #16 sieve) was 67% calcined bauxite. The Hamburg Wheel-Tracking Device (HWTD) performance test identified a potential for stripping in the laboratory mixture, so the field mixture was produced with an anti-strip agent. Figure 3 shows the summary plot of the HWTD test and Figure 4 is a photo of one of the failed test specimens. Based on the excessive amount of clean sand around the failed HWTD specimens, it appears that the stripping was caused by incompatibility between the granite and asphalt.

Table 2 Gradation by Percent Passing of Section W3 SMA with Calcined Bauxite Aggregate

Sieve Size	JMF Blend	Bauxite (40%)	Granite (59%)	Fly Ash (1%)
½”	100	100	100	100
3/8”	100	100	100	100
#4	100	100	100	100
#8	69	32	85	100
#16	47	1	65	100
#30	32	1	50	100
#50	20	1	35	100
#100	12	1	23	100
#200	8.1	0.5	14.1	100.0

Test Track Sections W8 and W9 – HFST. As a point of reference, three HFSTs from the earlier FHWA study remained in place and were tested quarterly (*1*). W8A used granite aggregate, W8B used calcined bauxite aggregate, and W9 used flint aggregate. W8B complies with the AASHTO PP 79-14 HFST standard with the calcined bauxite meeting the minimum 87% aluminum-oxide specification requirement and is considered the “gold standard” for high friction.

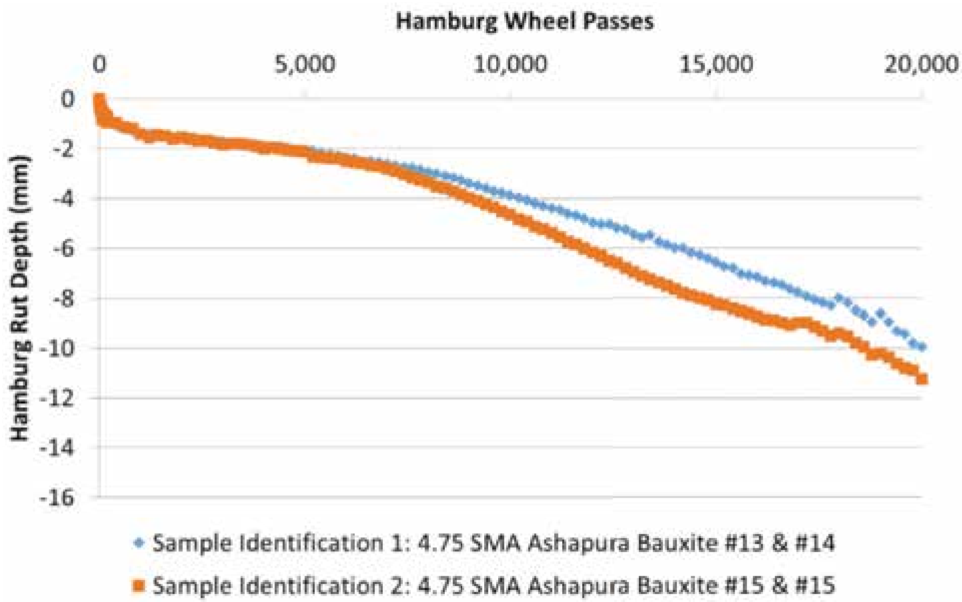

Figure 3 Calcined Bauxite SMA Laboratory Mixture Hamburg Wheel-Tracking Test Results

Figure 4 Calcined Bauxite SMA Mixture Stripping Failure

5.5 Construction

Test Track Section W7 – Micro-Surfacing. This test section was super-elevated in the west curve and immediately preceded HFST Sections W8 and W9. This location permitted a reasonable comparison to the past performance of the HFST sections. Like the HFST sections already in place, Section W7 was subdivided into two 100-foot test sections, one each for the calcined bauxite and sandstone micro-surfacing mixes. Construction of the two 100-foot test sections was performed by Vance Bros. as part of the Test Track and Pavement Preservation construction during the summer of 2015. Both sections were placed the same afternoon.

Test Track Section W3 – SMA. The SMA test section with calcined bauxite was placed later in the 2015 research cycle on April 4, 2017. This section is super-elevated in the west curve. Surface preparation included micro-milling the existing surface, sweeping, and water flushing. A tack coat was applied the same day ahead of the paving and was allowed to cure before paving. It was observed that the rubber tire on the paver picked up some tack during placement, most likely due to residual milling dust on the existing pavement surface. The SMA was placed to a compacted thickness of 0.75 inches.

Test Track Sections W8 and W9 – HFST. As previously noted, these HFST sections were placed during the previous research cycle in April 2011. This is an important factor for monitoring and comparing friction performance. At the beginning of the 2015 research cycle, these sections had already carried 13 million equivalent single axle loads (ESALs) of truck traffic polishing.

5.6 Accelerated Laboratory Friction Testing on Mixtures

Test Track Section W7 – Micro-Surfacing. The study included laboratory evaluation of both field produced micro-surfacing mixtures with the NCAT accelerated laboratory friction process. Three test slabs were prepared for each mixture using mixture obtained from the slurry equipment placing the mixtures on Section W7. The plan was to polish the slab surfaces using the TWPD and test friction and texture with the DFT and CTM. Planned testing increments were 0, 5K, 20K, 70K, and 140K polishing cycles. NCAT had only worked with micro-surfacing slabs on one other study and the protocols for preparing the slab were still being developed.

Two factors made preparation of field slabs challenging. First, placing a thin slurry application to a slab is difficult. The intent of testing laboratory slabs is to compare the dominant coarse aggregate, so the approach was to place a relatively thick lift (1/2-inch) of micro-surfacing on the slabs to avoid the influence of the underlying surface. Second, a micro-surfacing layer requires compaction, which is accomplished by traffic after a normal field placement. The attempt to develop a procedure to compact micro-surfacing was not successful in this study. The slab micro-surfacing material requires some pre-heating (to mimic solar heating) and some compaction with a linear kneading compactor. The laboratory made numerous attempts but was not able to compact the micro-surfacing surface sufficiently to perform the friction polishing procedure. The surface raveled from the torque generated by the tires as they turned on the surface as shown in the right photo of Figure 5. Additional compaction was performed on another test slab and the TWPD protocol was revised to reduce the torque. This reduced the raveling in the surface, as shown in the left photo, but compromised the value of the test. No further accelerated laboratory testing was performed because the results would not have been comparable to previous accelerated laboratory tests on HFST.

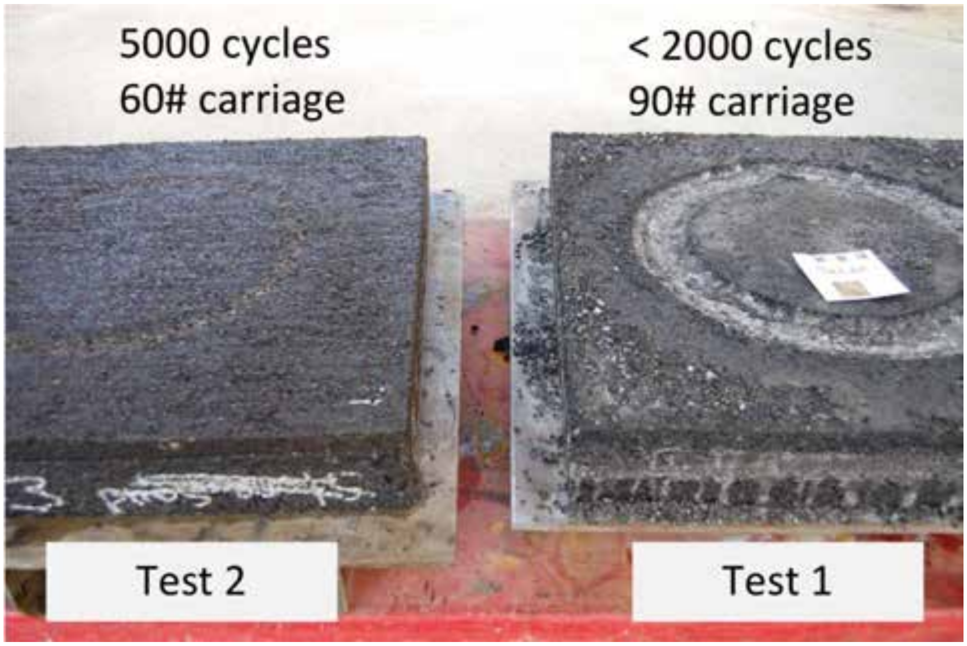

Figure 5 Calcined Bauxite Micro-Surface Surface Raveling After TWPD Polishing

Test Track Section W3 – SMA. Accelerated laboratory friction testing related to this test section provided the opportunity to compare the two stockpiles of calcined bauxite. The initial mix design was prepared with calcined bauxite produced in China with alumina-oxide content above 87%, and that mix design was later adjusted for the India calcined bauxite stockpile (84% alumina-oxide) used for the Test Track installation. Laboratory prepared mix was used to make two slabs of each mixture for accelerated friction testing. Figure 6 displays the results of the laboratory DFT tests and shows that the two stockpiles of calcined bauxite produced similar friction performance curves. Both laboratory polishing terminal friction values are 0.50, which are good terminal values but are lower than laboratory polishing terminal values above 0.75 for HFST (*1, 2*).

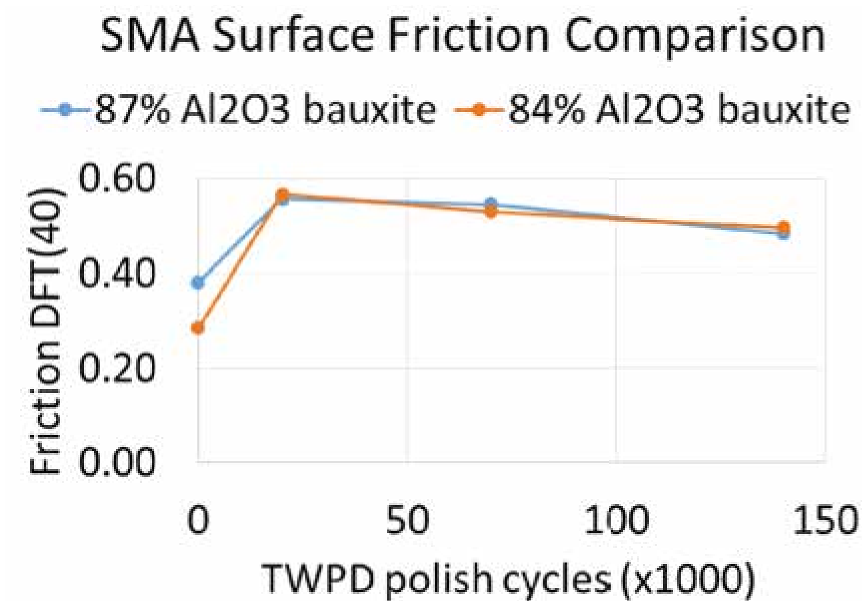

Figure 6 Laboratory Accelerated Friction Performance of Calcined Bauxite SMA Surfaces

5.7 Field Performance Monitoring

The friction performance results must be kept in perspective to the amount of time the surface was polished by the Test Track truck traffic. At the conclusion of the 2015 research cycle, the W8 and W9 HFST sections had been polished 23 million ESALs, the W7 micro-surfacing sections had been polished a full 10 million ESALs, and the W3 SMA had been polished only 3.4 million ESALs. The Gantt chart in Figure 7 provides a summary of the truck traffic polishing on the friction study sections.

Test Section	2011			2012				2013				2014				2015				2016				2017				2018
	Apr-Jun	Jul-Sep	Oct-Dec	Jan-Mar	Apr-Jun	Jul-Sep	Oct-Dec	Jan-Mar	Apr-Jun	Jul-Sep	Oct-Dec	Jan-Mar	Apr-Jun	Jul-Sep	Oct-Dec	Jan-Mar	Apr-Jun	Jul-Sep	Oct-Dec	Jan-Mar	Apr-Jun	Jul-Sep	Oct-Dec	Jan-Mar	Apr-Jun	Jul-Sep	Oct-Dec	Jan-Mar
W8 and W9																												
W7																												
W3 SMA																												

Figure 7 Timeline of Traffic Polishing

Micro-texture – Skid Trailer

A primary measure of safety related to the tire/pavement interface is the friction created between the tire and pavement surface aggregate micro-texture. One measure of friction used on the Test Track is the ASTM E274 locked-wheel skid trailer test owned and operated by the Alabama Department of Transportation (ALDOT). Friction measurements were taken before truck traffic began and approximately every month over the two-year research cycle. The unit of friction measure is SN40R, indicating a measured skid number at 40 mph using a standard ribbed tire, which is more sensitive to pavement surface micro-texture than macro-texture.

Results for this friction testing are shown in Figure 8. Higher SN40R values indicate higher surface friction.

Test Track Section W7 – Micro-Surfacing. Based on the field test results presented in Figure 8, the W7A calcined bauxite section consistently provided higher friction than the W7B Texas sandstone section. Over the last six months of the 2015 research cycle traffic, the average measured friction was SN40R=55 for calcined bauxite micro-surfacing compared to SN40R=50 for the sandstone micro-surfacing. It is valuable to note that both friction values are generally considered very good.

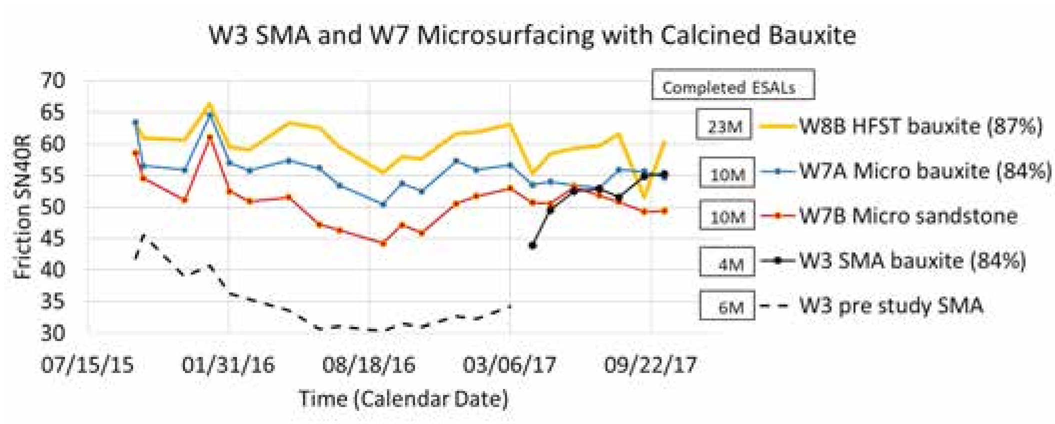

Figure 8 Locked-Wheel Skid Trailer Friction Results

Test Track Section W3 – SMA. The initial friction value of SN40R=44 was lower due to asphalt binder film covering the pavement surface, as expected. The final measure of friction was SN40R=55 and comparable to the W7A calcined bauxite micro-surfacing, but the SMA has only been polished by one-third of the truck traffic (less than 3.4 million ESALs). For comparison purposes, the black dashed line in Figure 8 is a 9.5 mm NMAS granite SMA placed at the beginning of the 2015 Test Track research cycle. Friction of the granite SMA peaked at SN40R=45 once the aggregate was exposed as the asphalt film was removed and dropped to values in the low 30s within about 3 million ESALs as the aggregate surfaces polished.

Test Track Sections W8 and W9 – HFST. As a point of reference, these HFST sections were placed on the track in 2011 and have already been subject to 13 million ESALs at the start of the 2015 test track cycle. W8B, the standard HFST calcined bauxite surface, reduced slightly in friction over the 2015 research cycle. The average fiction was SN40R=61 in the first 12 months of the 2015 cycle and SN40R=60 in the last 12 months. The granite and flint HFST sections are not shown in Figure 8, but their friction performance is worth noting. Section W9 flint surface continued to reduce in friction. The average fiction was SN40R=45 in the first 12 months of the 2015 research cycle and SN40R=41 in the last 12 months. Section W8A granite surface continued to reduce in friction. The average fiction was SN40R=38 in the first 12 months of the 2015 research cycle and SN40R=36 in the last 12 months. All of the reported ending friction values were after a total 23 million ESALs of truck polishing.

Micro-texture – Dynamic Friction Tester

Friction was also measured with the DFT using ASTM E1911-09a *Standard Test Method for Measuring paved Surface Frictional Properties Using the Dynamic Friction Tester*. The proposed test plan scheduled DFT and CTM measurements before traffic taken every week for the first month, every month for the first three months, and every three months over the balance of the two-year trafficking period (0, 1 wk, 2wk, 3wk, 1mo, 2mo, 3mo, 6mo, 9mo, etc). Due to DFT equipment malfunctions and limited access to the test sections, early performance monitoring was not successful. Field tests were conducted, but the accuracy of the data was suspect and discarded. Once the DFT was repaired, testing resumed in May 2016. The reported unit of measure is DFT(40) indicating a friction value measured at 40 km/hr. Other friction values can be reported by the DFT, such as 0, 20, and 60 km/hr, but NCAT friction studies generally reference the 40 km/hr value based on the repeatability of this measure from previous studies. Results for this friction study are shown in Figure 9. Higher DFT(40) values indicate higher surface friction.

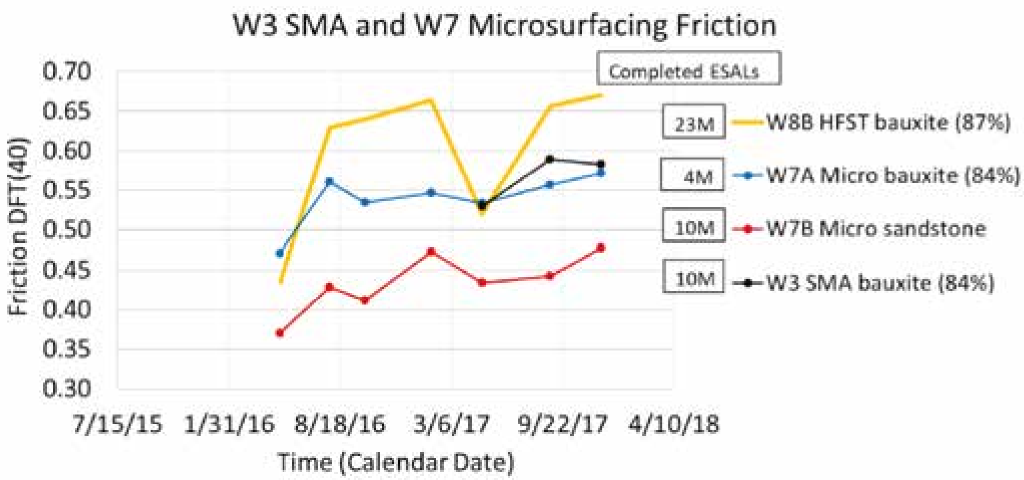

Figure 9 DFT Friction Results for Sections W3 SMA, W7 Micro-Surfacing, and W8B HFST

Test Track Section W7 – Micro-Surfacing. During the eighteen-month shortened testing period after the DFT repair, the DFT(40) friction measurements for the 84% Al_2O_3 calcined bauxite micro-surface held between 0.53 and 0.56 and the sandstone micro-surface maintained DFT friction values between 0.43 and 0.47.

Test Track Section W3 – SMA. During the shortened six-month period of polishing for the SMA section, the DFT(40) maintained values above 0.55. Weekly tests for the initial five weeks of polishing looked at the impact of binder film on the surface. There was a slight increase from an early low value of 0.52 to a later high value of 0.57, then further quarterly tests showed normal test variation. Figure 10 displays the early weekly tests using the DFT and the monthly skid trailer tests. Both sequences of measured friction confirm the early low values and the increase over time.

Test Track Sections W8 and W9 – HFST. As noted earlier, these HFST sections had received over 23 million ESALs of traffic polishing at the end of the 2015 research cycle. The measured friction of the W8B calcined bauxite HFST in May 2016 and April 2017 are unusually low, but there is no evidence in the test data to warrant removing the values. During the final 12 months of the 2015 research cycle, Section W8B, the standard HFST calcined bauxite surface, maintained an average friction of DFT(40)=0.65, discarding the low values. Section W9, the flint surface, maintained an average friction of DFT(40)=0.38 during the final 12 months. Section W8A, granite surface, maintained an average friction of DFT(40)=0.33 during the final 12 months.

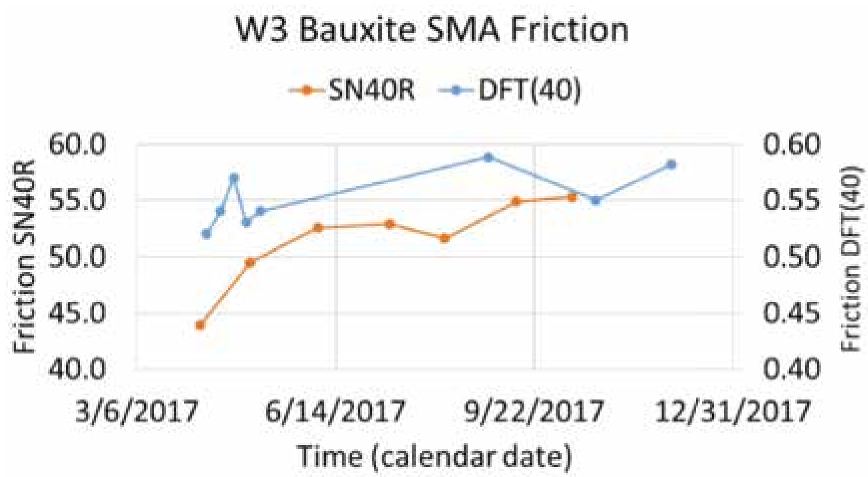

Figure 10 Increase in Friction at the Beginning of Traffic on Section W3

A second component of safety related to the tire/pavement interface is the macro-texture of the pavement surface. Testing friction with the skid trailer using a ribbed tire and DFT predominantly measure the aggregate polishing characteristics, commonly defined as the micro-texture. Safety of the pavement surface is also a function of the surface macro-texture, which provides channels for the water to pass through to keep the tire in contact with the surface in wet surface conditions, thus reducing the potential for hydroplaning. The Test Track studies measured surface macro-texture with two different devices.

Macro-texture – High Speed Laser

One method was a high-speed profile laser mounted on a van equipped for automated pavement condition testing. Measurements with this system were taken every week when the pavement surface was dry. The laser creates a data set of measurements taken every 0.4 mm with a laser spot diameter of 2.0 mm and vertical resolution of 0.05 mm. The unit of measure is mean profile depth (MPD) computed for each 100 mm of linear measure and is expressed as average profile depth in millimeters. MPD results for this study are shown in Figure 11.

Test Track Section W7 – Micro-Surfacing. Both micro-surfacing sections performed similar with respect to surface macro-texture. Immediately after the micro-surfacing was placed, the surface measured an MPD of 0.38 to 0.39 mm. The surfaces quickly increased to an MPD of 0.48 to 0.49 mm after the first two months, then gradually dropped to 0.30 over 10 months, then increased and remained steady at an MPD of 0.33 to 0.37 mm. The changes in macro-texture likely reflect the early compaction of the micro-surfacing from the traffic, further embedment of the coarse aggregate particles, and minor losses of mastic, but the cause for macro-texture depth change was not a part of the study, so these causes were not specifically evaluated.

Test Track Section W3 – SMA. The 4.75 mm NMAS calcined bauxite SMA was placed in April 2017 and the previous surface was a 9.5 mm NMAS SMA placed at the beginning of the 2015 research cycle. The previous 2015 SMA surface had an MPD of 0.40 mm prior to milling. The new SMA with calcined bauxite started with an MPD of 0.30 mm but dropped below 0.20 mm after three months and continued to vary from 0.15 to 0.20 mm.

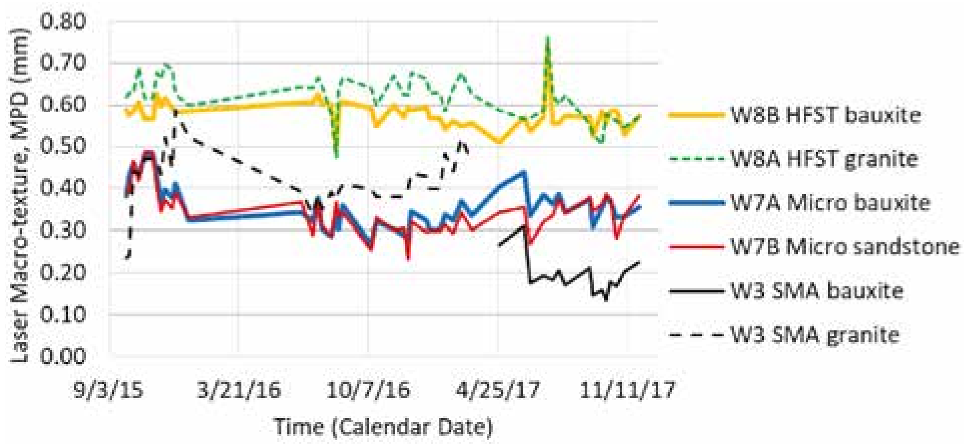

Figure 11 High Speed Laser Macro-Texture Results

Test Track Sections W8 and W9 – HFST. As a comparison, the surface macro-texture of section W8B, the HFST standard with calcined bauxite, started the 2015 research cycle with an MPD of 0.59 mm and reduced to 0.57 mm at the end of the cycle. Section W8A started at an MPD of 0.62 mm and ended at 0.57 mm, and section W9 (not shown in the figure) started at 0.54 mm and ended at 0.50.

Macro-texture – Circular Texture Meter

The second device used to measure surface macro-texture was the CTM (ASTM E2157). The CTM uses a laser to measure the profile of the surface along the same circular path tested by the DFT. The speed of the CTM creates a data set of measurements taken every 0.87 mm with a laser spot diameter of 0.07 mm and vertical resolution of 0.003 mm. This is a higher resolution measurement than the van's high-speed laser. Measurements were taken every three months when the pavement surface was dry. The macro-texture value is computed and recorded as mean profile depth (MPD) expressed in millimeters. Results from the CTM tests are shown in Figure 12.

Test Track Section W7 – Micro-Surfacing. The test plan proposed for DFT and CTM measurements included before traffic, every week for the first month, every month for the first three months, and every three months over the balance of the two-year conditioning period (0, 1wk, 2wk, 3wk, 1mo, 2mo, 3mo, 6mo, 9mo, etc). Due to rain events and limited access to the test section, frequent testing early in the research cycle was limited. As expected, the pre-traffic surface macro-texture measured by the CTM was high, 0.94 mm MPD for the W7A micro-surfacing with calcined bauxite and 1.16 mm for the W7B micro-surfacing with sandstone. After two weeks of traffic, the values dropped to 0.69 mm and 0.95 mm, respectively. Over the two-year traffic period, the calcined bauxite surface maintained an average MPD of 0.74 mm and the sandstone averaged 0.95 mm. The difference in the macro-texture between the surfaces is the size of the aggregate used in the mixture. The calcined bauxite micro-surface was dominated by 25% aggregate passing the #4 sieve and retained on the #8 sieve, while the sandstone micro-surface was dominated by 10% aggregate passing the 3/8-in. sieve and retained on the #4 sieve and 37% aggregate passing the #4 sieve and retained on the #8 sieve.

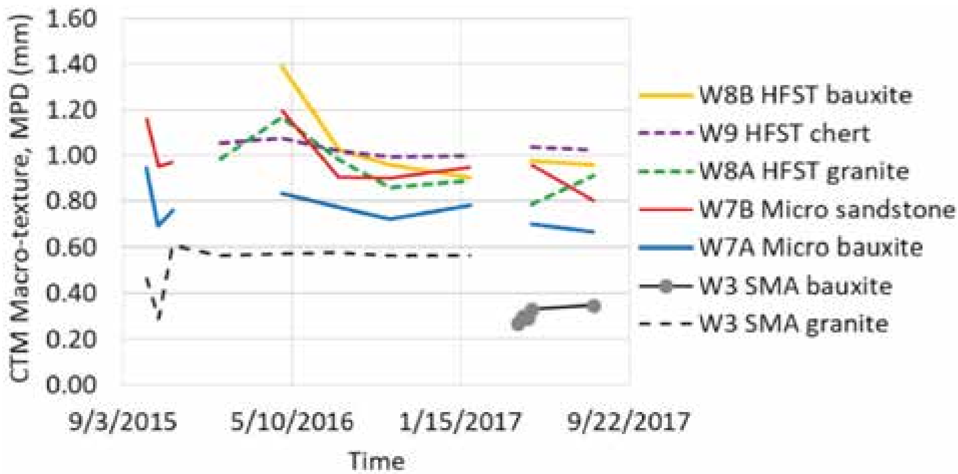

Figure 12 CTM Macro-Texture Results

Test Track Section W3 – SMA. The 4.75 mm NMAS SMA with calcined bauxite was placed in April 2017. The previous surface was a 9.5 mm NMAS SMA with granite placed at the beginning of the 2015 research cycle and had an MPD of 0.58 mm prior to milling. The new SMA with calcined bauxite started with an MPD of 0.27 mm, quickly increased to 0.33 mm during the first month, and leveled out around 0.35 mm. An increase in CTM measured macro-texture was noted in Test Track Section S4 (a thin-lift dense-graded mix) over the entire 2015 research cycle. If the W3 SMA macro-texture increases over time, the hydroplaning risk will reduce.

Test Track Sections W8 and W9 – HFST. As a comparison, the surface macro-texture of Section W8B, the HFST standard with calcined bauxite, MPD ranged between 0.90 and 1.00 mm and Section W8A, HFST with granite, maintained similar MPD values. The CTM macro-texture of Section W9, HFST with flint, ranged between an MPD of 1.05 and 1.00 mm.

Other general performance parameters for the three sets of friction test sections are documented below. Sections W7, W8, and W9 were thin surface treatments that reflect the condition of the underlying pavement. Section W3 was a mill and inlay and reflects the quality of the construction of the test section.

Test Track Section W7 – Micro-Surfacing. The international roughness index (IRI), of Section W7A remained steady at 1.3 m/km (82 in/mi) and Section W7B started at 1.3 m/km (82 in/mi) and increased to 1.6 m/km (101 in/mi). The measured rutting was a nominal 0.3 mm for W7A and 0.6 mm for W7B, both very low.

Test Track Section W3 – SMA. The IRI of Section W3 increased from 1.5 m/km to 2.0 m/km (95 to 127 in/mi) when the SMA was placed in April 2017 and remained at 2.0 m/km (127 in/mi) during the final 7 months of traffic. The measured rutting for W3 was 0.7 mm after the traffic began on the section and did not change.

Test Track Sections W8 and W9 – HFST. The IRI values of the HFST sections (W8A, W8B, and W9) were 4.1 m/km (260 in/mi), 3.2 m/km (203 in/mi), and 2.7 m/km (171 in/mi), respectively. The measured rutting on these same sections were 2.0 mm, 2.0 mm, and 5.0 mm, respectively.

5.7 Cost Comparison

The objective of this study compared the friction performance of three pavement surfaces using calcined bauxite to enhance the friction. This section briefly discusses a general cost comparison between the pavements in this study. The key factors in the comparison are the cost of materials and cost of construction. The cost of materials must take into account the amount of calcined bauxite required per unit of pavement surface area as well as the unit cost of each material itself. A HFST uses a very high cost calcined bauxite aggregate and even higher cost polymer resin binder to secure the aggregate to the pavement surface. The enhanced friction micro-surfacing replaced 50% of the aggregate with calcined bauxite and used a more costly highly polymer modified CSS-1HP emulsion. The enhanced friction SMA replaced 40% of the aggregate with calcined bauxite and required a higher binder content due to the smaller 4.75 mm NMAS mixture size. The amount of calcined bauxite applied is 12 to 15 lb/sy for a HFST and 6 to 8 lb/sy for micro-surfacing. The SMA requires 14 to 18 lb/sy of calcined bauxite to account for the larger lift thickness.

The cost of construction varies by the availability of contractors and the size of a project. There are a small number of specialty contractors that perform HFST construction. The availability of micro-surfacing contractors depends on the amount of preservation construction in a region of the country. All areas of the country have conventional asphalt contractors, but not all contractors have experience placing thin 4.75 mm mixtures. The unit bid cost for any of the studied friction surfaces will reflect local cost history plus the added cost of the calcined bauxite, use of a modified asphalt binder, and increased asphalt binder content. In general terms, the unit bid cost of HFST is expected to be \$20 to \$30 per square yard, the unit cost of a SMA enhanced friction thin lift may increase to \$6 to \$8 per square yard, and the unit cost of an enhanced friction micro-surfacing may increase to \$4 to \$5 per square yard.

In addition to the unit cost of materials and construction, an understanding of the probable performance life of each friction surface is needed to compare the cost-effectiveness of these surfaces. A HFST provides very good friction and high macro-texture, as demonstrated by five years of Test Track data. The expected performance life of HFST (7 to 10 years) is controlled by the aging of the polymer resin binder which impacts the ability to retain the calcined bauxite, as experienced on earlier Test Track Sections E2 and E3 data. An asphalt 4.75 mm SMA with calcined bauxite is predicted to maintain good friction but has low macro-texture, as demonstrated by the trend of eight months of Test Track data. The expected performance life of the 4.75 mm SMA with calcined bauxite may be controlled by the aging of the asphalt binder. A conservative estimate of the service life of this SMA is 10 to 12 years based on typical performance lives of SMA surfaces (*3*). A micro-surfacing with calcined bauxite is predicted to maintain good friction and moderate macro-texture, as demonstrated by the trend of two years of Test track data. The performance life of micro-surfacing with calcined bauxite is also expected to be controlled by the adhesion of the modified asphalt binder to retain the calcined bauxite. The predicted performance life of micro-surfacing with calcined bauxite is four to six years based on typical performance of micro-surfacing. Better cost-effectiveness decisions require more regional long-term pavement performance and friction performance data.

5.8 Conclusions

The following conclusions are provided for the pavement performance data presented above.

- Micro-surfacing with 50% calcined bauxite (84% Al_2O_3), Section W7A, performed well based on two-year terminal measured friction (SN40R=55, DFT(40)=0.55) and macro-texture (CTM MPD=0.70 mm). Micro-surfacing with sandstone, Section W7B, also performed well based on two-year terminal measured friction (SN40R=50, DFT(40)=0.44) and macro-texture (CTM MPD=0.90 mm). However, these sections have lower friction and macro-texture values than the five-year performance of the standard HFST with calcined bauxite (87% Al_2O_3), Section W8B.
- SMA with 40% calcined bauxite (84% Al_2O_3), Section W3, is performing well based on measured friction, but its surface macro-texture value was low, which increases risk for hydroplaning. This conclusion must be kept in perspective since the test section has been exposed to only eight months (3.4 million ESALs) of trafficking.
- Friction measured with the DFT compared well with friction measured with the locked-wheel skid trailer using a ribbed tire. The surfaces rank the same and the relative difference between surfaces is similar, but the DFT showed more sensitivity to friction aggregate differences.
- Macro-textures measured with the CTM and with the high-speed profiler van were very different for all surfaces. The CTM measured expected change in macro-texture over time and between the surfaces but the high-speed profiler van results were significantly lower and appeared to be less sensitive to changes after traffic.
- General conditions (smoothness and rutting) of the underlying pavement surface are reflected in the HFST and micro-surfacing surfaces. Smoothness for the SMA was not as good as expected (higher IRI), but rutting resistance was very good, as expected.
- Pavement surface friction studies must recognize the importance of both micro-texture (friction) and macro-texture (texture) to define the safety of the pavement surface. More research is needed to study the combined impact of both surface features on reduction of crash rates. For example, is there a measureable reduction in crash rate when the micro-texture is greater than 50 SN40R and the macro-texture is greater than 0.60 mm MPD? Figure 13 displays the combined value of micro-texture friction and surface macro-texture for the key test sections in this study. These data points must be

kept in perspective as it relates to the amount of polishing by the truck traffic each section has received.

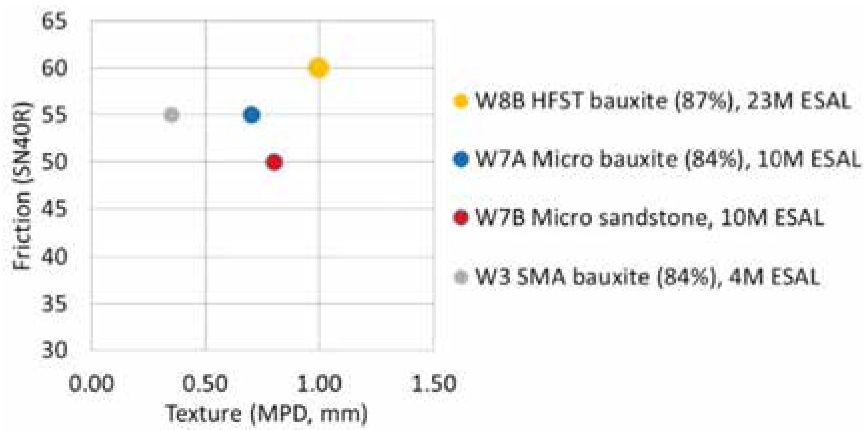

Figure 13 Comparison of Friction Micro-Texture and Surface Macro-Texture Results

5.9 References

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CHAPTER 6 FLORIDA DEPARTMENT OF TRANSPORTATION CRACKING STUDY

6.1 Introduction

As many state agencies have successfully mitigated rutting as a primary cause of pavement deterioration, more emphasis has been placed on identifying properties of mixtures that may influence the overall durability of the pavement structure. One such distress that affects durability is top-down cracking, which has been documented worldwide. The Florida Department of Transportation and the University of Florida were some of the first to recognize the widespread nature of this distress with over 90% of cracking in the state of Florida categorized as top-down (*1*).

As the name implies, these cracks form at the top of the pavement structure and are load-related, as they tend to originate in the wheelpath. However, Roque et al. note the complex interaction of load, thermal, and aging effects as contributing to top-down cracking. They further explain that after reviewing a wide variety of material characteristics, there is not one single mixture property that could reliably discern between acceptable and poor cracking performance (*2*).

6.2 Objective and Scope

The objective of this experiment was to determine which asphalt mixtures were more prone to surface cracking. A secondary objective of this work was to characterize the mixtures' properties in the laboratory to determine which cracking tests might most successfully predict cracking resistance. To complete this research, four mixtures were placed in 100-foot test strips in sections E7 and E8 during the reconstruction of the 2015 test cycle. The mixtures varied in terms of binder type (PG grade) and recycled material content.

6.3 Mix Design and Construction

Four mixtures were designed at the NCAT laboratories for this experiment. All mixtures utilized asphalt binder modified with styrene-butadiene-styrene (SBS) and used a similar aggregate skeleton. The change in the RAP content appears to have influenced the combined gradation, in some cases by a magnitude of 10% (i.e. percent passing #8 sieve, Table 1). The main difference among all four mixtures is the amount of reclaimed asphalt pavement (RAP), which varies from 20% to 30%. Mixture gradations, base binder grades, volumetrics, and construction data for all four mixtures are provided in Table 1. The performance grade (PG) of the binder for section E8B was originally intended to be a polymer modified PG 58-28 binder. However, the final product was a PG 64-28 after the polymer modification. At the time of the study, no state in the region was specifying a polymer modified PG58-28; therefore, the closest available was the PG 64-28 polymer modified binder.

Mixtures E8-1A and E8-1B had the same aggregate structure with a different binder. In this study, volumetric design was performed on E8-1A using the PG 76-22 only; no verifications were made on the polymer modified PG 58-28 mixture because it was not available to NCAT prior to construction. The Cantabro test (ASTM D 7064) was performed on design samples at three binder contents (5.0, 5.5, and 6.0%) and quality control samples. Table 1 shows calculated Cantabro percent of weight loss at optimum binder content for design samples.

Table 1 Florida DOT Mixture Characteristics

Mix Design Parameters	**E7-1A**	**E7-1B**	**E8-1A**	**E8-1B**	**E7-1A**	**E7-1B**	**E8-1A**	**E8-1B**
Design Method	Superpave							
Compactive Effort	100 Gyrations							
Results	**Design Data**				**Quality Control Data**			
Binder Grade	76-22	76-22	76-22	58-28	76-22	76-22	76-22	64-28
$P_{3/4''}$, %	100	100	100	100	100	100	100	100
$P_{1/2''}$, %	97	97	97	97	98	98	95	97
$P_{3/8''}$, %	85	86	86	86	90	92	87	91
$P_{\#4}$, %	56	58	60	60	54	58	56	63
$P_{\#8}$, %	42	45	48	48	40	44	44	50
$P_{\#16}$, %	33	35	38	38	33	36	37	40
$P_{\#30}$, %	21	23	24	24	24	25	26	27
$P_{\#50}$, %	11	12	13	13	13	13	14	13
$P_{\#100}$, %	7.0	7.0	8	8	7.0	7.0	8.0	8.0
$P_{\#200}$, %	4.5	4.9	5.4	5.4	4.1	4.3	4.6	5.1
Modifier Type	SBS							
Total Binder Content, %	5.2	5.2	5.1	5.1	4.8	5.0	5.0	5.0
Effective Binder Content, %	4.6	4.5	4.4	4.4	4.2	4.4	4.3	4.3
% RAP	20.9	26.4	31.8	31.8	20.0	23.9	28.9	28.8
Air Voids, %	4.0	4.0	4.0	4.0	4.3	4.0	3.5	4.5
Voids in Mineral Aggregate, %	14.6	14.5	14.4	14.4	14.0	14.0	14.0	14.0
Voids Filled with Asphalt, %	73	72	72	72	69	72	74	69
Cantabro % Loss	5.89	6.59	6.30	NA	5.41	5.45	4.55	3.49
Tensile Strength Ratio, %	96.3	97.8	92.4	NA				
Production/Construction Data								
Lift Thickness, in.	1.5				1.8	1.9	1.8	1.7
Type of Tack Coat					NTSS-1HM			
Undiluted Target Tack Rate/residual, gal/sy					0.08/0.05			
Temperature at Plant, °F					340	340	340	340
Average Mat Compaction, %					93.9	91.6	92.5	93.5

6.4 Laboratory Testing

While the field experiment was being conducted, materials (plant-produced loose mix and asphalt binder) that had been sampled during construction were taken back to the NCAT laboratory for evaluation. The binder was tested using the performance grade, multiple stress creep recovery (MSCR) specifications, and frequency sweeps. The mixtures were evaluated for cracking potential using five different tests:

- Semi circular bend test (SCB-LTRC), Louisiana Method – Draft;
- Illinois flexibility index test (I-FIT), Illinois Method – Draft;
- Energy ratio, Florida – Draft;
- Overlay test (OT) – Tex-248-F; and
- Overlay test (OT) – NCAT modified.

Additional laboratory performance evaluations included rutting using the Hamburg wheel-track test and the dynamic modulus test. The Hamburg test also gives a measure of the moisture resistance of these mixes.

Binder Characterization

The virgin binders for the asphalt mixtures were sampled from the tank at the plant and tested in the NCAT binder laboratory to determine the PG grade in accordance with AASHTO M 320. The blended asphalt binders were extracted and recovered using AASHTO T 164 Method A (using Trichloroethylene - TCE) and ASTM D 5404 before AASHTO M 320 was conducted. The test results for these two procedures are shown in Tables 2 and 3. Table 4 provides the test requirements from AASHTO M332, non-recoverable creep compliance. In addition to testing the virgin binders and extracted binders, the PG properties of the RAP binder were determined.

The results indicated a significant increase in the stiffness of both binders when RAP was added to the mix. Both critical high and low temperatures of the PG 76-22 binder were increased to 94°C and -16°C, respectively. Although the same trend was observed for the PG 64-28 binder, the critical low temperature was more affected with an increase in temperature from -28°C to -16°C.

Table 2 Florida DOT Cracking Study Performance Grades

Mix / Material	Extracted	% RAP	T_{crit}, °C					True High PG	True Low PG	PG
			High Original	High RTFO	Interm.	Low S	Low m			
76-22	No	NA	80.2	80.9	21.2	-29.4	-26.4	80.2	-26.4	76-22
64-28	No	NA	67.5	67.5	11.0	-35.6	-33.2	67.5	-33.2	64-28
E7-1A	Yes	20	97.1	94.0	24.1	-26.7	-19.5	94.0	-19.5	94-16
E7-1B	Yes	25	100.7	97.6	23.1	-28.5	-19.3	97.6	-19.3	94-16
E8-1A	Yes	30	99.6	96.2	22.7	-25.8	-19.1	96.2	-19.1	94-16
E8-1B	Yes	30	92.0	89.1	21.1	-30.2	-18.5	89.1	-18.5	88-16
RAP	Yes	100	115.4	112.0	30.5	-22.9	-13.8	112.0	-13.8	112-10

The increase in stiffness for all test sections is noticed in the non-recoverable creep values, J_{nr}, and the final AASHTO M 332 grade. The J_{nr} values of all sections are much smaller than the PG 76-22 and PG 64-28 binders, which suggests a more rut resistant binder. This is reflected in the final AASHTO M 332 grade, especially for the PG 64-28 binder. While the PG 64-28 binder was labeled "*H*" for high trafficking, all mixtures with recycled binders were designated as "*E*" binders (extremely high traffic).

Table 3 Florida DOT Cracking Study MSCR Results at 64°C

Mix / Material	Extracted	Avg % Recovery		Avg J_{nr}, kPa^{-1}		Diff, % Recovery	Diff, % J_{nr}	M332 Grade Suffix
		100 Pa	3200 Pa	100 Pa	3200 Pa			
76-22	No	76.46	68.89	0.133	0.179	9.9	34.6	E
64-28	No	70.34	53.77	0.617	1.067	23.6	72.8	H
E7-1A	Yes	87.65	81.65	0.023	0.026	6.8	13.1	E
E7-1B	Yes	83.95	77.77	0.028	0.035	7.4	21.1	E
E8-1A	Yes	81.64	76.21	0.022	0.028	6.7	29.6	E
E8-1B	Yes	81.54	77.37	0.017	0.021	5.1	22.9	E

Table 4 Requirements for Non-Recoverable Creep Compliance (AASHTO M 332)

Traffic Level	Max $J_{nr3.2}$ (kPa^{-1})	Max J_{nrdiff} (%)
Standard Traffic "S" Grade	4.5	75
Heavy Traffic "H" Grade	2.0	75
Very Heavy Traffic "V" Grade	1.0	75
Extremely Heavy Traffic "E" Grade	0.5	75

Dynamic Modulus (AASHTO TP 79)

Dynamic modulus (E*) testing was conducted in accordance with AASHTO TP 79 on the four previously described mixtures. This testing was performed using an IPC Global Asphalt Mixture Performance Tester (AMPT). Specimens were produced in accordance with AASHTO PP 60. A Pine Instruments gyratory compactor was used to compact specimens to 150 mm in diameter and 175 mm in height. These samples were then cored using a 100 mm core drill and trimmed to 150 mm in height. The air voids for these cut specimens were 7 ± 0.5%.

To provide the necessary information for mechanistic-empirical (M-E) pavement analyses, the three samples of the four completed mix designs were tested using three temperatures (4, 20, and 45°C) and three frequencies (10, 1, and 0.1 Hz) in an unconfined state. The mixes were also tested at the 0.01 Hz frequency at the high test temperature. This testing produced a data set for generating master curves for all four mixtures using the procedure outlined in AASHTO PP 61.

The test data were checked to ensure they met the data quality and within-lab repeatability requirements of AASHTO TP 79. The general form of the mastercurve equation is shown as Equation 1. As mentioned, the dynamic modulus data are shifted to a reference temperature 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 mastercurve equation, shown as 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. The P_c term, Equation 6, is simply a variable required for Equation 5. A limiting binder modulus of 1 GPa is assumed for this equation. Non-linear regression is then conducted using the 'Mastersolver.exe' program to develop the coefficients for the mastercurve equation. These curves are expected to have a Se/Sy term of less than 0.05 and an R^2 value of greater than 0.99 according to AASHTO TP 79.

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

$$logf_r = log\ f + \frac{\Delta E_a}{19.14714}\left[\frac{1}{T} - \frac{1}{T_r}\right] \quad (2)$$

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

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

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

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

where

$\lvert E^*\rvert$	=	dynamic modulus (psi);
Max	=	limiting maximum modulus, psi;
f	=	loading frequency at the test temperature (Hz);
f_r	=	reduced frequency at the reference temperature (Hz);
$\alpha, \delta, \beta, \gamma$	=	regression coefficients;
ΔE_a	=	activation energy (treated as a fitting parameter);
T	=	test temperature, °K;
T_r	=	reference temperature, °K;
a(T)	=	The shift factor at temperature, T;
VMA	=	Voids in mineral aggregate, %; and
VFA	=	Voids filled with asphalt, %.

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. This is particularly useful for mixtures containing RAP or recycled asphalt shingles (RAS) where the degree of binder blending is unknown. For the four master curves (Figure 1), the mixture with the PG 64-28 binder was the "softest" at all temperatures and frequencies. On the other hand, the mixture with PG 76-22 and 25% RAP showed higher dynamic moduli at almost all temperatures and frequencies. Table 5 shows the master curve coefficients and regression parameters for all mixtures.

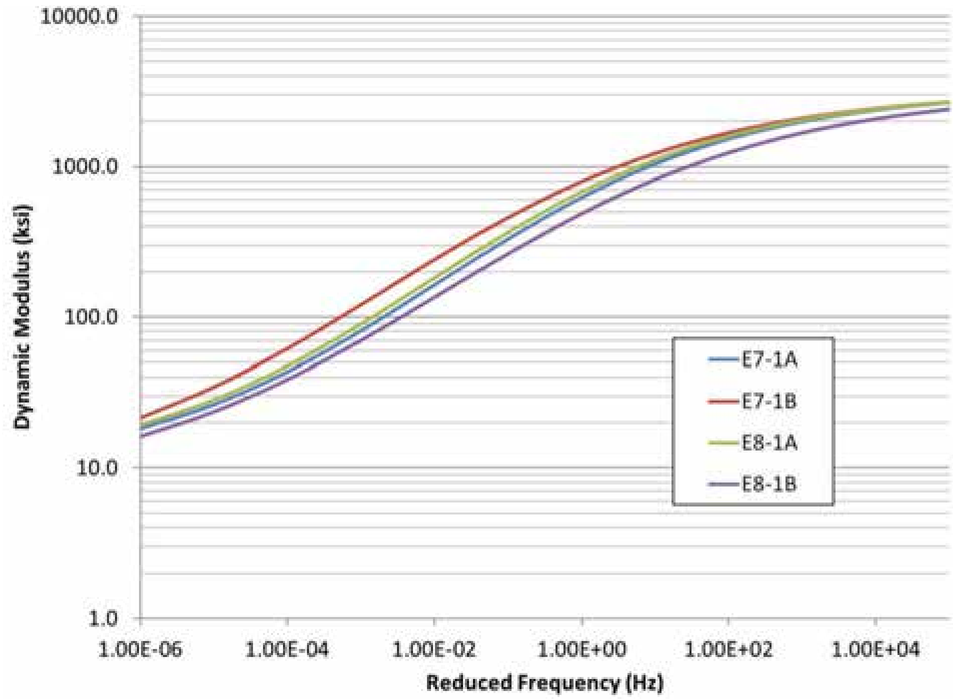

Figure 1 Florida DOT Cracking Study Master Curves

Table 5 Master Curve Coefficient and Regression Parameters

Mix	Max E* (Ksi)	Min E* (Ksi)	Beta	Gamma	EA	R^2	Se/Sy
E7-1A	3211	8.9	-0.950	-0.488	199954	0.998	0.031
E7-1B	3190	7.2	-1.215	-0.456	204156	0.998	0.028
E8-1A	3185	9.1	-1.025	-0.490	202090	0.998	0.034
E8-1B	3212	6.8	-0.819	-0.439	206247	0.998	0.033

The results of statistical analyses conducted at the tested temperatures and frequencies are shown in Table 6. To assess statistical differences in E* results, the general linear model (GLM) (α = 0.05) was conducted on the test data measured at a frequency of 10, 1.0, and 0.1 Hz and temperatures of 4, 20, and 40°C. The Tukey-Kramer test (α = 0.05) was used to determine where these statistical differences occurred and how the mixtures grouped within each project. For each temperature-frequency combination, mixtures with the same letter were not statistically different. At low temperatures, E* values of the mixture with the PG 64-28 binder were statistically different (lower) that the other three mixtures. At intermediate temperatures, E* values for the 25% RAP, PG 76-22 mixture was statistically higher and the E* values for the 30% RAP, PG 64-28 mixture tended to be statistically lower. At high temperatures, E* values for the 25% RAP, PG 76-22 mixture tended to be statistically higher than the other mixtures except at a frequency of 0.01 Hz.

Table 6 E* Statistical Grouping (Tukey-Kramer Test at α = 0.05)

Mix ID	4°C - 10 Hz	4°C - 1 Hz	4°C - 0.1 Hz	
E7-1B - FL 25% RAP, PG 76-22	A	A	A	
E8-1A - FL 30% RAP, PG 76-22	A	A	A B	
E7-1A - FL 20% RAP, PG 76-22	A	A	B	
E8-1B - FL 30% RAP, PG 64-28	B	B	C	
Mix ID	20°C - 10 Hz	20°C - 1 Hz	20°C - 0.1 Hz	
E7-1B - FL 25% RAP, PG 76-22	A	A	A	
E8-1A - FL 30% RAP, PG 76-22	A B	B	B	
E7-1A - FL 20% RAP, PG 76-22	B	B	B C	
E8-1B - FL 30% RAP, PG 64-28	C	C	C	
Mix ID	45°C - 10 Hz	45°C - 1 Hz	45°C - 0.1 Hz	45°C - 0.01 Hz
E7-1B - FL 25% RAP, PG 76-22	A	A	A	A
E8-1A - FL 30% RAP, PG 76-22	B	B	B	A B
E7-1A - FL 20% RAP, PG 76-22	B C	B	B	B
E8-1B - FL 30% RAP, PG 64-28	C	B	B	B

Hamburg Wheel-Track Test (AASHTO T 324-16)

Hamburg wheel-track testing was performed to determine the rutting and stripping susceptibility of the four mixtures. Samples were prepared in accordance with AASHTO T 324. For each mix, three replicates were tested. The specimens were originally compacted to a diameter of 150 mm and a height of 115 mm. These specimens were then trimmed so that two specimens with a height between 38 mm and 50 mm were cut from the top and bottom of each gyratory compacted specimen. The air voids on these cut specimens were 7 ± 1%.

The samples were tested under a 158 ± 1 lb wheel load for 10,000 cycles (20,000 passes) while submerged in a water bath maintained at a temperature of 50°C. During testing, rut depths were measured by a linear variable displacement transducer (LVDT), which recorded the relative vertical position of the load wheel after each load cycle. After testing, these data were used to determine the point at which stripping occurred in the mixture under loading and the relative rutting susceptibility of those mixtures. These data show the progression of rut depth with number of cycles. Two tangents are evident from this curve: the steady-state rutting portion of the curve and the portion of the curve after stripping. The intersection of these two curve tangents defines the stripping inflection point of the mixture.

The mixtures did not show any signs of stripping; therefore, it is not expected that any of the mixtures will be susceptible to moisture damage. Additionally, all four mixtures showed good resistance to rutting, as 12.5 mm is a common rutting threshold for this test. The mixture with 20% RAP had the most rutting numerically at 2.3 mm; however, an ANOVA (p-value = 0.442 > α = 0.05) showed no statistical differences between the performance of the four mixtures.

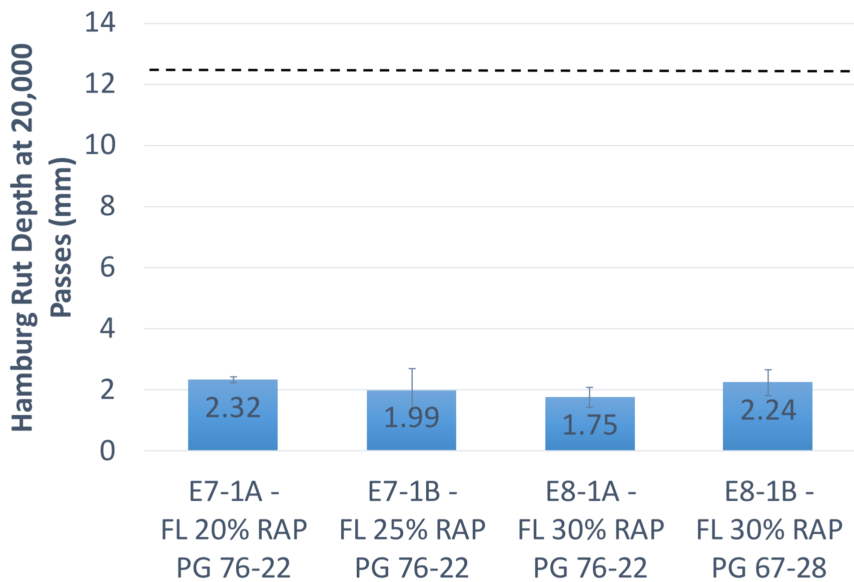

Figure 2 Florida DOT Cracking Study Hamburg Results

Overlay Test

The overlay test was performed in accordance with Tex-248-F using the OT kit designed for the IPC Global AMPT. For this test, samples with a 150 mm diameter and 125 mm target height were produced using the Superpave gyratory compactor. Two test specimens from each gyratory sample were trimmed to the following dimensions: 150 mm long by 76 mm wide by 38 mm tall. After trimming, four replicates with air voids between 6 and 8% were tested at 25°C in controlled displacement mode. Loading occurs when a movable steel plate attached to the asphalt specimen slides away from the other plate, which remains fixed. Loading occurs at a rate of one cycle every 10 seconds with a sawtooth waveform. The maximum load the specimen resists in controlled displacement mode is recorded for each cycle. The test continues until the sample fails, which is defined as 93% reduction in load magnitude from the first cycle. Tex-248-F specifies a maximum opening displacement of 0.025 inches. There is no definitive pass/fail criterion for the OT, with minimum recommended cycles to failure at the above parameters ranging between 150 and 700 depending on the type of mixture tested (*3, 4, 5*). An NCAT modified method of this test was also performed with a higher frequency (1 Hz), modified strain rate and modified failure criteria (Figure 3) where the peak value of the normalized load multiplied by number of cycles (NL x Cy) indicates the point of failure.

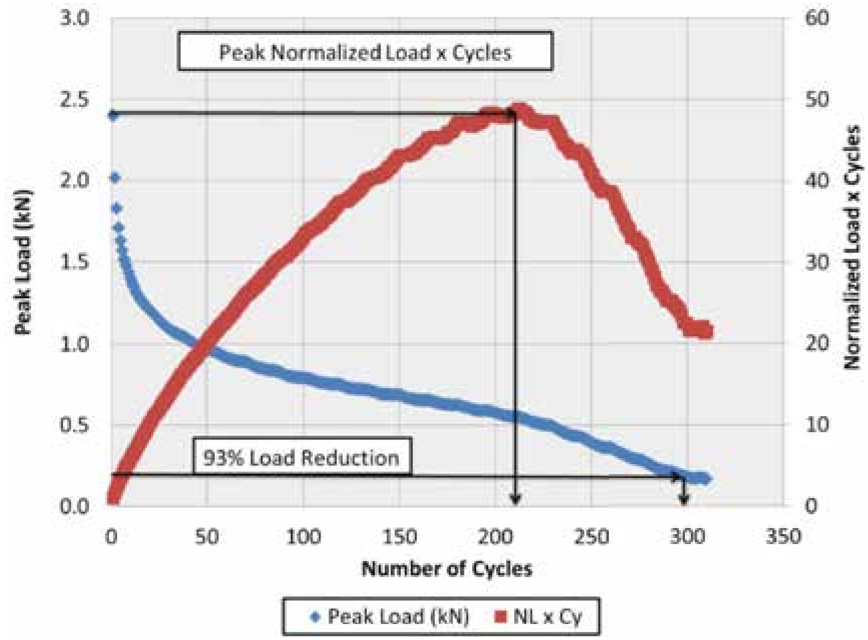

Figure 3 NCAT Modified OT Example Data

Figures 4 and 5 provide the OT results for the four mixtures. None of the mixtures performed well in this test, highlighting the need for state-specific parameters in performance tests to match field performance. Statistical analyses indicated that all mixtures were not statistically different from each other when following the Texas OT standard practice. However, when the modified NCAT method was used, the 25% RAP PG 76-22 (E7-1B) mixture was the only statistically different mixture and had the lowest number of cycles to failure.

Figures 4 and 5 suggest that the lack of statistical significance is largely due to the relatively high variability noted with mixture E8-1B compared to the others. Depending upon what the error bars represent the other three mixtures may very well be statistically different with either of the test methods, potentially.

The Critical Fracture Energy (crack initiation parameter) and Crack Resistance Index (crack propagation parameter) were determined from the TO test results according to Tex-248-F-2017 test procedure. These results are shown and compared against the number of cycles to failure in Table 7. Both cracking parameters were able to identify mix E7-1B as the only statistically different mixture with the highest initiation and propagation parameters. This indicates that mix E7-1B would not permit easily the initiation of a crack but it would not attenuate the rate of the propagation of the crack after it is initiated as well as the other mixtures. The overall variability of the results when computing these parameters was significantly decreased as observed with the coefficient of variability (COV), especially for the crack initiation parameters with COV values below 11%.

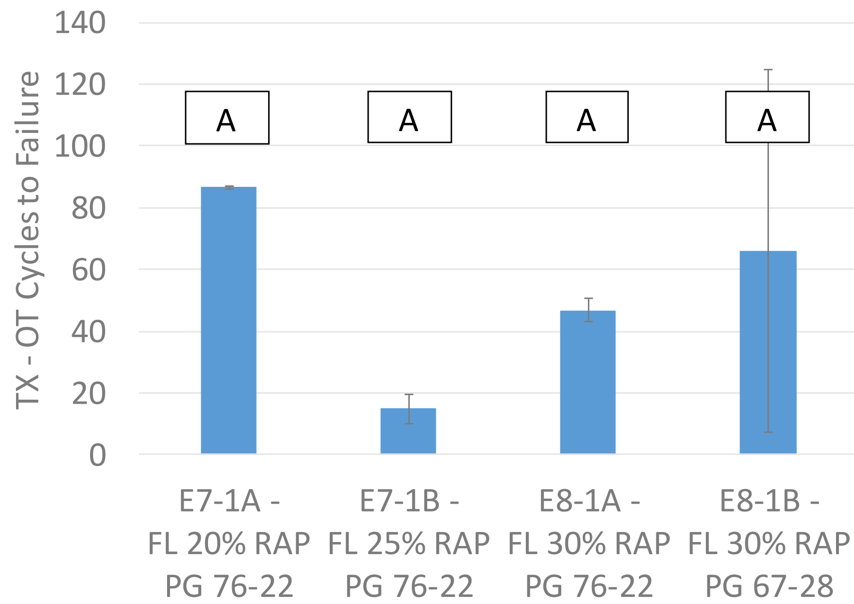

Figure 4 Texas Overlay Tester Results

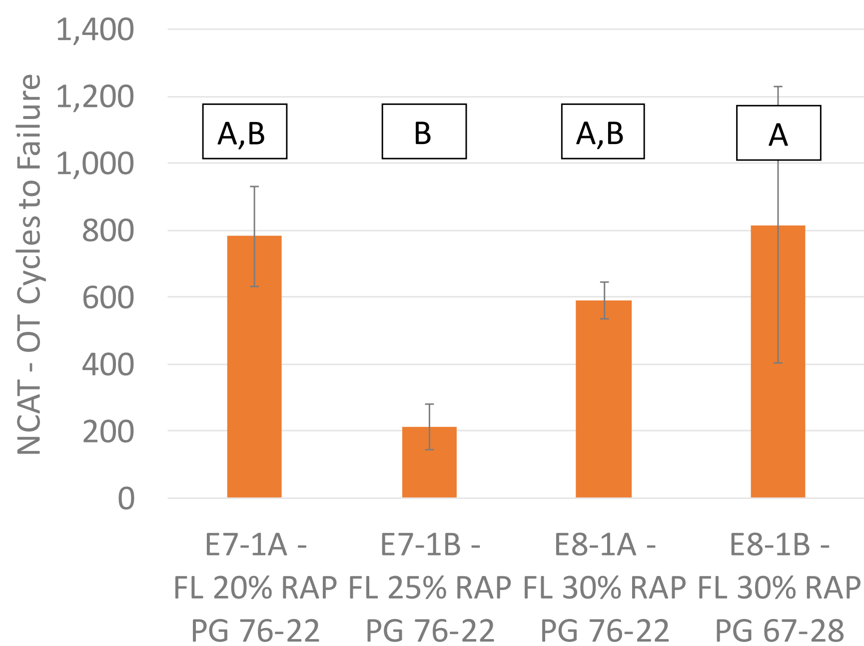

Figure 5 NCAT Modified OT Results

Table 7 OT Cracking Parameters

Mix	N_f (93%)				Crack Propagation Parameter				Crack Initiation Parameter			
	Ave	Std Dev	COV	Group	Ave	Std Dev	COV	Group	Ave	Std Dev	COV	Group
E7-1A	87	0.6	1%	A	0.49	0.03	7%	A	2.77	0.14	4.9%	A
E7-1B	15	4.6	31%	A	0.99	0.23	23%	B	3.48	0.26	7.5%	B
E8-1A	47	3.8	8%	A	0.53	0.05	10%	A	2.89	0.31	10.7%	A
E8-1B	66	59	89%	A	0.61	0.22	36%	A B	2.59	0.12	4.7%	A

Energy Ratio

The energy ratio test procedure was developed to assess an asphalt mixture's resistance to top-down or surface cracking (*2*). This testing procedure has been used in past research cycles at the NCAT Test Track as a predictor of whether a mixture might be susceptible to top-down cracking (*6*). Energy ratio is determined using a combination of three tests: resilient modulus, creep compliance, and indirect tensile strength. These tests are described in greater detail below. 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 mm thick, cut from gyratory compacted samples. The target air voids for the specimens was 7 ± 0.5%.

The resilient modulus was obtained by applying a repeated haversine waveform load in load control mode. The load was applied for 0.1 second and then followed by a 0.9 second rest. The resilient modulus was calculated using the values from the stress-strain curve. The creep compliance test was performed as described in AASHTO T 322-07; however, the temperature of the test was 10°C with a test duration of 1000 seconds. The power function properties of the creep compliance test were determined by curve-fitting the results obtained during constant load control mode. Finally, the tensile strength and dissipated creep strain energy (DCSE) at failure were determined from the stress-strain curve of the given mixture during the indirect tensile strength test. Detailed testing procedures and data interpretation methods for the three testing protocols are described elsewhere (*2, 6, 7*).

The results from these tests were then used to evaluate each mixture's surface cracking resistance using Equation 7. Data analysis was performed using a software package developed at the University of Florida. The details of the software operation are documented elsewhere (*7*). Table 8 lists the recommended thresholds for the energy ratio as a function of rate of traffic. A higher energy ratio provides more resistance to surface cracking. Additionally, a DSCE at failure of less than 0.75 kJ/m3 has been used to identify excessively brittle mixes in the field (*2*). The energy ratio criteria in Table 7 are only recommended for mixtures with a DSCE at failure of between 0.75 and 2.50 kJ/m3 (*2*).

$$ER = \frac{DSCE_f\left[7.294\times10^{-5}\times\sigma^{-3.1}(6.36-S_t)+2.46\times10^{-8}\right]}{m^{2.98}D_1} \tag{7}$$

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.

Table 8 Recommended Energy Ratio Criteria (*2*)

Traffic (ESALs/yr)	Minimum Energy Ratio
Greater than 250,000	1.00
Greater than 500,000	1.30
Greater than 1,000,000	1.95

Note: $DSCE_f$ must be greater than 0.75 kJ/m³ or the mix is considered brittle.

Table 9 shows the energy ratio results for the four mixtures in the cracking study. When comparing these data, all mixtures except the 25% RAP, PG 76-22 mixture had energy ratios above 1.95 and $DSCE_f$ values higher than 0.75 kJ/m³. Therefore, three mixtures are expected to sustain over one million equivalent single axle loads (ESALs) per year and one mixture could resist between 250,000 and 500,000 ESALs per year, although it may be too brittle and negatively affect cracking performance. One of the objectives of this study was to examine the potential impact of using a softer polymer modified grade on resistance to cracking, and several of the results shown in Table 8 suggest this to be the case. For instance, mixture E8-1B with a PG 64-28 virgin binder had the highest $DSCE_f$ value and it showed higher ductility compared to the other mixtures (highest failure strain).

Table 9 Florida DOT Cracking Study Energy Ratio Results

Mix	m-value	D_1	S_t (MPa)	M_R (GPa)	FE (kJ/m³)	$DCSE_f$ (kJ/m³)	$DCSE_{MIN}$ (kJ/m³)	Failure Strain	Creep Compliance Rate	ER
E7-1A	0.479	3.11E-07	2.57	10.06	4.2	3.87	0.761	2,152	4.092E-09	5.09
E7-1B	0.414	3.25E-07	2.05	12.41	0.8	0.63	0.483	648	2.354E-09	1.30
E8-1A	0.436	2.58E-07	2.49	13.66	3.1	2.87	0.470	1,522	2.279E-09	6.12
E8-1B	0.401	7.71E-07	2.16	9.02	5.8	5.54	1.054	3,301	4.929E-09	5.26

Semi-Circular Bend (SCB) Test

An MTS® servo-hydraulic testing system equipped with an environmental chamber was used to perform the SCB test. As shown in Figure 6, 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 load versus external displacement is used to compute the area under the curve to the peak load.

Figure 7 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 must be measured for each notch depth of each mixture. This area represents the strain energy to failure, U. The values of U at each notch depth are then plotted versus notch depth to obtain a changing slope of U from a regression line, Figure 8. This slope is the value of (dU/dA in Equation 8. Finally, the J_c can be computed by dividing the dU/dA value by the specimen width of b.

Figure 6 Semi-Circular Bend Test

$$J_c = -\left(\frac{1}{b}\right)\frac{dU}{dA} \quad (8)$$

where

J_c = critical value of fracture resistance,
b = sample thickness (mm),
U = strain energy to failure (kN-mm), and
dU/dA = slope of the area vs. displacement curve. (kN/mm).

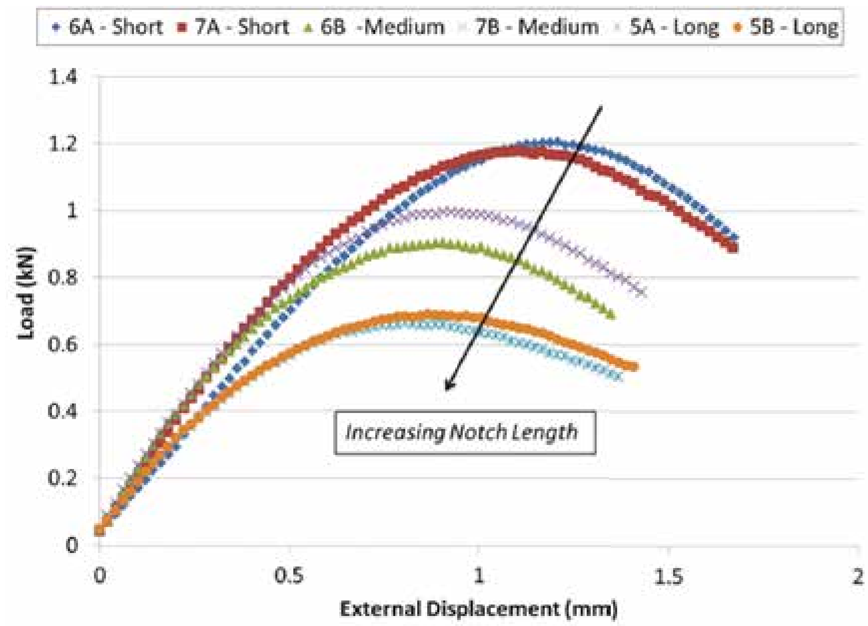

Figure 7 Typical Plot of Load versus Load Line Displacement

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. Little differences were obtained in terms of slope and J_c values (Figures 8 and 9 respectively). Higher J_c values indicate an improvement in fracture resistance; however, none of the mixtures met the minimum 0.5 criteria. The statistical grouping analysis indicated that only the mixtures from sections E7-1A and E8-1B were different at $\alpha = 0.05$.

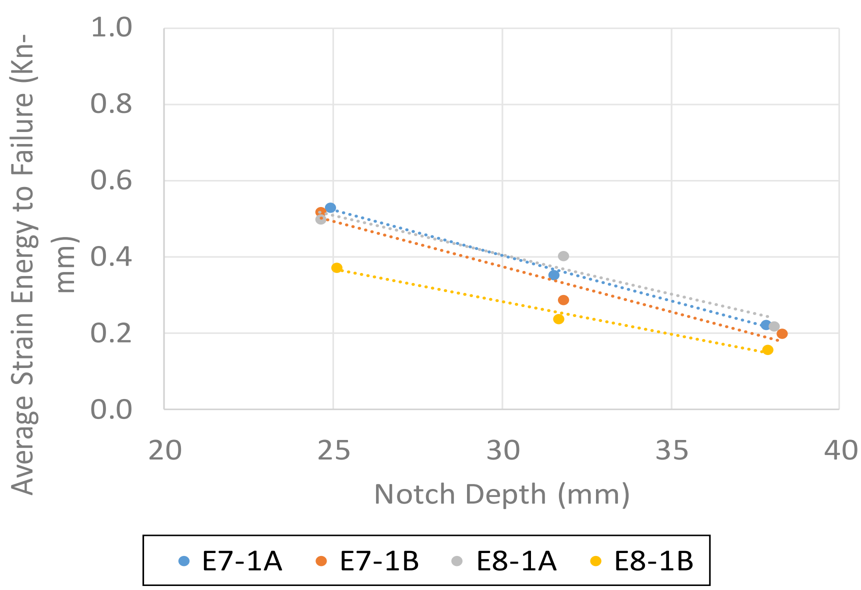

Figure 8 Plot of Area versus Specimen Notch Length

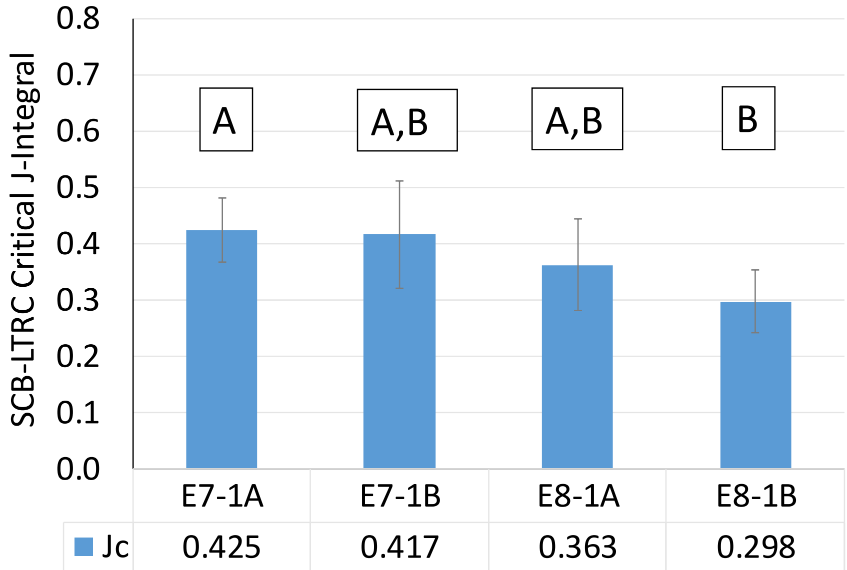

Figure 9 J-Integral Values

Fracture Energy and Flexibility Index Test

The development of flexibility index (FI) threshold values is ongoing, but research conducted for the Illinois Center for Transportation by the University of Illinois at Urbana-Champaign has made some lab to field comparisons between FI and field cracking performance (*8*). Comparisons between the FI results from loose mix samples and mixture performance at FHWA's accelerated loading facility (ALF) showed good agreement between FI and load repetitions to failure of the accelerated sections. For the FHWA ALF, the three poor-performing sections had an FI value of 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 Illinois DOT districts. The FI clearly showed the effects of aging on these cores with a reduction in FI for cores 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 preliminary recommendation of 8 has been given for minimum flexibility index.

The Illinois flexibility index test (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 by the system at a rate of 50 Hz.

The test results from all mixtures are given in Table 10. All mixtures had FI values below the preliminary criterion, but significant differences were observed. The 30% RAP, PG 64-28 mixture (section E8B) was statistically the top performer, followed by the 20% RAP, PG 76-22 mixture (section E7A), followed by the two remaining sections at the same level.

Table 10 I-FIT Test Results

Mix	Average of Fracture Energy (J/m2)	Std. Dev. of Fracture Energy (J/m2)	Average of Flexibility Index (FI)	Std. Dev. of Flexibility Index (FI)	Statistical Group
E7-1A	1,688	73	3.54	0.37	B
E7-1B	1,344	83	1.82	0.34	C
E8-1A	1,434	159	1.86	0.55	C
E8-1B	1,399	119	5.59	0.80	A

Finally, Table 11 shows a ranking analysis to organize all four mixtures from one to four with one performing best based on several laboratory cracking related parameters. Most parameters put the 30% RAP, PG 64-28 mixture (section E8B) as the most resistant to cracking and the mixture with 25% RAP, PG 76-22 (section E7B) as the least resistant. Fitting parameters used to describe the shape of the mastercurve sigmoidal function and the E* value obtained at 20°C and 10 Hz were also incorporated in this analysis. The inflection point frequency parameter $-\beta/\gamma$ is being studied as a potential cracking susceptibility indicator (9, 10). The lower this parameter, the more susceptible the mixture could be to fatigue cracking. The stiffness of the mixture at intermediate temperatures could also be used as a cracking indicator (E* at 20°C and 10 Hz). Finally, the Cantabro test has provided strong relationships to fatigue cracking in the field and seems to be able to detect differences among common mixture variables (11). In this case, Cantabro percentage loss was able to identify the top performer and the bottom performer similarly to other laboratory performance tests.

Table 11 Laboratory Test Results Ranking Analysis

Mix ID	PG Grade	ER	OT-NCAT	FI	SCB - J_c	Cantabro % Loss	$-\beta/\gamma$ (Inflection Point)	E* 20C, 10 Hz (ksi)	
E7-1A	94-16	5.1	782	3.5	0.43	5.41	-1.95	986	**Combined Ranking**
E7-1B	94-16	1.3	212	1.8	0.42	5.45	-2.66	1154	
E8-1A	94-16	6.1	591	1.9	0.36	4.55	-2.09	1042	
E8-1B	88-16	5.3	816	5.6	0.30	3.49	-1.87	795	
Mix ID	**% RAP**		**Individual Ranking**						
E7-1A	20	3	2	2	1	3	2	2	2
E7-1B	25	4	4	4	2	4	4	4	4
E8-1A	30	1	3	3	3	2	3	3	3
E8-1B	30	2	1	1	4	1	1	1	1

6.5 Field Performance

The final phase of the evaluation was to correlate the laboratory performance of these mixtures to their performance at the Test Track. Figures 10 to 13 show field performance parameters including roughness (International Roughness Index, IRI), mean texture depth (MTD), rutting (rut depth), and cracking (express as percentage of the lane) for all sections from 0 to 10 million ESALs of traffic. As can be seen, all sections showed slight variations in IRI values from the beginning, but this value remained almost constant in all cases. On the other hand, a small steady increase in the mean texture depth can be observed after 5 million ESALs. Almost no rutting was reported after 10 million ESALs with rut depths below 2 mm.

The 25% RAP, PG 76-22 mixture was the first to crack; however, cracking after 10 million ESALs was the highest for the 20% RAP, PG 76-22 mixture and the lowest cracking was reported for the 30% RAP, PG 76-22 mixture, which also was the last mix to crack. All of the quantified cracks in these sections were low severity cracks at the end of trafficking. However, it was observed that after five million ESALs, cracks did not change much in extension but were increasing in severity, close to the point of being classified as medium severity cracks. It was also determined that most cracks were reflective cracks based on the cracking map created prior to the placement of these mixtures.

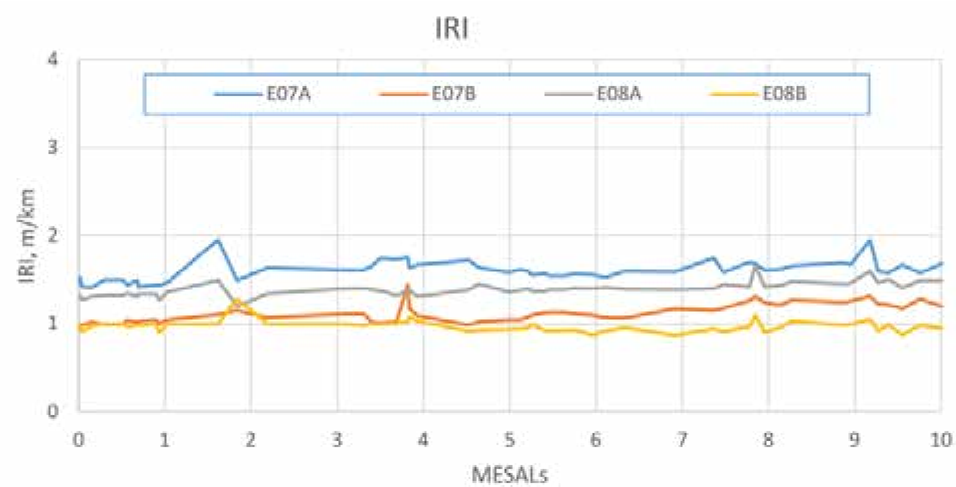

Figure 10 Measured Roughness

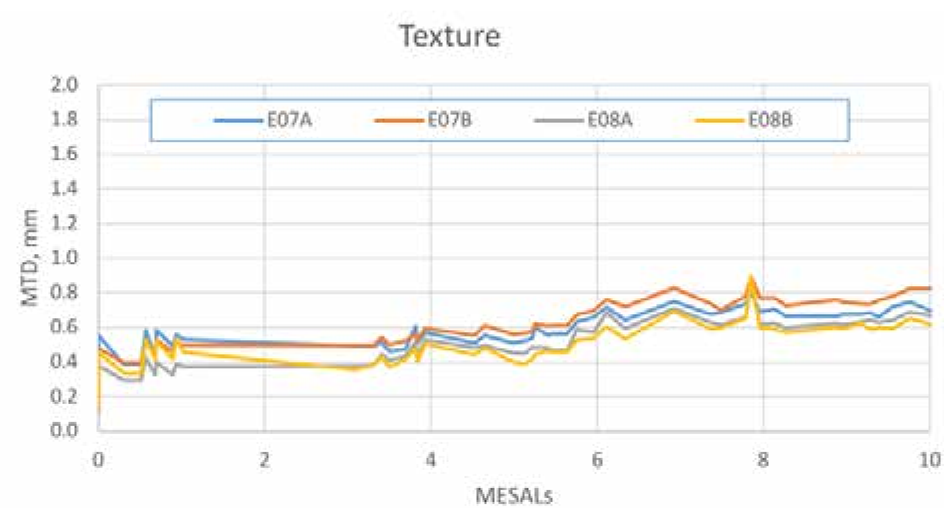

Figure 11 Measured Texture

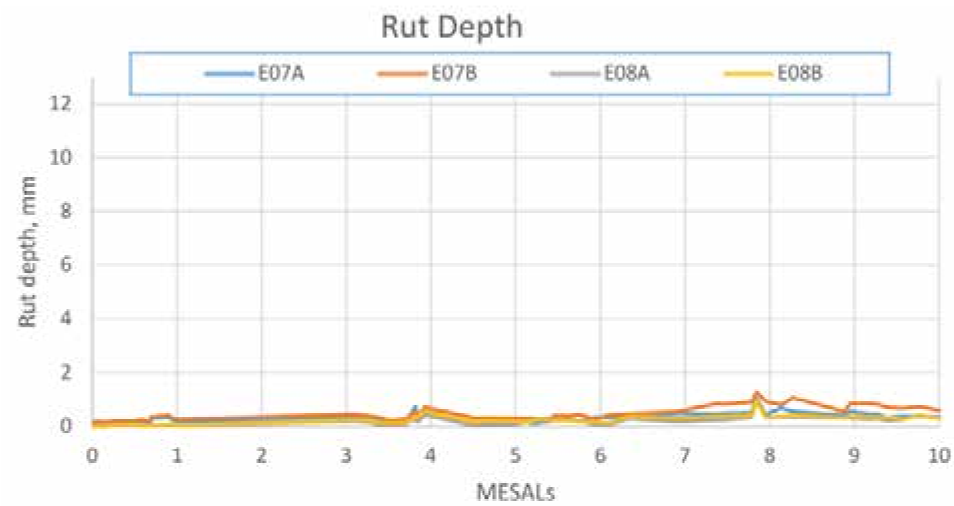

Figure 12 Measured Rutting

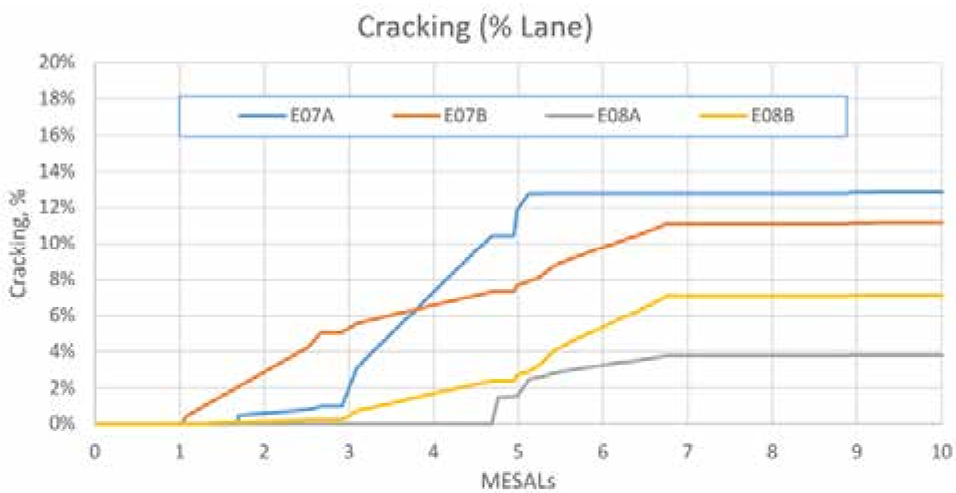

Figure 13 Measured Cracking

Pearson correlations were developed between the average laboratory mixture properties/results and the percent cracking in the field. These results are given in Table 12. R-values close to 1 and -1 show high degrees of correlation. R-values near zero are indicative of non-correlated variables. Despite some good correlations found among laboratory parameters, percent field cracking did not show a good correlation with any of the laboratory parameters at 10 million ESALs, which followed the expected trend. The only good correlation was obtained with the J_c parameter; however, the relationship follows the opposite expected trend: higher J_c values lower the expected cracking susceptibility. The crack initiation and propagation parameters from the OT results showed poor correlation field cracking with R-values of 0.3 and 0.29, respectively.

Table 12 Pearson Correlation Analysis

	Cracking	OT-TX	OT-NCAT	S_t (MPa)	M_R (GPa)	$DCSE_f$ (kJ/m3)	ER	SCB - J_c	FI
Cracking	1.00								
OT-TX	0.15	1.00							
OT-NCAT	-0.14	0.93	1.00						
S_t (MPa)	-0.06	0.67	0.54	1.00					
M_R (GPa)	-0.40	-0.66	-0.66	0.09	1.00				
$DCSE_f$ (kJ/m3)	-0.22	0.81	0.95	0.27	-0.74	1.00			
ER	-0.55	0.72	0.84	0.71	-0.17	0.75	1.00		
SCB - J_c	0.72	-0.13	-0.47	0.24	0.30	-0.67	-0.50	1.00	
FI	0.01	0.63	0.76	-0.11	-0.92	0.89	0.39	-0.64	1.00

In addition to field measurements, core samples were obtained from every section after two years of trafficking to evaluate the change in binder grade due to aging. Table 13 shows the performance grade of all binders after two years of trafficking. High critical temperature was little affected by aging. The true high PG value increased 3% on average. On the other hand, the low critical temperature was increased by two PG grade levels going from -16°C to -10°C; therefore, aging produced a significant effect on low temperature susceptibility to cracking. The true low PG increased 39% on average. Moreover, as shown in Table 14, all mixtures became more susceptible to block cracking as suggested by the decrease in the Delta T_c parameter. On average, this parameter decreased 46%. Delta T_c is defined as the numerical difference between the low continuous grade temperature determined from the Bending Beam Rheometer (BBR) stiffness criteria (the temperature where stiffness, *S*, equals 300 MPa) and the low continuous grade temperature determined from the BBR m-value (the temperature where *m* equals 0.300).

Table 13 Florida DOT Cracking Study Performance Grades After Two Years

Test Section	Extracted	T_{crit}, °C					True High PG	True Low PG	PG
		High Original	High RTFO	Intermediate	Low S	Low m			
E7-1A	Yes	98.8	97.6	33.0	-20.9	-10.0	97.6	-10.0	94-10
E7-1B	Yes	99.9	103.9	32.5	-24.9	-11.5	99.9	-11.5	94-10
E8-1A	Yes	96.9	101.1	30.2	-22.8	-13.1	96.9	-13.1	94-10
E8-1B	Yes	92.4	103.4	26.7	-28.3	-11.7	92.4	-11.7	88-10

Table 14 Comparison of Delta T_c Values (20 hr PAV)

Test Section	Delta T_c	
	Production Sample	Two-year Sample
E7-1A	-7.1	-10.9
E7-1B	-9.1	-13.4
E8-1A	-6.7	-9.7
E8-1B	-11.7	-16.6

6.6 Conclusions

The results of this study support the following conclusions.

- Laboratory results did not exhibit expected trends in terms of cracking potential. The mixture with the highest content of RAP and highest critical performance temperature (30% RAP, PG 76-22) was expected to be more susceptible to cracking. However, laboratory testing showed mixed results. On the other hand, the mixture with the softest binder did exhibit less susceptibility to cracking based on laboratory results (except the SCB test).
- All mixtures were not susceptible to rutting as indicated by the Hamburg test, which was also corroborated with almost non-existent field rutting.
- The 25% RAP, PG 76-22 mixture was ranked as a poor performer based on laboratory test results and was the first mix to crack in the field; however, it was not the mixture with the highest amount of field cracking.
- Energy ratio testing was unable to designate these RAP mixtures as brittle in the laboratory, except for the 25% RAP, PG 76-22 mixture.
- After 10 million ESALs of traffic, field cracking did not correlate to any of the laboratory parameters.
- At the end of trafficking, cracking in the test sections was classified as low severity.
- After 10 million ESALs of traffic, field performance was good with cracking at less than 20%, no changes in roughness, little permanent deformation, and increase in texture.
- After two years of trafficking, the low critical temperature and Delta T_c parameter were the most affected by aging, thus increasing cracking susceptibility.
- Field aging compared to the laboratory aging level may potentially influence the lack of correlation between the measured field cracking and laboratory parameters.

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CHAPTER 7 GEORGIA DEPARTMENT OF TRANSPORTATION INTERLAYER STUDY FOR REFLECTIVE CRACK PREVENTION

7.1 Background

In 2012, the Georgia Department of Transportation (GDOT) placed two test sections at the National Center for Asphalt Technology (NCAT) Test Track to evaluate two potential methods for reducing reflective cracking. The agency's traditional approach has been to place a single surface treatment application of No. 7 stone over the existing surface and then level with 75 to 80 lbs/sy of asphalt mix before placing an overlay. The leveling course keeps the surface treatment stone from moving during placement and compaction of the overlay and fills surface texture in the surface treatment so the spread rate of the overlay can be controlled. Stone sizes are provided in GDOT specifications, Table 800.1, and are similar to ASTM D448 size designations.

The asphalt binder used in the treatment provides a seal over the existing cracks, and the open texture of the surface treatment provides a stress relief mechanism by creating a discontinuity between the existing surface and overlay so that underlying cracks are dissipated within the interlayer rather than reflected through to the surface. Pavement stresses are caused by bending and flexing of the pavement layer under loading and thermal stresses from seasonal variation as the pavement expands and contracts.

The use of the chip seal method described above, however, has not been as effective as desired in Georgia. For this reason, GDOT decided to place two alternative treatments to the traditional approach in the 2012 Test Track cycle: a surface treatment interlayer and an open-graded interlayer (OGI).

7.2 Section Preparation and Construction

To simulate cracking in the pavement structure, deep saw cuts 1/8-inch wide (width of saw blade) were made in the existing pavement (Figure 1) for the full depth of the pavement structural layer in order to create the effect of a cracked surface. The saw cuts were made in a longitudinal direction at 3-foot intervals across the width of the lane and at 15-foot intervals in a transverse direction. Therefore, the saw cut area represented about one-third of the total test section surface area when using one foot on each side of the crack as the potential area of influence. The cuts were then filled with sand to keep the cracks from healing back together during warm weather.

Section N12 was covered with a surface treatment interlayer consisting of double surface treatment and sand seal placed about 0.7 inches thick and surfaced with a 1.5-inch thick layer of 3/8 inch (9.5 mm, Type 2) nominal maximum aggregate size (NMAS) Superpave mix. The N12 reflective cracking treatment was constructed by placing No. 7 stone followed by No. 89 stone. A sand seal surface was then placed over the No. 89 stone (Figure 2) before adding the asphalt surface layer. CRS-2h emulsion application rates and subsequent stone spread rates are provided in Table 1. The total residual application of emulsion was 0.53 gal/sy.

Section N13 was covered with a 1.1-inch thick OGI mixture and 1.1-inch thick surface layer using the same 3/8-inch mix as Section N12. This resulted in a thicker surface layer for N12, but the goal was to place the same combined thickness of about 2.2 inches above the saw cuts for both treatments.

Figure 1 Deep Saw Cuts to Simulate Pavement Cracks

Figure 2 Surface Treatment Layer with Sand Seal

Table 1 Emulsion and Stone Application Rates

	No. 7 Stone	No. 89 Stone	Sand Screenings
CRS-2h, gal/sy (residual)	0.23	0.20	0.10
Stone application, ft^3/sy	0.24	0.16	0.15

The OGI mix, shown in Figure 3, was a 1/2-inch NMAS porous friction course (PFC) mixture that was designed with a lower asphalt content than a typical PFC surface mix and used PG 64-22 binder grade. It also omitted fiber stabilizer for economic reasons, and instead used a reduced mix temperature of 250°F ± 20°F to resist drain-down. The purpose of this mix was to provide a discontinuity between the existing surface and overlay (to relieve stresses from loading and thermal forces) so stresses are dissipated and cracks are not as easily reflected.

PG 64-22 was used in the 9.5 mm, Type 2 surface mix for both N12 and N13, and the layer was compacted to 93.8% of maximum theoretical density. Optimum asphalt content for Superpave mixes in Georgia is based on 65 gyrations with a Superpave gyratory compactor. GDOT specifications contain two 9.5 mm mixes: the 9.5 mm, Type 1 is a finer gradation generally used for leveling courses and thin overlays; the 9.5 mm, Type 2 mix is generally used as surface course on more heavily traveled routes. Layer thickness of the 9.5 mm, Type 2 typically ranges from 1-1/8 inch to 1.5 inches. Mixture properties for the OGI and 9.5 mm mixes are shown in Table 2.

Figure 3 OGI Interlayer

Table 2 OGI and 9.5 mm Gradation and Mixture Properties

	OGI Interlayer		9.5 mm, Type 2	
Sieve Size	**% Passing**	**Specification**	**% Passing**	**Specification**
3/4"	100	100	100	100
1/2"	96	80-100	100	98-100
3/8"	59	40-65	95	90-100
No. 4	14	10-25	64	55-75
No. 8	8	2-8	44	42-47
No. 200	2.0	1.0-4.0	5.9	5.0-7.0
Mixture Properties				
Asphalt Content (AC), %	4.5	4.0-5.0	5.6	5.25-7.00
Air Voids, %	22.2	22±1	4.1	4.0
Voids in Mineral Aggregate (VMA), %	30.8	-	15.4	16.0
Film Thickness, µm	23.2	-		>7.0

7.3 Field Performance

For the first year after placement, there was virtually no cracking on either of the sections. At the end of the 2012 cycle and after more than 10 million equivalent single axle loads (ESALs), the surface treatment section of N12 had slightly less reflective cracking than the OGI section, but the cracking was just beginning to develop in both sections. Rutting began to develop more quickly in N12 during the second year of traffic. In November 2014 at the end of the 2012 cycle, N12 had rutted almost 1/2 in. (11.5 mm) while rutting in N13 was only about 1/4 in. (6.2 mm).

After trafficking for two more years and more than 20 million ESALs of loading, the amount of cracking for the OGI interlayer of N13 increased significantly so that 50% of the saw cut area had reflected through to the surface (Figure 4). Meanwhile, reflective cracking in N12 was only 6% of the saw cut area. A side-by-side visual comparison is provided in Figure 5 with cracking identified by the white paint markings at the end of the 2015 cycle in November 2017. The paint marking tends to exaggerate the degree of cracking because cracking in each section is still at a low severity level (Figure 6). Low severity cracking is defined as ≤ 6mm in width.

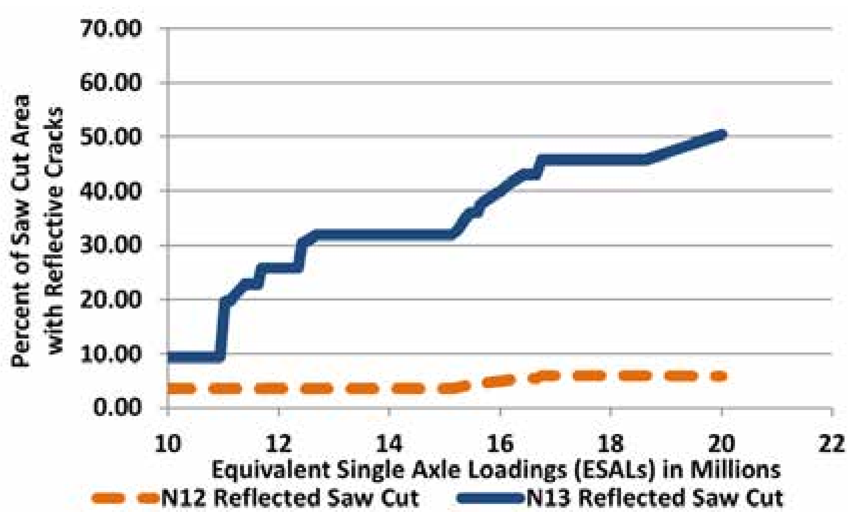


Figure 4 Reflective Cracking Comparison of N12 and N13

(a) N12 (b) N13

Figure 5 Cracking at End of 2015 Loading Cycle (November 2017)

Figure 6 Low Severity Cracking in N12

There was rutting in both sections after more than 20 million ESALs, but the rutting was much more significant in N12, which has the surface treatment interlayer. Accurate wireline measurements were taken at the end of the 2015 cycle in November 2017 in both wheelpaths at 10 locations, for a total of 20 measurements taken within both N12 and N13 test sections. As shown in Figure 7, maximum rutting in N12 exceeded 3/4 inch (21 mm) and averaged just under 1/2 inch (10.4 mm). Maximum rutting in Section N13 was 1/4 inch (6 mm) with an average of 3/16 inch (4.2 mm) ruts. A visual example of rutting in N12 is shown using a 4-foot straightedge in Figure 8.

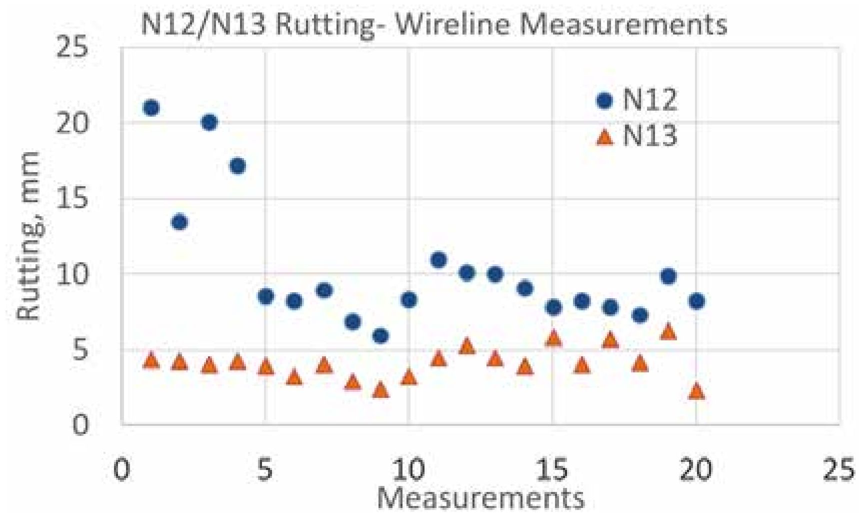


Figure 7 Rutting Comparison of N12 and N13

Figure 8 Rutting in Section N12

7.4 Findings

- The total amount of reflective cracking increased significantly for N13 during the 2015 cycle, to the level that 50% of the saw cuts have reflected through to the surface. Only

6% of the saw cuts in the surface treatment interlayer in N12 have reflected through to the surface.

- Cracking in both N12 and N13 sections is still at low severity (≤ 6 mm).
- N12 (Surface treatment interlayer) has significantly more rutting than N13. Based on wireline measurements, N12 has a maximum of more than 3/4 inch ruts (21 mm) in the inside wheelpath at the end of the 2015 cycle while N13 has a maximum of only 1/4 inch (6 mm) rut depth.

CHAPTER 8 KENTUCKY TRANSPORTATION CABINET LONGITUDINAL JOINTS AND MIX DURABILITY EXPERIMENT

8.1 Introduction

One of the advantages of constructing asphalt pavements is that they can minimize traffic disruption since they can be paved and opened to traffic rapidly. However, paving one lane at a time creates a problem because it requires a longitudinal joint between lanes. When the first lane is constructed, there is an unconfined edge with no structural support to restrain new mix from moving laterally during compaction. Conversely, the second lane will have a confined edge during compaction at the joint where the first lane was paved. As a result, two uneven surfaces can form at the joint due to the confined and unconfined edges. This difference in structural support can lead to lower density, higher permeability, and premature raveling, making longitudinal joints the weakest location of an asphalt pavement and often the most common location for premature failure even with sound pavements.

Over the past few years, the Kentucky Transportation Cabinet (KYTC) has experienced quick deterioration of their asphalt pavements' longitudinal joints. Since longitudinal joints are inevitable, guidance is needed to improve the durability of asphalt pavements and therefore, the performance of longitudinal joints.

Dense graded asphalt mixtures currently specified by KYTC are coarse-graded mixtures meaning that their gradation passes above the primary control sieve (PCS) point. During the initial implementation of Superpave, it was believed that coarse-graded mixtures would provide a stronger aggregate structure, and therefore, better resistance to rutting. Over the past several years, research conducted in past NCAT test track cycles has shown that fine-graded mixtures perform at least as well as coarse-graded mixtures in terms of rutting performance.

8.2 Objective and Scope

The objective of this experiment was to construct two test sections: a test section with a standard Kentucky mix (S7A) and a second section (S7B) with a finer mix designed with a lower number of gyrations. Both mixes contained the same aggregate components with the second mix having different percentages to achieve the finer gradation when compared to the standard mix. The goal was to improve the performance of longitudinal joint and overall mix durability without compromising rutting performance.

To support this effort, both the inside and outside lanes of the Test Track were paved, taking care to wait a few days between mat placements to simulate actual staged construction. There was no special mat edge treatment at the joint (i.e., the standard screed end gate was in place for both lanes).

8.3 Methodology

In 2015, the KYTC sponsored two test sections at the NCAT Test Track. One test section used an approved KYTC surface mix with a 9.5mm nominal maximum aggregate size (NMAS), 100 gyrations, SBS modified mix. The second section used a mix proposed by NCAT with the same

NMAS, utilizing the same aggregates and asphalt binder, but 65 design gyrations with a finer aggregate gradation. The aggregate percentages used for both mixes are shown in Table 1. Quality control information compiled during construction is presented in Table 2. As it can be observed from this table, critical sieve sizes to achieve the finer gradation for Section S7B are 3/8", #4, #8, and # 16.

Table 1 Kentucky Aggregate Percentages

Aggregate Type	% of Total Aggregate	
	S7A	S7B
Limestone #9	43	
Limestone sand	25	49
Washed friction sand	20	25
Natural sand		16
RAP	12	10

Table 2 Kentucky Mix Design Information

Mix Design Parameters	S7A	S7B
Design Method	Superpave	
Compactive Effort	100 Gyrations	65 Gyrations
Binder Grade	76-22 (SBS Modified)	
Quality Control		
Compactive Effort	100 Gyrations	65 Gyrations
$P_{3/4"}$, %	100	100
$P_{1/2"}$, %	100	100
$P_{3/8"}$, %	93	100
$P_{\#4}$, %	51	79
$P_{\#8}$, %	26	46
$P_{\#16}$, %	16	32
$P_{\#30}$, %	12	24
$P_{\#50}$, %	9	12
$P_{\#100}$, %	7	7
$P_{\#200}$, %	5.3	5.1
Plant Binder Setting (%)	5.6	6.2
Effective Binder Content, %	4.7	4.9
Rap Binder Ratio	12.8	10.3
G_{mm}	2.476	2.434
G_{mb}	2.403	2.370
G_{sb}	2.630	2.590
Air Voids, %	3.0	2.6
Production/Construction Data		
As-Built Lift Thickness, in	1.3	1.4
Type of Tack Coat	NTSS-1HM	
Undiluted Target Tack Rate, gal/sy	0.08	0.08
Temperature at Plant, F	345	345
Average Mat Compaction, %	92.1	95.1

8.4 Laboratory Testing

Plant produced mix for each mix was obtained during construction and tested at NCAT's main laboratory. Mixes were evaluated for rutting and stripping susceptibility using the Hamburg wheel tracking test (HWTT) in accordance with AASHTO T 324. Resistance to cracking was assessed using the overlay tester (OT) per Texas test procedure Tex-248-F. Moisture damage resistance was assessed in accordance with AASHTO T 283. Finally, the abrasion resistance of mixtures as an indication of durability of the mixes was evaluated using the Cantabro abrasion test in accordance with ASTM D7064.

Hamburg Wheel Tracking Test Results

Both mixes were assessed for rutting resistance using the HWTT. Tests were conducted at 50°C. For each mix, two replicates were tested. The specimens were originally compacted to a diameter of 150 mm and a height of 115 mm. These specimens were then trimmed so that two specimens, with a height between 38 mm and 50 mm, were cut from the top and bottom of each gyratory-compacted specimen. The air voids on these cut specimens were 7 ± 2%, as specified in AASHTO T 324. The samples were tested under a 158 ± 1 lb wheel load for 10,000 cycles (20,000 passes) while submerged in a water bath that was maintained at a temperature of 50°C. The average rut depths at 20,000 passes for both mixtures are presented in Table 3. The results show higher rut depth for the fine-graded mixture but not to a level that would indicate inferior performance when compared to typical specifications criterion that limit rut depth to a maximum of 12.5 mm for 20,000 passes.

Table 3 HWTT Results

Mix ID	Rut Depth at 20,000 Passes, mm
S7A	3.3
S7B	6.4

Overlay Tester Results

OT testing was performed on an Asphalt Mixture Performance Tester (AMPT) in accordance with Tex-248-F. TX-OT specimens were compacted in an SGC to a target height of 125 mm. After achieving the desired height, two specimens per sample were trimmed to the following dimensions: 150 mm long, by 76 mm wide, by 38 mm tall. Target air voids for the cut specimens were 7.0 ± 1.0%. The specimens were glued to two aluminum plates using a two-part epoxy. Four replicates were tested per mix. The samples were tested at 25°C in a controlled displacement mode. The Texas overlay results are summarized in Table 4. From these results, the fine mix shows better performance with higher number of cycles to failure when compared to the coarse mix. For both mixes, the average coefficient of variation (CV) for the test results is high, but they are still in agreement with other test results conducted by NCAT for other research studies.

Test criteria for OT test have been suggested by agencies such as Texas and New Jersey, but in some instances, these criteria are still changing. New Jersey criterion for a PG 76-22 surface mix requires a minimum number of cycles to failure of 175. Texas criterion for thin overlay mixes requires a minimum of 300 cycles to failure. The results obtained for both of the mixes in this study would pass the New Jersey requirement, but the coarser mix from Section S7A would not pass the Texas requirement.

Table 4 OT Test Results

Mix ID	Cycles to Failure	CV (%)
S7A	220	71
S7B	348	58

Tensile Strength Ratio Results

Moisture susceptibility testing was performed in accordance with AASHTO T 283. Six specimens of each mix were compacted to a height of 95 mm and an air void level of 7 ± 0.5%. The conditioned specimens were vacuum saturated to the point at which 70 to 80% of the internal voids were filled with water. These samples then underwent a freeze-thaw cycle as specified by AASHTO T 283. Table 5 provides the average conditioned tensile strength, average unconditioned tensile strength, and tensile strength ratio for each mixture. The TSR value of the fine mix is slightly higher, but both mixtures exceed the criterion of 0.80 suggesting the mixtures should be resistant to moisture damage.

Table 5 TSR Test Results

Mix ID	Conditioned ITS (kPa)	Unconditioned ITS (kPa)	TSR
S7A	1,087	1,197	0.91
S7B	1,180	1,237	0.95

Cantabro Test Results

Although the Cantabro test is typically used for open graded asphalt mixtures, in this study, Cantabro test results were used as a relative measurement of durability between the coarse and fine mixes. The test method followed for this testing was AASHTO TP108-14. For this test, laboratory compacted samples are individually placed in the Los Angeles abrasion machine without the steel charges and tested for 300 revolutions at a rate of 30 to 33 revolutions per minute. The loose material is then discarded and the final specimen weight is recorded. The percent loss is calculated by subtracting the final weight from the original weight. Three samples of each mix were tested, and the results are given in Table 6. The results only show a slight improvement in the percentage loss for the finer mix, but the results are comparable.

Table 6 Cantabro Abrasion Results

Mix ID	Cantabro % Loss	CV (%)
S7A	10.6	7.1
S7B	9.2	5.8

8.5 Field Performance

The field performance of the sections was routinely assessed. Sections were inspected for signs of cracking, and multiple measurements of rutting and surface texture were made. After 10 million ESALs of trafficking, neither mixture showed signs of cracking. Figures 1, 2, and 3 illustrate the field performance measurements of each test section in terms of rutting, roughness, and texture. Both test sections had rut depths of less than 3 mm. Roughness in terms of IRI values for Section S7A were higher than for Section S7B. Initial IRI were approximately 1.2 m/km and 0.7 m/km for Sections S7A and S7B, respectively, indicating an improved smoothness for the finer mix. The IRI values for both sections remained relatively constant throughout the test cycle. A similar trend can be observed in terms of mean texture depth (MTD), showing a higher MTD for Section S7A when compared to Section S7B but remaining relatively constant for the duration of the test cycle.

In addition, permeability was measured directly at the pavement joint of each section using the NCAT field permeameter near the completion of trafficking. Higher permeability can lead to durability problems. The average values measured for Sections S7A and S7B were 1,294x10^{-5} cm/s and 243 x10^{-5} cm/s, respectively. These results clearly indicate that the coarse-graded mix exhibited higher measured permeability than the fine-graded mix. Although no durability problems were observed at the end of the test cycle, these results suggest that the finer mix proposed in this study may potentially improve the performance of the test section, particularly at the joint.

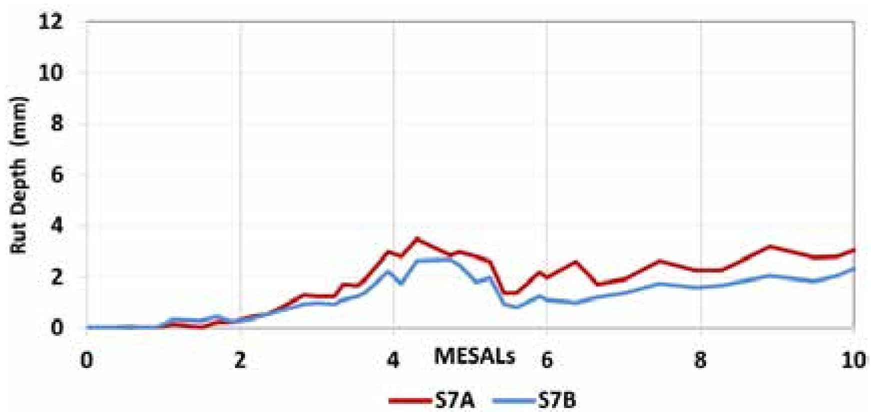


Figure 1 Measured Rutting

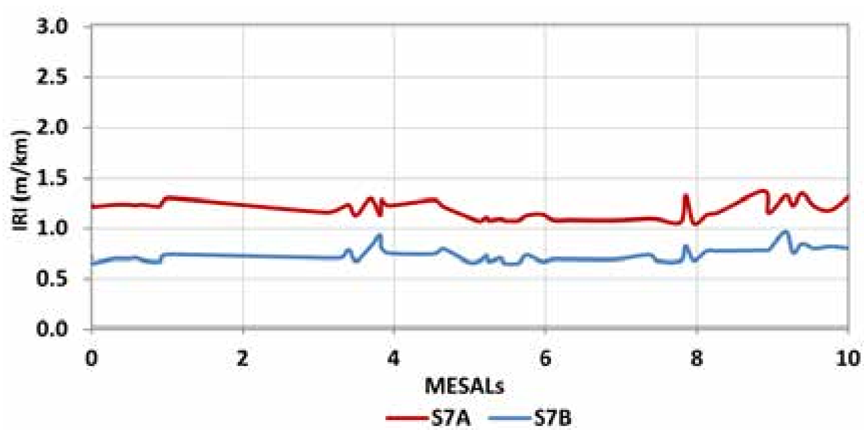


Figure 2 Measured Roughness

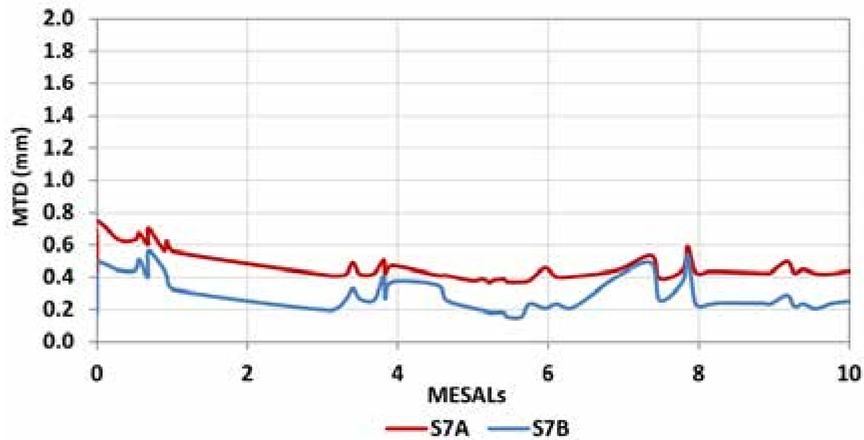


Figure 3 Measured Texture

8.6 Summary of Findings

This experiment compared the Test Track performance and laboratory test results of two test sections: a standard Kentucky mix (S7A) versus a mix with a finer blend and lower design gyration (S7B). The final objective was to assess if the finer gradation would improve the performance of longitudinal joint and overall mix durability. The following conclusions were reached:

- OT results show higher number of cycles to failure for the S7B mix when compared to Section S7A.
- HWTT results show higher rut depth for mix S7B when compared to Section S7A but are still below the max specifications criterion.
- Similar TSR and Cantabro test results were observed for both mixes.
- After two years of trafficking with 10 million accumulated ESALs, no cracking was observed in any of the sections.
- Rut depths for both sections were less than 3 mm, indicating that the fine-graded mix (S7B) performed at least as well as coarse-graded mix (S7A) in terms of rutting resistance.
- Roughness in terms of IRI for Section S7A was significantly higher than for Section S7B. This can be partially attributed to the construction effect of shorter test sections, but it is also believed that the finer gradation made the section smoother.
- Field permeability measurements were taken directly on the longitudinal joint in both test sections. The permeability value measured on Section 7B (fine-graded mix) was less than 20% of that measured on Section 7A (coarse-graded mix), which should translate into better joint performance, particularly in the freeze-thaw climate typical of Kentucky.
- Based on the results of this study, it is recommended that KYTC considers allowing fine-graded mixtures design for lower gyrations and not be limited to coarse-graded mixes.
- Since no durability problems were observed at the end of the test cycle, it is recommended to continue trafficking on these sections during the next cycle to assess their long term performance.

CHAPTER 9 MISSISSIPPI DEPARTMENT OF TRANSPORTATION EVALUATION OF THINLAY MIX WITH RAP AND LOCAL AGGREGATES

9.1 Background

The Mississippi Department of Transportation (MDOT) has been evaluating thinlays on the NCAT Test Track for 15 years. In 2003, a ¾-inch thick, low volume road, 4.75 mm nominal maximum aggregate size (NMAS) mix was placed on the track (Section W6) with the expectation that it would only last for a half-million equivalent single axle loads (ESALs); however, this mix has proven to be one of the most versatile surface layers in the history of the Test Track. To date, this section has supported over 50 million ESALs with no cracking, rutting, roughness, raveling, or friction deficiencies noted. The 2003 Mississippi thinlay consisted of 69% imported limestone screenings, 30% hard sand, 1% hydrated lime antistrip agent, 6.1% polymer modified liquid asphalt, and no reclaimed or recycled materials.

In order to address the need to maintain more lane miles with limited budgets, MDOT decided to use the 2012 research cycle to redesign thinlays by adding reclaimed asphalt pavement (RAP), changing from polymer modified to neat asphalt, eliminating imported stone screenings, and relying completely on locally available surplus sand stockpiles in Mississippi. The result of this effort was a new thinlay mix placed on the surface layer of Section S3 in 2012. The redesigned preservation thinlay surface consisted of hard sand locally available in Mississippi, RAP, Portland cement filler, hydrated lime antistrip, and neat virgin liquid asphalt. The target lift thickness on the track was 1 inch, but this mix could be used for preservation in lifts as thin as ¾ inch. Both W6 and S3 thinlay mixtures were placed on the originally constructed Test Track sections with approximately two feet of asphalt structure. A low cost per mile can be achieved as a result of the use of all local materials, RAP, and neat liquid asphalt in a relatively thin surface layer (i.e., low spread rate).

9.2 Objective

The objective of Section S3 for the 2012 research cycle was to evaluate the rutting performance of the redesigned thinlay mix on the NCAT Test Track. After the application of 10 million ESALs at the end of the 2012 cycle, no significant rutting or surface cracking was observed. MDOT chose to continue traffic through the 2015 research cycle in order to expand the scope of the mix evaluation to include cracking and durability.

9.3 Mix Design

Table 1 provides information about the redesigned preservation thinlay surface, which consisted of 72% hard local sand, 25% processed RAP, 2% Portland cement filler, 1% hydrated lime antistrip, and 5.1% neat virgin liquid asphalt. The asphalt binder contributed by the RAP source was 1.6%, resulting in 6.7% total asphalt content. Quality control data for the binder and surface mixes sampled during production are shown in Table 2. No problems were noted during production, placement, or compaction. A loose mix sample was taken during construction for laboratory performance testing. A final density of 94.2% was measured in the compacted mat with no tenderness observed at any temperature. Tables 1 and 2 also include information for the original thinlay mixture placed in Section W6 for comparison.

Table 1. Mix Design Information for Mississippi Thinlay Mixtures

Design/Materials	S3 Thinlay Mixture		W6 Thinlay Mixture	
Gyrations	Ndes	50	Ndes	50
Asphalt	Virgin PG 67-22 (unmodified)	5.1%	Virgin PG 76-22 (SBS-Modified)	6.1%
	Asphalt from RAP	1.6%		
	Total Asphalt	6.7%	Total Asphalt	6.1%
Aggregate	-3/8" Bailey Coarse Gravel	40%	Cherokee Limestone	69%
	Bailey Crushed Fines	8%	Guntown Crushed Gravel	19%
	Bailey Coarse Sand	24%	Mississippi Natural Sand	11%
RAP	Blaine RAP	25%		
Others	Hydrated Lime	1%	Hydrated Lime	1%
	Portland Cement	2%		Sttucture

Table 2. Gradation and Volumetric Properties of Mississippi Thinlay Mixtures

Mix Properties	S3 Thinlay Mixture		W6 Thinlay Mixture	
Sieve	Mix Design (Percent Passing)	Quality Control	Mix Design (Percent Passing)	Quality Control
1/2" (12.5 mm)	100	100	100	100
3/8" (9.5 mm)	99	99	100	100
No. 4 (4.75 mm)	83	86	99	98
No. 8 (2.36 mm)	61	62	72	75
No. 16 (1.18 mm)	46	46	43	50
No. 30 (0.60 mm)	35	34	30	35
No. 50 (0.30 mm)	17	17	18	22
No. 100 (0.15 mm)	9	9	11	15
No. 200 (0.075 mm)	7.5	6.4	8.0	11.3
Volumetrics				
Total Asphalt (%)	6.7	6.4	7.5	6.1
Virgin Asphalt (%)	5.1	4.8	7.5	6.1
RAP Asphalt (%)	1.6	1.6		0.0
Lab Air Voids (%)	5.0	4.1		4.0
Voids in Mineral Aggregate (VMA) (%)	18.4	17.3		16.0
Voids filled with Asphalt (VFA) (%)	73	76		
In-Place Density (% of Gmm)		94.2		92.2

9.4 Laboratory Performance Testing

As part of the 2012 cycle, the loose mix sample taken during construction was reheated in the NCAT laboratory to prepare test specimens for evaluating rutting performance of the redesigned thinlay mix. The Asphalt Pavement Analyzer (APA) was used in accordance with AASHTO T 340-10 (2015). The APA is a modification of the Georgia Loaded Wheel Tester (GLWT) and follows a similar rut testing procedure where a wheel is loaded onto a pressurized linear hose and tracked back and forth over a testing sample to induce rutting, as shown in Figure 1. Six specimens were compacted with a Superpave gyratory compactor to target air voids of 7.0 ± 0.5 percent and a height of 75 mm. The specimens were then tested at 64°C (the 98% reliability temperature for the high performance grade of the binder for the Test Track) using a vertical load of 100 lbs. and a hose pressure of 100 psi for 8,000 cycles. Rut depth readings were taken manually at two locations on each specimen after being seated for 25 loading cycles and at the end of testing (8,000 cycles in addition to 25 seating cycles) and automatically during the test. Higher rutting depth indicates the less resistance to rutting.

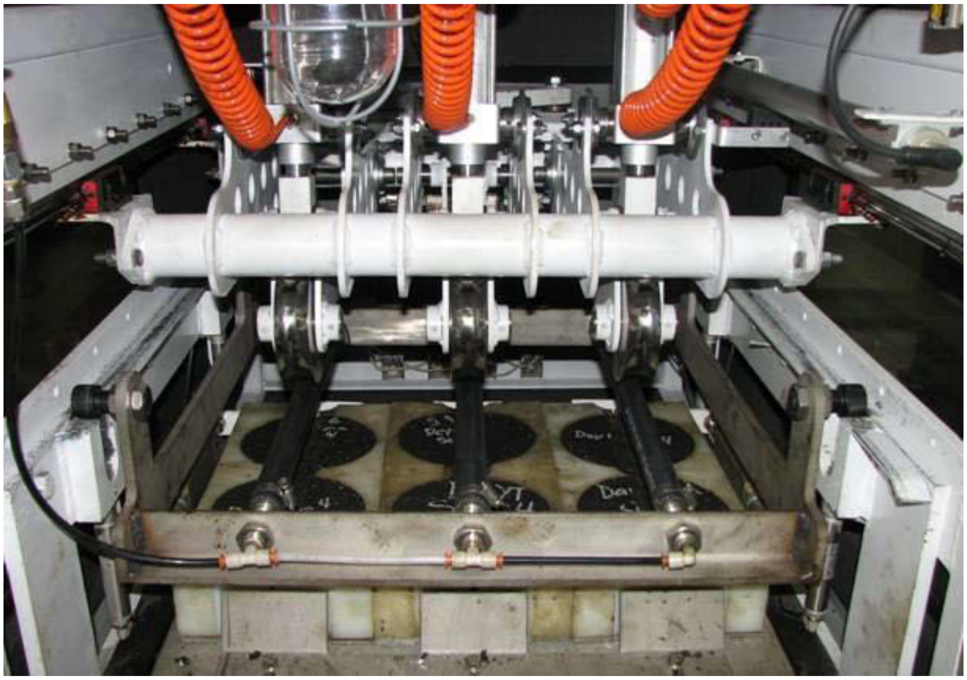


Figure 1 Asphalt Pavement Analyzer

Table 3 shows the manual and automated rut depth measurements for the thinlay mixture. Past research has shown that if a mixture has an average APA rut depth of less than 5.5 mm, it should be able to withstand 10 million ESALs of truck traffic at the Test Track without accumulating more than 12.5 mm of field rutting. Based on the data shown in Table 3, the thinlay mixture met this criterion. These APA results agree with the field rutting results since the thinlay mix did not fail due to rutting during the 2012 research cycle.

Table 3. Summary of APA Test Results

Sample ID	Air Voids (%)	Manual Rut Depth (mm)	Automated Rut Depth (mm)	Mold Placement
5	6.9	5.2	3.8	Left Rear
7	7.0	4.6	4.6	Left Front
9	6.9	5.3	4.5	Center Rear
6	7.1	5.4	4.9	Center Front
10	6.9	6.2	4.8	Right Rear
8	7.0	5.5	5.1	Right Front
Average	7.0	5.4	4.6	
Std. Dev.	0.1	0.5	0.5	

9.5 Test Track Performance

Weekly monitoring of each test section was conducted on Mondays. Each section was inspected for signs of cracking and multiple measurements of rutting and surface roughness were made. Similar to the 2003 MDOT thinlay, this mix has now supported over 20 million ESALs with no cracking, rutting, roughness, raveling, or friction deficiencies noted. At the end of the 2015 cycle, the final rut depth measured was 0.05 inches, and the amount of cracking observed was 0.2% based on the total lane area. Figures 2 and 3 show two low-severity cracks observed in Section S3.

Figure 2 Crack Observed in Section S3 Near the Inside Edge of the Outside (Research) Lane

Figure 3 Crack Observed in Section S3 Near Center of the Outside (Research) Lane

Figure 4 shows the texture change of the thinlay mixture in Section S3 through the 2012 and 2015 cycles. Macrotexture increased slightly during the 2015 cycle, but the mean texture depth was still good at around 0.6 mm after 20 million ESALs. Pavement roughness quantified using international roughness index (IRI) during the 2015 cycle is shown in Figure 5. The IRI data are very consistent, indicating that the pavement remained smooth.

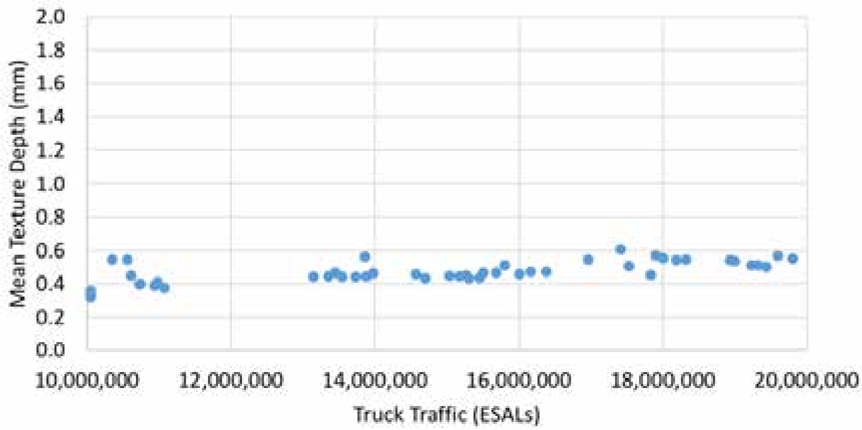


Figure 4 Macrotexture Results for Thinlay Mixture After 20 Million ESALs

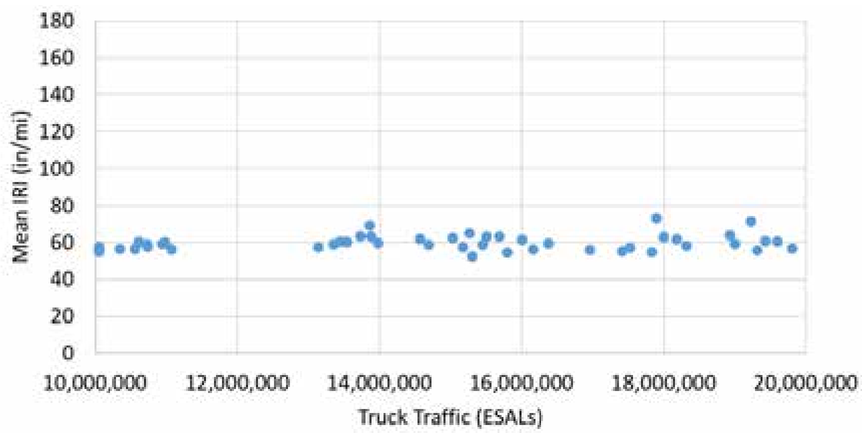


Figure 5 Pavement Roughness Results for Thinlay Mixture After 20 Million ESALs

9.6 Conclusions

The objective of Section S3 in the 2012 and 2015 Test Track cycles was to evaluate the production, placement, rutting, cracking, and durability performance of the redesigned thinlay mix consisting of all locally available surplus sand materials, RAP, and neat liquid asphalt (PG 67-22) in a relatively thin surface layer (i.e., a target lift thickness of 1 inch or less). Through the 2012 and 2015 cycles, this mix has supported over 20 million ESALs of heavy truck traffic with no cracking, rutting, roughness, raveling, or friction deficiencies noted.

CHAPTER 10 OKLAHOMA DEPARTMENT OF TRANSPORTATION OPEN GRADED FRICTION COURSE STUDY

10.1 Objective

The Oklahoma Department of Transportation (ODOT) wanted to identify maximum surface friction performance of asphalt surface mixtures using regionally available aggregates as alternatives to a standard high friction surface treatment using resin binder and imported calcined bauxite aggregate. The mixture type selected for the study was open-graded friction course (OGFC) to obtain the best macro-texture using conventional asphalt paving practices. A second objective was to determine the proper amount of tack coat to bond the OGFC to an existing pavement surface and monitor change in bond strength for a short period of time after construction.

The primary performance criterion was surface friction. The secondary performance criterion was to improve bond between the OGFC and the underlying pavement surface. Standard Test Track study measures of plant produced mixture in the laboratory and field pavement performance were also reported.

10.2 Laboratory Study

The laboratory study was performed in advance of the 2015 track construction period. The Phase I study consisted of two tasks and was documented as Oklahoma report FHWA-OK-15-10 (*1*). The first task of the study was a laboratory evaluation to compare aggregate/mixture combinations that were expected to have the best potential to provide high pavement surface friction characteristics. A testing and conditioning protocol developed at NCAT was used for measuring the friction performance of pavement surfaces. The protocol includes preparing asphalt mixture slabs, testing with the dynamic friction tester (DFT) using ASTM E1911, and conditioning (polishing) with the NCAT Three Wheel Polishing Device (TWPD).

The second task of the laboratory study was an evaluation of bond strength between layers using one tack binder material and two application rates. The laboratory built multiple two-layer slabs (OGFC over conventional dense-graded mixture) with two different tack coat rates applied between the lifts for comparison. Preparing laboratory slabs involved compacting the underlying slab, conditioning the surface of the slab to remove the asphalt film on the surface to mimic normal traffic wear, applying the tack coat, and placing the OGFC layer on top. To best simulate field tack coat application, the dense graded slabs were taken to the Test Track and the tack distributor truck applied tack to the slabs during the distributor calibration procedure. Cores were taken from the slabs for bond strength shear testing. Cores from the slabs with the higher tack coat application rate had higher interface bond strength (*1*).

10.3 Materials

The ODOT staff identified four regionally available aggregate sources with good friction performance characteristics: mine chat, rhyolite, sandstone, and granite. Based on the Phase I laboratory study friction results (*1*), the sandstone OGFC mixture was selected for further study on the NCAT Test Track.

The OGFC mix was a 12.5 mm (1/2 in) nominal maximum aggregate size (NMAS) gradation with 6.4% PG 76-28 styrene-butadiene-styrene (SBS) modified binder. The plant mixed, laboratory compacted density measured 20.2% air voids using the ODOT job mix formula requirements of 300°F laboratory compaction temperature and 50 gyrations.

The tack coat applied for this study was UltraFuse, which is the standard tack coat material used for Test Track construction. This hot-applied tack coat eliminates the time required to allow emulsion tack coat to break before paving.

10.4 Construction

On August 14, 2015 the micro-milled surface (Figure 1) of Section N9 was tacked with UltraFuse at an application rate of 0.05 gal/yd^2 residual for the first 100 ft and 0.10 gal/yd^2 residual for the second 100 ft (Figure 2). The 0.10 gal/yd^2 rate for the second 100 ft was accomplished by repeating the 0.05 gal/yd^2 rate over the 0.05 gal/yd^2 rate initially applied to the entire 200 ft section. The left longitudinal joint was tacked with an overlap of at least 2 in. to maximize longitudinal joint performance (Figure 3). An area approximately one foot wide on the right edge was not tacked because the width of the distributor spray bar was extended to accommodate the left longitudinal joint overlap. The omission of tack on the right edge of the lane is not expected to impact the study. The sandstone OGFC was placed 0.75-in. thick on the tacked surface (Figure 4).

Figure 1 Finished Micro-Milled Surface of Section N9


Figure 2 Application of Tack Coat on Section N9


Figure 3 Transition Between Low and High Tack Application Rates in Section N9

Figure 4 OGFC Placement in Section N9

10.5 Laboratory Performance of Production Mixture

In addition to the friction testing related to the scope of the study, mixture performance tests were performed to quantify the characteristics of the mixture.

Hamburg Wheel-Track Testing Results

Hamburg Wheel-Track testing was performed to determine both the rutting and stripping susceptibility of the N9-1 OGFC surface mix. Testing was performed in accordance with AASHTO T 324-16 at a test temperature of 50°C. Two replicates were tested from the production mix sample, with each replicate consisting of two trimmed specimens (four specimens total per mix). The specimens were originally compacted using a Superpave gyratory compactor (SGC) to a diameter of 150 mm (6 in.) and an N_{des} of 50 gyrations. The specimens were then trimmed from one side to a height of 60 mm (2.4 in.) to fit in the Hamburg molds for testing.

The rut depth versus wheel passes plot for both replicates is shown in Figure 5. The average rut depth for both replicates was 4.9 mm (0.2 in.), and no sign of stripping was evident in the test results. A maximum rut depth of 12.5 mm (0.5 in.) is a commonly used failure threshold in the Hamburg test (*2*). Hence, the N9-1 surface mix appeared resistant to both rutting and stripping in the Hamburg test.

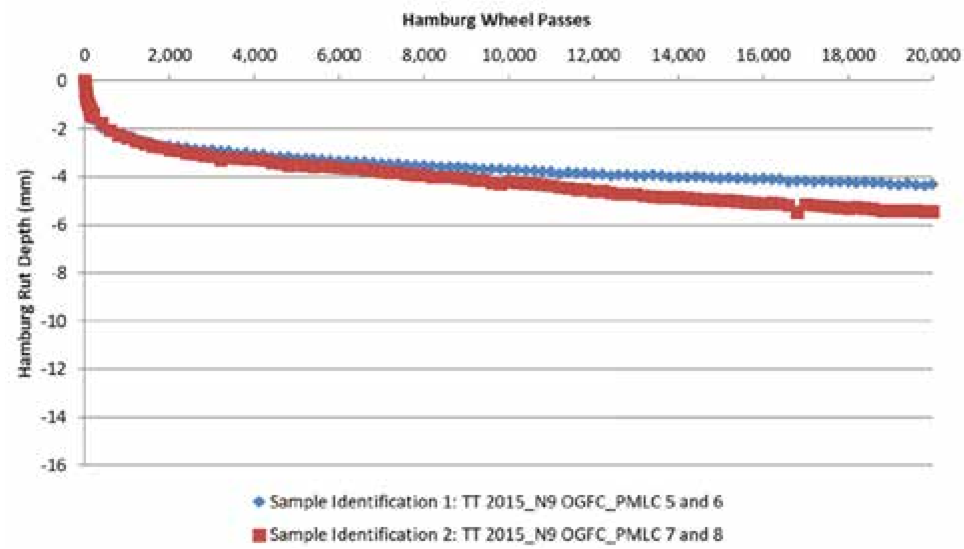


Figure 5 Hamburg Rut Depth Versus Wheel Passes

Overlay Tester Results

The Overlay Tester (OT) is a testing device designed to simulate accelerated reflective cracking in asphalt concrete overlays. No national standard (AASHTO or ASTM) currently exists for the OT, and the two state agencies that currently have a specification are the Texas Department of Transportation (TxDOT) and New Jersey Department of Transportation (NJDOT). TxDOT runs the test in accordance with the Tex-248-F standard while NJDOT runs the test in accordance with the NJDOT B-10 standard. NCAT runs the test using a fixture and software within the IPC Global® Asphalt Mixture Performance Tester (AMPT).

For the OGFC mix in N9, plant produced mixture reheated in the oven with no additional laboratory aging was compacted in the SGC to 50 N_{des}. From each compacted gyratory sample, one specimen was trimmed to 150 mm (6 in.) long, by 76 mm (3 in.) wide, by 38 mm (1.5 in.) tall. Specimen air voids were determined using the Corelok method after saw trimming. Four replicates were tested for this mixture using the TxDOT protocol of 25°C (77°F) at a frequency of 0.1 Hz with a sawtooth waveform and a 0.025" maximum opening displacement.

The NJ DOT is currently recommending greater than 175 cycles to failure for surface mixtures with a PG 76-22 binder (*3*). In their 2014 specifications, TxDOT requires a minimum of 300 cycles to failure for their OGFC mixtures (*2*).

The OT results for the N9-1 surface mix are summarized in Table 1. Two of the replicates did not test to failure during the test duration of 1,200 cycles. Their cycles to failure is listed as greater than 1,200. The average cycles to failure in the OT was greater than 1,048 considering the two replicates with a minimum cycles to failure of 1,200. This would meet the minimum cycles to failure criteria suggested in literature and suggests the mix has sufficient flexibility.

Table 1. Summary of OT Cycles to Failure

Specimen ID	Specimen Air Voids After Trimming (%)	Peak Load (lb)	Cycles to Failure (93% Load Reduction)	Average Cycles to Failure
N9-1 #1	14.2	462.4	1,140	> 1,048
N9-1 #2	14.3	504.7	> 1,200	
N9-1 #3	15.3	482.8	> 1,200	
N9-1 #6	15.0	496.6	651	

Cantabro Testing Results

The Cantabro test (AASHTO PP 108) is intended to examine the raveling potential of a compacted asphalt mixture. Three standard 150 mm gyratory specimens were prepared with reheated plant produced mix using a compaction effort of 50 gyrations to achieve a target of 15% air voids. The test result is expressed as percent mass loss after 300 conditioning revolutions in the LA Abrasion tumbler. The Cantabro percent loss for the three specimens averaged 6.0% and ranged from 4.6 to 8.7%, which is well below the commonly accepted agency criteria of 20% maximum loss.

Friction Testing Results

Two 20x20x2 inch slabs were compacted from plant produced OGFC mixture for accelerated friction performance testing. DFT tests (ASTM E1911) were performed at 0, 5k, 10k, 40k, 80k and 140k cycles of polishing with the NCAT TWPD in the NCAT laboratory. Peak friction DFT(40) of 0.46 was measured at 10k cycles and gradually decreased to DFT(40) of 0.37 at the terminal 140k cycles as shown in Figure 6. The peak friction result was similar to the Phase I laboratory study (0.48), but the terminal friction was lower than the laboratory study (0.45). The difference in terminal friction may be associated with mixture size, aggregate source variation, and aggregate degradation during plant production.

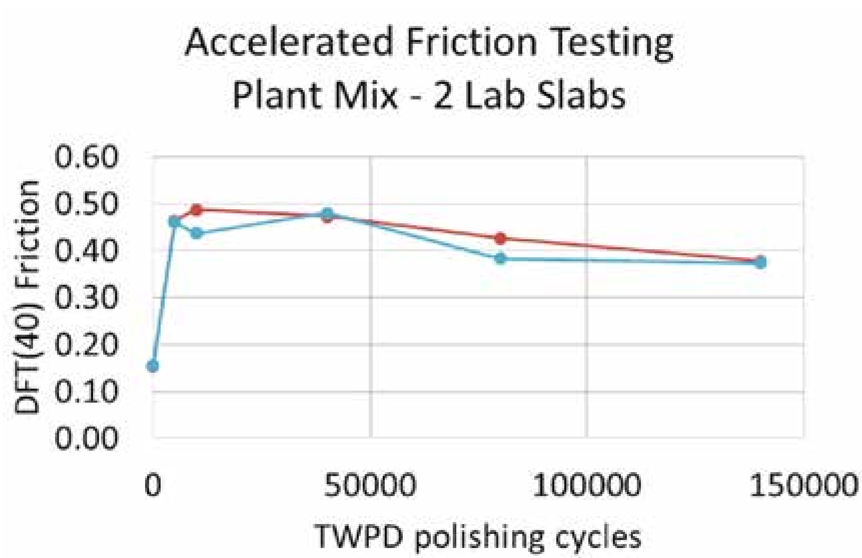


Figure 6 Accelerated Friction Performance of Plant Produced Mix

10.6 Field Performance

Friction and Texture

The pavement surface characteristics of the N9 surface were measured with four different test methods and each are discussed. The Alabama Department of Transportation (ALDOT) friction skid trailer measured ribbed-tire friction monthly using ASTM E274 procedures. The NCAT DFT measured friction using ASTM E1911 and circular texture meter (CTM) measured surface texture using ASTM E2157 at three random locations in the left wheel path every three months. The Test Track automated performance measurement van's high speed laser measured texture every week. Figure 7 combines the field friction and texture measurements for easy comparisons.

Friction performance measured with the skid trailer dropped slightly over the two-year traffic loading period. The highest measured SN40R values of 57 were in April through June 2016 and final SN40R values of 53 were in the last three months of truck traffic. Friction performance measured with the DFT was DFT(40)=0.60 after twelve months and declined to a terminal value of 0.50 in the last four months of traffic loading. Measurements taken in the first five months of traffic were not included in the evaluation due to DFT instrument failure. It was determined that the transducer that measures the skid plate deflection had failed and was producing errant readings.

Both friction tests measured a decline in friction over the two-year period. The greater change in the DFT measurements indicates that the DFT was more responsive to the change in surface friction characteristics than the skid trailer SN40R. The measured friction was higher than the normal 45 to 35 SN40R for dense-graded asphalt pavement surfaces placed on the track and lower than standard high friction surface treatment values above 65 SN40R. This ODOT OGFC out-performed other ODOT surface mixtures placed on the Test Track (Figure 8).

Pavement surface texture measured with the high-speed laser mounted to the van showed that macro-texture values started at 1.2 mm mean profile depth (MPD) and dropped slightly to 1.1 mm over the two years of traffic. The texture measured with the CTM show that macro-texture values started at 2.0 mm mean profile depth (MPD) and dropped to 1.8 mm for the first twelve months. In the second twelve months the values varied between 1.8 mm and 1.5 mm. This most likely reflected differences in the amount of fine material filling the voids and the common variation in surface texture when measuring at random locations in the wheel path.

Macro-texture values measured with the CTM are more accurate than the high-speed laser and the variation represents the difference between testing at three locations with the CTM versus a total length average with the high-speed laser. Both values indicate very good macro-texture, as expected from OGFC surfaces in comparison to dense-grade mix surfaces.

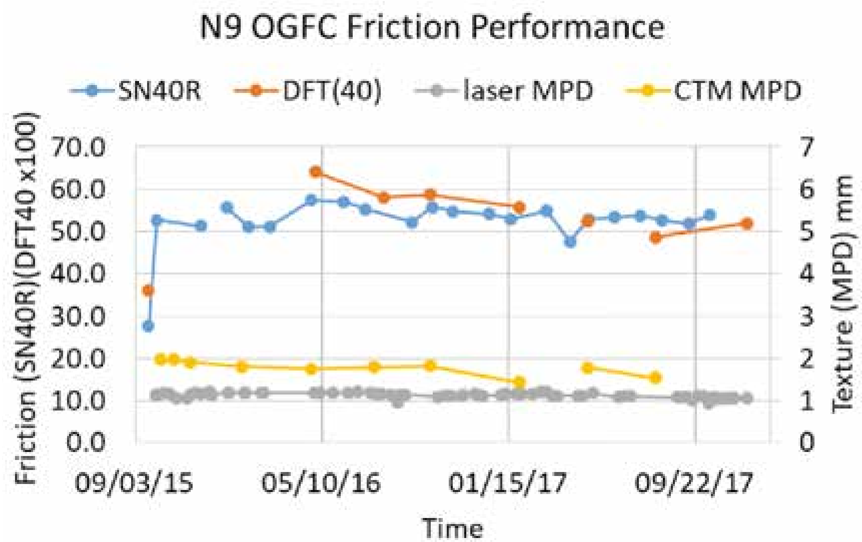


Figure 7 Friction and Surface Texture Performance

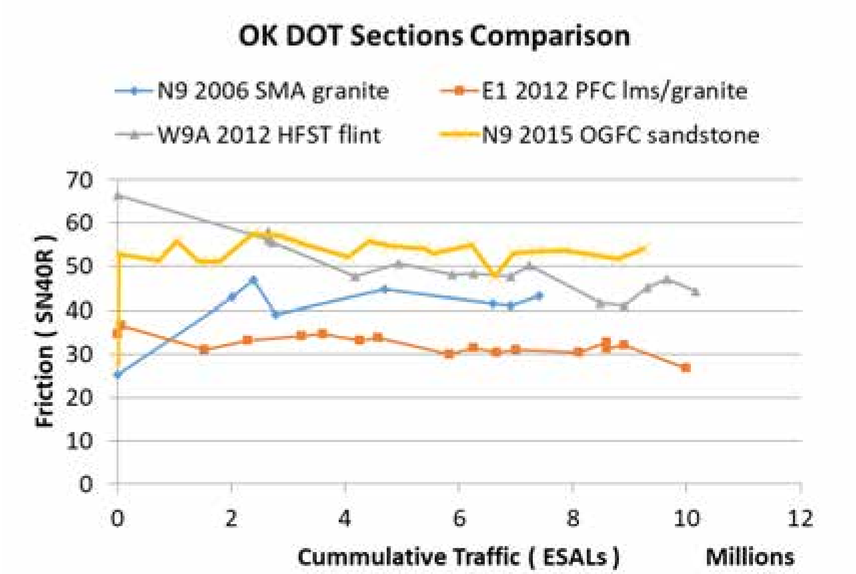


Figure 8 Comparison of Friction Performance of Four ODOT Surface Mixtures

Bond Strength

Field bond strength proved to be difficult to measure, but the results generally indicated that the higher tack coat application rate measured higher tensile strength. The preferred method to assess the interface bond strength is to remove a full-depth core from the pavement, trim the ends as needed, and test in the laboratory. This method takes time and leaves full depth core holes in the pavement. The number of cores required for time-series measurements would seriously weaken the test section. The approach is also problematic for OGFC mixes because the thickness of the OGFC lift is not sufficient for shear testing across the interface between the OGFC surface and underlying pavement. To address this problem, this study proposed a field procedure to test the interface tensile bond strength between surface mix and the underlying asphalt layer. A pull-out test used for Portland cement concrete testing was adapted for use on this project.

The study proposed applying a direct tensile pull-out force on small diameter cores as a field quality control (QC) test for interface bond strength. Six 2-inch diameter cores were cut into the pavement surface 12 inches apart in a longitudinal line between the wheel paths to minimize the impact on mixture in the wheel path (Figure 9). The depth of each cut was approximately 1.25 inches to cut through the OGFC surface and partially into the underlying layer. Tests were performed at one week, two weeks, one month, and two months after placement to attempt to monitor any change in bond strength after construction. Small 2-in. diameter cores were selected to accomplish the high number of tests needed for the time-series plan within the confines of the 25-ft destructive testing zones at the beginning and end of the section and further limited to the area between the wheel paths.

The instrument selected for the testing was a Germann Instruments LOK-TEST pull-out tensile test device with a load capacity of 100 kN (22 kips). A neoprene spacer was made to raise the device off the pavement surface to connect the test device to a 2-inch diameter, one half-inch thick metal disc epoxied to the in-place cut core face.

As testing proceeded, two parameters of the test were modified. One, the pavement surface must be a low, early morning temperature, and two, the rate of tensile pull was increased. A summary of each day of testing follows in Figures 10 through 13.

Measured test values from test day to test day cannot be compared because of the test procedure revisions, but the bond strength performance between the tack coat application rates can be compared for the same test day. Test results on the second and third test day clearly show improved tensile bond strength for the section placed with the higher tack coat rate. A majority of tests performed on the lower tack coat rate broke at the tack interface. On the higher tack rate the majority of the tests broke in the OGFC above the interface.

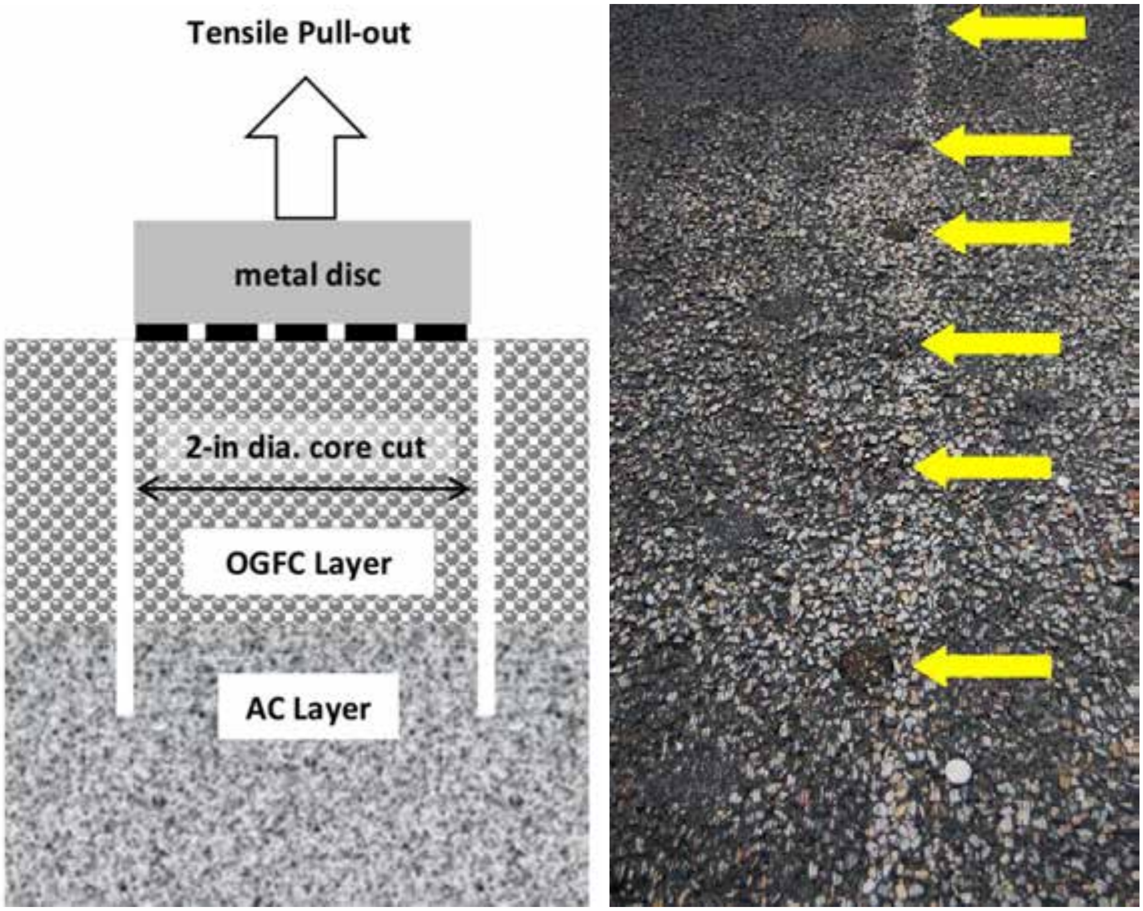
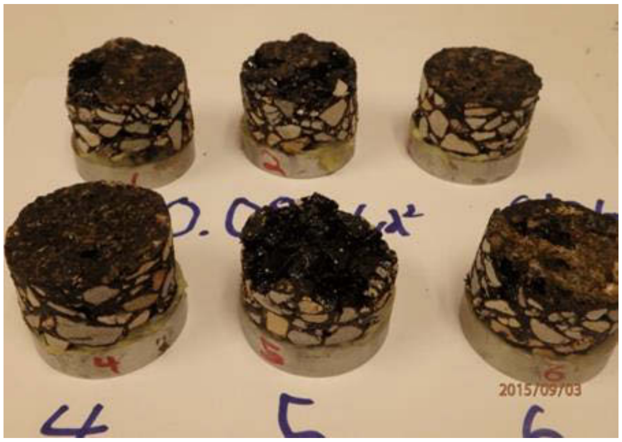


Figure 9 Schematic and Photo of 2-inch Core Locations for Bond Strength Testing

Test Date	8/21/2015	Age=7 days	Pavement Surface Temp = 132° F
Test #	**Section**	**Load (kN)**	**Comments**
1	N9A	0	Broke in OGFC without registering load
2	N9A	0	Broke in OGFC without registering load
3	N9A	0	Broke in OGFC without registering load
4	N9A	0	Broke in OGFC without registering load
5	N9A	0	Broke in OGFC without registering load
6	N9A	0	Broke in OGFC without registering load
7	N9B	0	Broke in OGFC without registering load
8	N9B	0	Broke in OGFC without registering load
9	N9B	0	Broke in OGFC without registering load
10	N9B	0	Broke in OGFC without registering load
11	N9B	0	Broke in OGFC without registering load
12	N9B	0	Broke in OGFC without registering load

NOTE: The OGFC temperature was too high and no load was registered by the testing device because the OGFC layer was stretching and tearing.

Figure 10 Photo and Table Record of First Test Day



Test Date	8/28/2015	Age=14 days	Pavement Surface Temp = 80°F
Test #	**Section**	**Load (kN)**	**Comments**
1	N9A	0.2	Broke at tack interface
2	N9A	0.3	Broke at tack interface
3	N9A	0.2	Broke at tack interface
4	N9A	0.2	Broke at tack interface
5	N9A	0.2	Broke in OGFC
6	N9A	0.2	Broke at tack interface
7	N9B	0.3	Broke at tack interface
8	N9B	0.4	Broke in OGFC
9	N9B	0.3	Broke in OGFC
10	N9B	0.3	Broke in OGFC
11	N9B	0.2	Broke at tack interface
12	N9B	0.2	Broke in OGFC

NOTE: Cooler temperatures led to measurable loads on the testing device, but the values are very small considering the maximum load of the device (100 kN). There is a visual difference between tests ran on N9A and N9B. A majority of N9A broke at the tack interface. N9B had a higher tack rate, and the majority of the tests broke in the OGFC above the interface.

Figure 11 Photos and Table Record of Second Test Day

Test Date	9/17/2015	Age=34 days	Pavement Surface Temp = 65°F
Test #	**Section**	**Load (kN)**	**Comments**
1	N9A	0.7	Broke at tack interface
2	N9A	0.7	Broke at tack interface
3	N9A	0.8	Broke at tack interface
4	N9A	0.6	Broke at tack interface
5	N9A	0.9	Broke at tack interface
6	N9A	0.8	Broke at tack interface
7	N9B	1.2	Broke in OGFC
8	N9B	1.0	Broke at tack interface
9	N9B	1.0	Broke in OGFC
10	N9B	0.9	Broke in OGFC
11	N9B	1.0	Broke in OGFC
12	N9B	0	Broke in dense-mix

NOTE: Similar results to previous testing date. Lower pavement temperatures recorder higher peak loads. There was a visual difference between N9A and N9B specimens.

Figure 12 Photos and Table Record of Third Test Day

Test Date	10/19/2015	Age=66 days	Pavement Surface Temperature = 60°F
Test #	**Section**	**Load (kN)**	**Comments**
1	N9A	0.1	Broke at epoxy/OGFC
2	N9A	0.1	Broke at epoxy/OGFC
3	N9A	0.6	Broke at epoxy/OGFC
4	N9A	0.3	Broke at epoxy/OGFC
5	N9A	0.9	Broke at epoxy/OGFC
6	N9A	0	Broke during coring
7	N9B	1.6	Broke at epoxy/OGFC
8	N9B	0.6	Broke at tack interface
9	N9B	0	Broke at epoxy/OGFC
10	N9B	1.0	Broke at epoxy/OGFC
11	N9B	0.6	Broke at epoxy/OGFC
12	N9B	0.9	Broke at tack interface

FIELD NOTE: Had trouble with specimens breaking at epoxy/OGFC interface. There appears to be less bond area for epoxy. Coring, gluing, and testing were all done before sunrise, so epoxy may not have had ample time to harden before testing or the pavement surface was not dry.

Figure 13 Photo and Table Record of Fourth Test Day

Lessons learned as the testing progressed are listed below.

- Pavement surface temperature influenced the test. The surface OGFC mix pulled apart when the mix temperature was high. The procedure was revised to perform all remaining tests in early morning.
- The available test device was built to measure Portland cement concrete pull-out strength. The load range was too high for asphalt mixture testing.
- Better results were achieved with a rapid pull rate. The asphalt mixture tended to stretch and pull apart at slow loading rates.
- The test method needs further development with a smaller load range and consistent rapid tensile pull rate.

General Field Performance

The reported ride for Section N9A is substantially different from the reported ride for Section N9B as shown in Figure 14. The results raise the question of how these two performance traces differ as the measurement travels from N9A to N9B. After more detailed examination, it was noted that the beginning of Section N9A was very rough due to the failure of 6-inch full depth core locations that were taken in the wheel path. The ride measurements for one day reported on 10-ft increments over the entire 200-ft N9 section is shown in Figure 15 and shows the high roughness in the first portion. The reported ride values shown in Figure 14 for section N9A are the average of the ride measurements across the entire 100 ft section, but that average value reflects a very rough surface in the first 30 ft of the section and then tapers to low roughness for the remainder of the section. Omitting the failed transition at the beginning, the ride of Section N9 did not change during the traffic period.

Figure 14 also shows the rutting and cracking performance of the section. The rutting of the OGFC surface was less than 1 mm (0.04 in.). Measured cracking remained constant at 2.4% and was limited to reflective cracking from the underlying pavement.

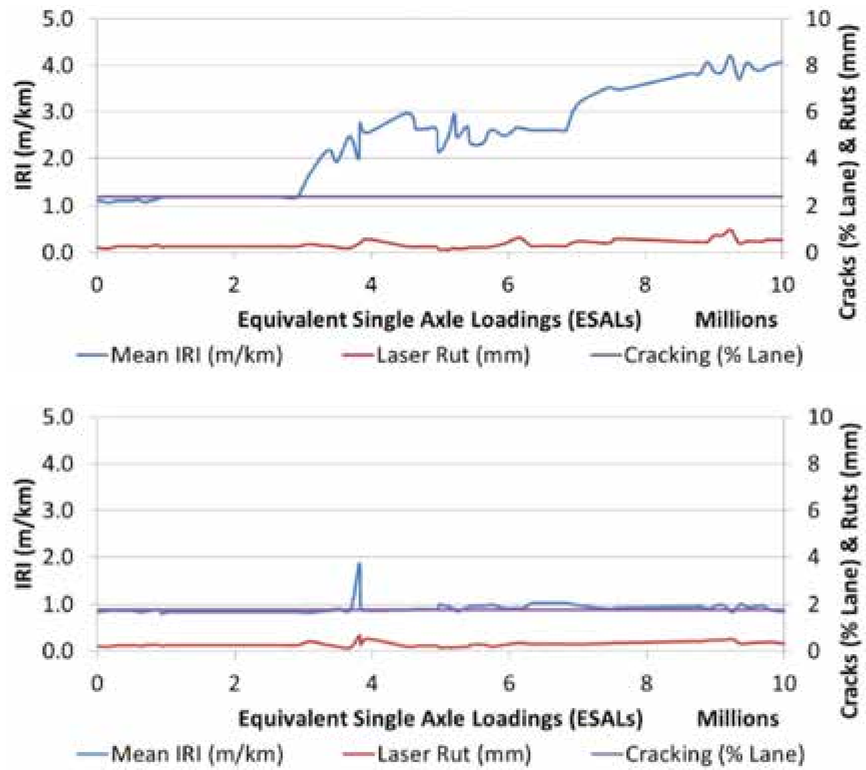


Figure 14 Field Performance of Section N9A (Top) First 100 Ft and N9B (Bottom) Last 100 Ft

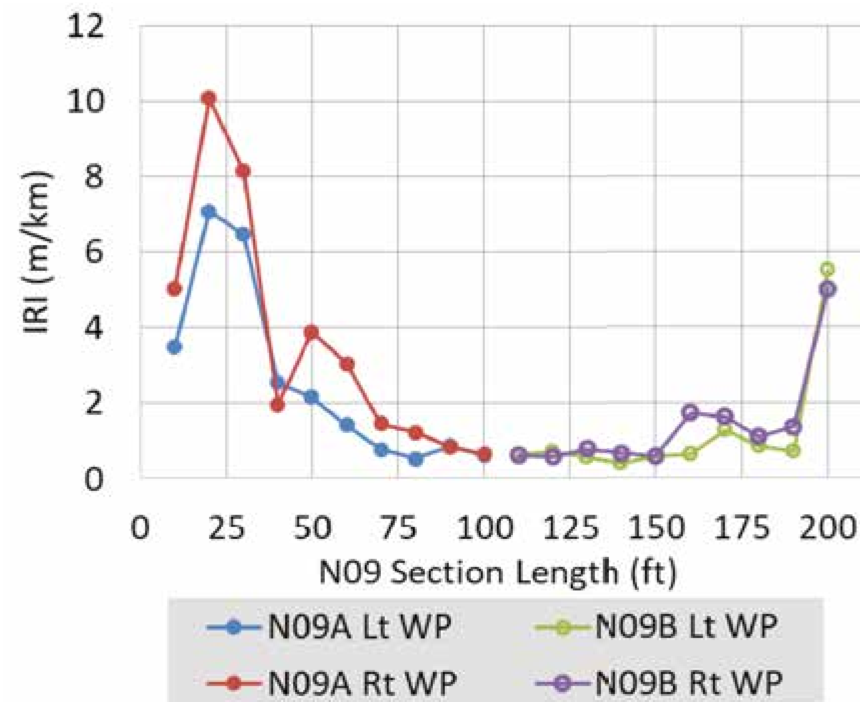


Figure 15 IRI Trace for November 30, 2017

10.7 Summary

Based on the measurements and observations reported above, the following conclusions can be made for the objectives of this study.

- Friction performance was very good, but the DFT showed a greater reduction in friction compared to the locked-wheel skid trailer. The surface friction performance was better than previous Oklahoma sections on the Test Track and even out-performed an experimental section of high friction surface treatment that used Oklahoma flint.
- Surface macro-texture was very good and the CTM measurements appear to be a better measure of texture on the OGFC surface.
- A field pull-out test to measure tack coat bond strength was not completely successful but warrants additional study. The test results, both visual and measured values, show that the higher tack coat rate did create better tensile bond between the OGFC and milled surface.

10.8 References

1. Heitzman, M., and M. Vrtis. *Development of Alternative High Friction Surfaces for Oklahoma*. Final Report FHWA-OK-15-10, ODOT SP&R Item Number 2269. Oklahoma Department of Transportation, Oklahoma City, 2015.
2. Texas Department of Transportation. *Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges. Section 342, Permeable Friction Course*, November 2014.

3. Sheehy, E. *Case Study: High RAP Pilot Project.* Presented March 6, 2013 at New Jersey Asphalt Paving Conference. Accessed August 20, 2014. https://cait.rutgers.edu/system/files/u10/High_RAP_NJDOT_--_Sheehy.pdf.

CHAPTER 11 TENNESSEE DEPARTMENT OF TRANSPORTATION THINLAY EXPERIMENT

11.1 Objective

The objective of this study was to determine if a thin-lift asphalt overlay would have satisfactory performance when placed as a thicker surface lift. The Tennessee Department of Transportation's (TDOT) conventional 4.75 mm mix is placed 0.75 inches thick and the Test Track study section was placed 1.25 inches thick. The criterion for satisfactory performance was the surface layer's ability to resist rutting. Other standard pavement performance criteria such as smoothness, friction, surface texture, and cracking were also measured and monitored.

11.2 Materials

TDOT prepared the job mix formula (JMF) for this study using their 75-blow Marshall mix design method. The aggregate blend, based on total weight of aggregate, consisted of 59% hard limestone, 10% soft limestone, 15% natural sand, and 16% fine (minus 5/16-in) reclaimed asphalt pavement (RAP). The hard limestone was obtained from Aggregates USA in Springfield, Tennessee, the soft limestone was from Aggregates USA in Dickson, Tennessee, and the natural sand was from Pine Bluff Sand and Gravel in Springfield, Tennessee. The fine RAP was obtained from an NCAT stockpile.

Figure 1 displays the gradation of the aggregate blend and clearly shows the predominance of the fraction retained on the #8 screen as 42% of the total blend. Based on JMF source gradations, 87% of the #8 retained aggregate is from the hard limestone stockpile. The Marshall mix design determined the optimum binder content to be 6.8% by weight of mix and was a blend of 87% PG 64-22 virgin binder and 13% RAP binder. The true grade of the RAP binder was PG 103-13. A southeast region standard PG 67-22 virgin binder was used as a reasonable alternative to the mix design PG 64-22 for production of the Test Track Section S4 mixture. Volumetric properties of the mixture are shown in Table 1. Marshall stability was 3900 pounds and flow was 14.5 (0.01-inch units). This JMF was completed by TDOT just ahead of test section construction, so the NCAT lab did not have time to verify the JMF using aggregate delivered to the Test Track.

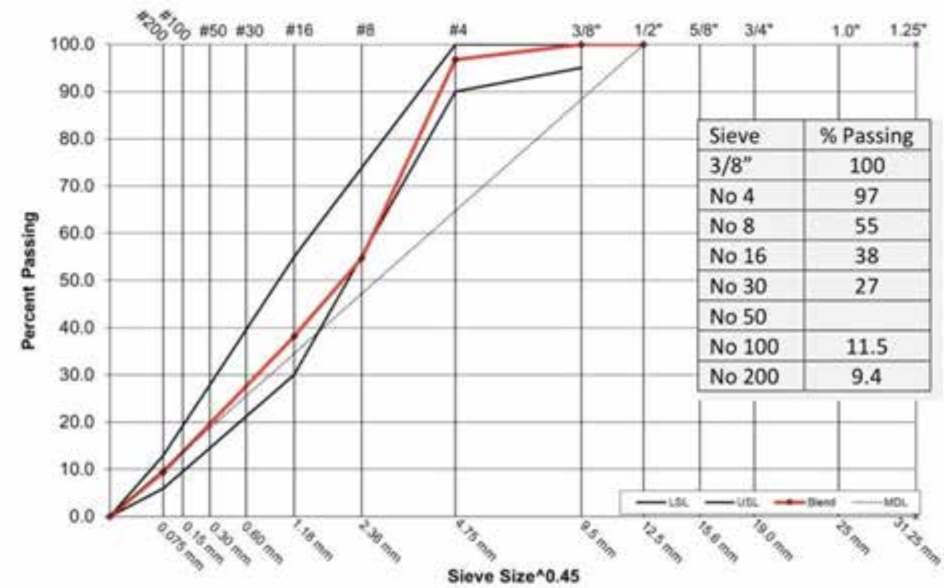


Figure 1 JMF Aggregate Gradation

Table 1 Volumetric Properties of the Mix Design

Air Voids, Va	4.0%
Voids in the Mineral Aggregate, VMA	19.3%
Voids Filled with Asphalt, VFA	79.1%
Volume of Effective Binder, Vbe	15.3%
Dust to Effective Asphalt Ratio, D:A	1.4

11.3 Construction

The test section was constructed as a mill and inlay on a 24-inch thick asphalt pavement on Test Track Section S4. Conventional milling removed 1.25 inches and the TDOT thin lift was placed 1.50 inches thick based on as-built measurements. The climate conditions for construction on August 13, 2015 were ideal for paving (95°F high temperature and no rainfall).

Quality control testing of the production mix identified an increase in the fraction retained on the #4 from 3% JMF to 18% at production, a decrease in the percent passing the #200 from 9.4 JMF to 8.1, reduction in the binder content from 6.8% JMF to 6.4% (4.9% to 4.7% effective binder), and reduction in lab compacted air voids from 4.0% to 2.6%. This mixture had a noted reduction in VMA from 19.5% JMF to 13% at production and is partially a result of the change in the coarse portion of the gradation, which brought the gradation closer to the maximum density line. The differences between the target JMF and construction QC tests are most likely a result of differences between the aggregate sampled for mix design in Tennessee and the aggregate delivered for construction. Mixture temperature at the plant was 310°F and the compacted density of cores was 95% of the theoretical maximum specific gravity (G_{mm}).

11.4 Laboratory Performance of Production Mixture

Hamburg Wheel-Track Test Results

The Hamburg Wheel-Track Test was performed to determine both the rutting and stripping susceptibility of the mixture tested for this project. Testing was performed in accordance with AASHTO T 324-16 at a test temperature of 50°C. Two replicates were tested with each replicate consisting of two trimmed specimens (four specimens total per mix). The specimens were compacted using a Superpave gyratory compactor (SGC) to a diameter of 150 mm and a height of 60 mm. The specimen ends were then trimmed to fit in the Hamburg molds for testing. The measured air voids on the Hamburg specimens ranged from 6.9 to 7.2%.

The rut depth versus wheel passes plot for both replicates is shown in Figure 2. The average rut depth for both replicates was 2.6 mm, and no sign of stripping was evident from the test results. A maximum rut depth of 12.5 mm is a commonly used failure threshold in the Hamburg test (*1*). Hence, the Section S4 surface mix appeared resistant to both rutting and stripping in the Hamburg test.

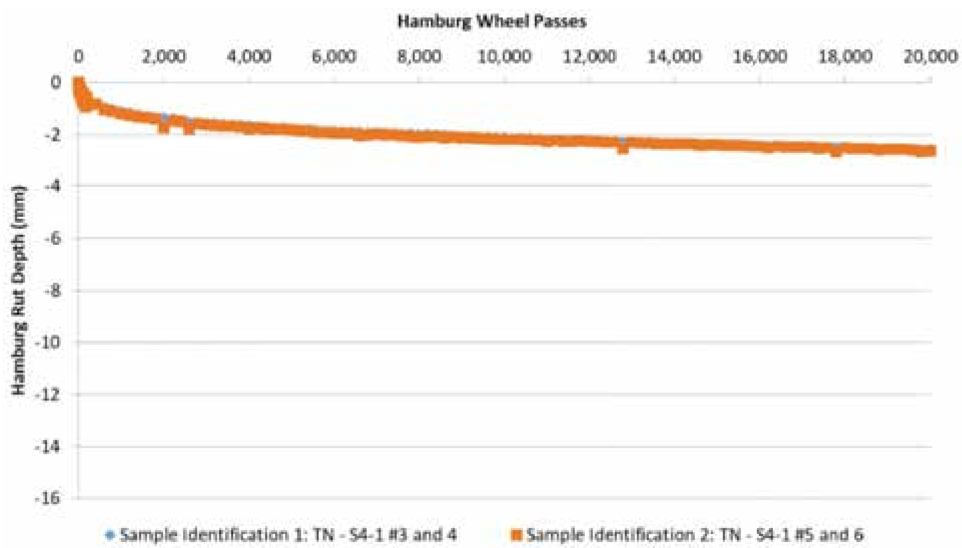


Figure 2 Hamburg Rut Depth versus Wheel Passes Section S4-1

Overlay Tester Results

The overlay tester (OT) is a device designed to simulate accelerated reflective cracking in asphalt concrete overlays. The two states that currently have a specification for the OT are Texas and New Jersey. No national standard (AASHTO or ASTM) currently exists for the OT. The Texas Department of Transportation (TxDOT) runs the test in accordance with the Tex-248-F standard, while the New Jersey Department of Transportation (NJDOT) runs the test in accordance with the NJDOT B-10 standard. NCAT conducted the test according to the Texas standard using a fixture and software within the IPC Global® Asphalt Mixture Performance Tester (AMPT). For this study, SGC samples were compacted to a target height of 125 mm. Each SGC sample was trimmed into two OT specimens to the following dimensions: 150 mm long, by 76 mm wide, by 38 mm tall. The measured air voids for the cut specimens ranged from 6.4 to

7.7%. Five replicate specimens were tested for this mixture. Specimens were tested at 25°C at a frequency of 0.1 Hz with a sawtooth waveform and a 0.025-inch maximum opening displacement.

Based on literature, there is not a definitive pass/fail criterion for a mixture being resistant to cracking in the OT. NJDOT currently recommends greater than 150 cycles to failure in the OT for high RAP surface mixtures with a PG 64-22 binder and greater than 175 cycles to failure for surface mixtures with a PG 76-22 binder (*2*). TxDOT's 2014 specification requires a minimum of 300 cycles to failure for their thin overlay mixtures (*2*).

The OT results for the Section S4 surface mix are summarized in Table 2. With the removal of a statistical outlier (ASTM E178-16a method), the average cycles to failure in the OT was 15 cycles. This is significantly below the threshold values suggested in literature, and in conjunction with the very low Hamburg rut depths, suggests this is a stiff mix that may benefit from design modifications to add flexibility. The mix could be modified to extend the time before reflective cracking appears without exceeding rut depth criteria.

Table 2 Summary of OT Cycles to Failure for Section S4

Specimen ID	Specimen Air Voids (%)	Peak Load (lb)	Cycles to Failure (93% Load Reduction)	Average Cycles to Failure (minus outlier)
S4-1 #7A	7.2	1,332	26	15
S4-1 #8A	7.5	1,340	5	
S4-1 #8B	6.4	1,284	18	
S4-1 #9A	7.7	1,327	12	
S4-1 #10B	7.5	1,271	58*	

*ASTM E178-16a statistical outlier with 90% confidence

Dynamic Modulus Results

Dynamic modulus (E*) testing was performed in accordance with AASHTO TP79-15 (2016) to characterize the stiffness of the Section S4 surface mix to be used as an input for mechanistic-empirical (ME) pavement design. Specimens were fabricated from re-heated plant produced mix in accordance with AASHTO PP60-14 (2016), while data collection and analysis were performed in accordance with AASHTO PP61-13 (2016).

Table 3 summarizes the raw E* and phase angle data collected from the S4 mix and Table 4 summarizes the specimen volumetrics that are required to generate the E* mastercurve. The E* mastercurve for S4 is shown in Figure 3. The E* mastercurve can be solved for the 30 testing conditions (five temperatures x six frequencies) required for the Mechanistic Empirical Pavement Design Guide (MEPDG). A summary of the E* input from this mix is provided in Table 5.

Table 3 Summary of Raw E* Data

Test Conditions		S4-1 #8		S4-1 #9		S4-1 #10	
Temp (°C)	Freq (Hz)	E*(ksi)	Phase Angle (Degrees)	E*(ksi)	Phase Angle (Degrees)	E*(ksi)	Phase Angle (Degrees)
4	0.1	1,461.4	11.83	1,521.6	11.89	1,591.6	11.53
4	1	1,879.7	9.35	1,965.3	9.45	2,040.4	9.04
4	10	2,296.8	7.62	2,411.8	7.79	2,484.9	7.38
20	0.1	540.8	23.05	565.9	22.87	586.0	22.91
20	1	873.1	18.32	910.0	18.39	940.9	18.29
20	10	1,282.0	14.18	1,336.8	14.36	1,375.4	14.09
40	0.01	53.4	29.01	56.2	27.91	57.2	28.00
40	0.1	105.2	31.89	111.1	30.72	111.7	31.05
40	1	214.7	31.11	225.2	30.22	227.6	30.60
40	10	418.6	27.39	436.1	27.03	442.7	27.22

Table 4 Summary of E* Specimen Volumetrics

Sample ID	Sample Air Voids, %	QC G_{sb}	QC P_b	G_{mm}	G_{mb}	VMA	VFA
S4-1 #8	6.9	2.550	6.4	2.431	2.263	16.9	59.2
S4-1 #9	6.9	2.550	6.4	2.431	2.263	16.9	59.2
S4-1 #10	6.8	2.550	6.4	2.431	2.266	16.8	59.6

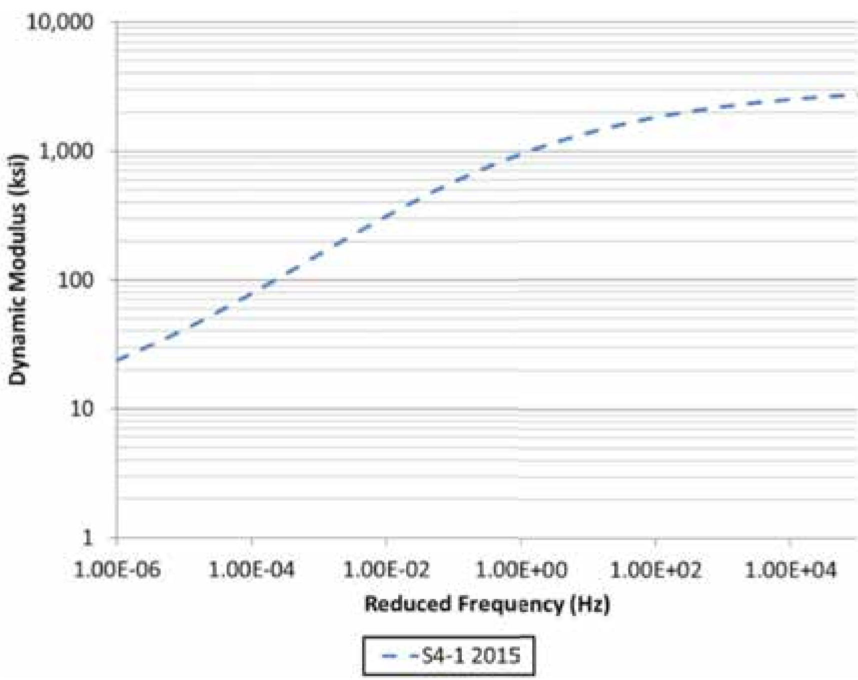


Figure 3 E* Mastercurve

Table 5 E* Input for M-E Design

Temperature (°C)	Temperature (°F)	Frequency (Hz)	Shift Factor	Reduced Frequency (Hz)	E* (ksi)	E* (MPa)
-10.0	14	25	4.459	7.19E+05	2,863.6	19,750.4
-10.0	14	10	4.459	2.88E+05	2,804.7	19,344.2
-10.0	14	5	4.459	1.44E+05	2,754.0	18,994.6
-10.0	14	1	4.459	2.88E+04	2,613.3	18,023.7
-10.0	14	0.5	4.459	1.44E+04	2,541.7	17,530.2
-10.0	14	0.1	4.459	2.88E+03	2,347.8	16,192.4
4.4	40	25	2.192	3.89E+03	2,387.1	16,463.6
4.4	40	10	2.192	1.56E+03	2,263.0	15,607.8
4.4	40	5	2.192	7.78E+02	2,160.4	14,900.5
4.4	40	1	2.192	1.56E+02	1,895.3	13,071.9
4.4	40	0.5	2.192	7.78E+01	1,771.0	12,214.3
4.4	40	0.1	2.192	1.56E+01	1,466.4	10,113.5
21.1	70	25	-0.148	1.78E+01	1,492.4	10,293.0
21.1	70	10	-0.148	7.12E+00	1,314.7	9,067.7
21.1	70	5	-0.148	3.56E+00	1,181.0	8,145.6
21.1	70	1	-0.148	7.12E-01	884.3	6,099.1
21.1	70	0.5	-0.148	3.56E-01	766.8	5,288.6
21.1	70	0.1	-0.148	7.12E-02	528.3	3,643.4
37.8	100	25	-2.236	1.45E-01	627.3	4,326.7
37.8	100	10	-2.236	5.80E-02	501.9	3,461.3
37.8	100	5	-2.236	2.90E-02	419.1	2,890.6
37.8	100	1	-2.236	5.80E-03	267.1	1,841.9
37.8	100	0.5	-2.236	2.90E-03	217.6	1,500.7
37.8	100	0.1	-2.236	5.80E-04	133.5	920.4
54.4	130	25	-4.113	1.93E-03	192.5	1,327.4
54.4	130	10	-4.113	7.72E-04	145.6	1,004.3
54.4	130	5	-4.113	3.86E-04	117.8	812.4
54.4	130	1	-4.113	7.72E-05	72.5	500.3
54.4	130	0.5	-4.113	3.86E-05	59.3	409.1
54.4	130	0.1	-4.113	7.72E-06	38.2	263.8

11.5 Field Performance

Pavement smoothness, measured as international roughness index (IRI), remained constant at 0.6 m/km for the entire period of truck traffic loading. Rutting began to appear in the summer of 2016 when the pavement temperature increased. Rutting over the two-year traffic period did not exceed 1.2 mm and was likely related to normal post-construction traffic consolidation of the surface mix. No cracking was identified over the two-year test cycle. Roughness and rutting performance are shown in Figure 4.

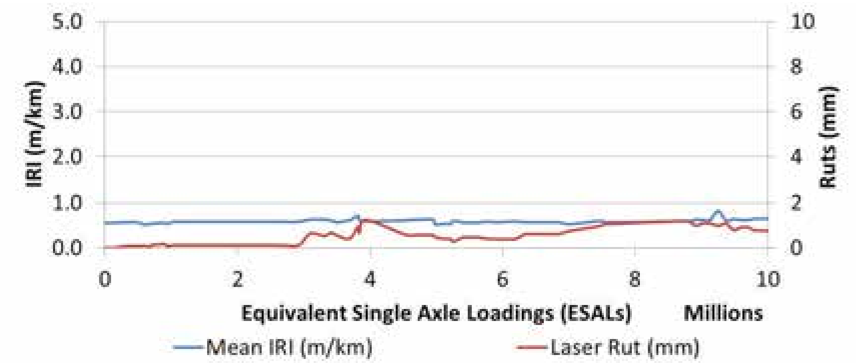


Figure 4 Test Section S4 TDOT Thin Lift Pavement Performance

Accelerated laboratory friction performance testing with a dynamic friction tester (DFT) was performed on plant production mix sampled during construction. The surface of the compacted mix achieved an early peak friction value of 0.45 DFT(40) and gradually decreased to a terminal friction of 0.32 DFT(40) after polishing with the NCAT Three Wheel Polishing Device (TWPD) as shown in Figure 5.

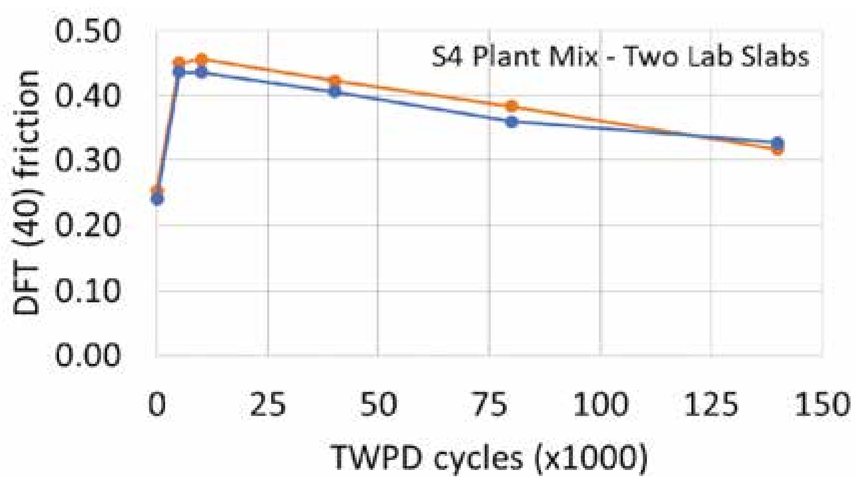


Figure 5 Accelerated Lab Friction Performance on Plant Mix

Field friction performance measured monthly with the Alabama Department of Transportation locked wheel skid trailer showed a nominal friction of 47 SN40R for the entire period of traffic. Individual values generally ranged between 44 and 50 with the highest measured friction of 53 and lowest value of 43. Field friction performance measured with a DFT showed the surface maintained friction between 0.40 and 0.43 DFT(40) in the second year of traffic. DFT measurements in the first year were suspect due to equipment problems and not used for analysis. Field friction measurements are shown in Figure 6.

The noted difference between decreasing lab friction performance in Figure 5 and field friction performance in Figure 6 is related to the physical location of Section S4. Based on the good correlation in past studies between lab friction and Test Track friction with sections placed in the curves, it is probable that the truck traffic on the tangent S4 section is insufficient to accelerate polishing the aggregate exposed on the surface. If the thin lift was placed on a curved section where transverse tire forces interact with the pavement surface, the field performance would be similar to the lab performance based on previous friction studies.

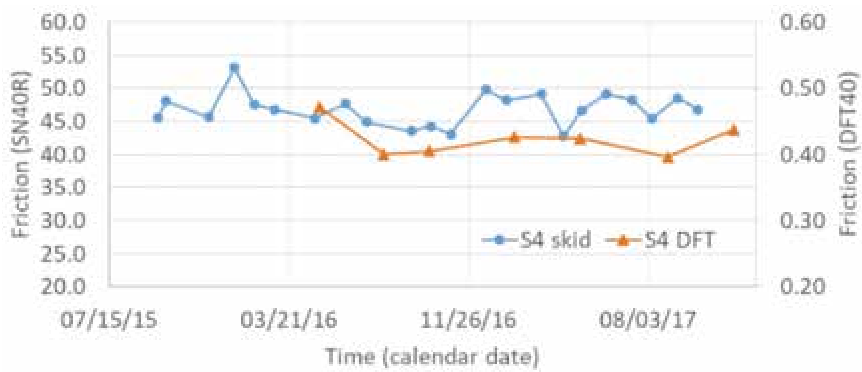


Figure 6 Field Friction Measurements

Field macrotexture measured with the automated distress van ranged from 0.2 to 0.5 mm mean profile depth (MPD). Field macrotexture measured with the circular texture meter (CTM) steadily increased from 0.2 mm to 0.6 mm MPD. Both plots are shown in Figure 7. Differences between the measurements are a result of variances between the laser on the van and the laser on the CTM. The CTM texture measurement is considered a more accurate value based on observed changes in the pavement surface as shown in Figure 8. The van laser creates a data set of measurements taken every 0.4 mm with a laser spot diameter of 2.0 mm and vertical resolution of 0.05 mm. The van texture measurement is computed for each 100 mm of linear measure and is expressed as average texture depth in millimeters. The CTM creates a data set of measurements taken every 0.87 mm with a laser spot diameter of 0.07 mm and vertical resolution of 0.003 mm. The CTM MPD computed for the 12-inch diameter circular path of the test.

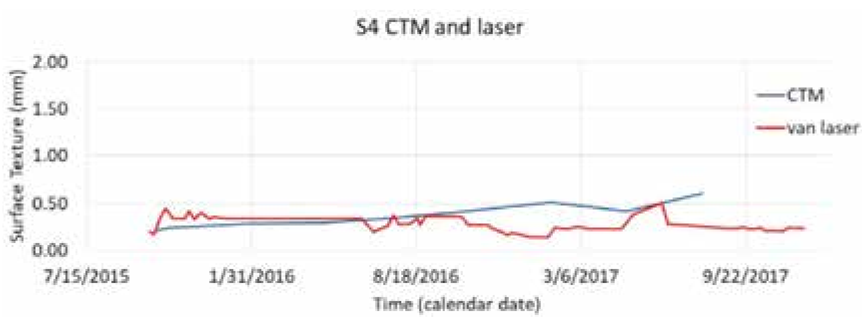


Figure 7 Field Surface Macrotexture Measurements

Figure 8 Change in Surface Appearance

Pre-traffic noise measurement using the on-board sound intensity (OBSI) system was 94 dB(A). As the mastic was removed and exposed more surface texture, the OBSI noise increased to 97 dB(A) on the average, ranging from 95 to 98 dB(A). The OBSI 3rd octave data was relatively flat, starting at 69 dB(A) for 315 Hz frequency, increasing to 89 dB(A) for 800 Hz, and dropping to 72 dB(A) for 4000 Hz prior to traffic. During the two-year trafficking cycle, the 3rd octave data measured as high as 76 dB(A) for 315 Hz, increasing to 93 dB(A) for 800 Hz, and dropping to 76 dB(A) for 4000 Hz. The impedance tube sound absorption measurements agreed with the OBSI values. Generally, 3% absorption was measured at 315 and 400 Hz, no absorption was measured at 630 and 800 Hz, and 13% absorption was measured at 1600 Hz. These noise characteristics are reasonable for a dense-graded small nominal maximum aggregate size thin lift.

11.6 Summary

- The 4.75 mm mix performed satisfactorily when placed 1.5 inches thick.
- There was significant VMA loss with the field produced mix related to a difference in stockpile gradations.
- The field tests measured good smoothness, low rutting, and no cracking. Laboratory performance testing indicated good rutting resistance but poor reflective cracking resistance.
- The mixture surface maintained a stable friction value during truck traffic polishing. The accelerated laboratory friction test did show friction loss, but the field friction performance did not show friction loss due to being in the tangent section of the track.
- The mixture surface texture tests measured low macrotexture but appeared to be increasing as the mastic continues to wear off the surface.
- Tire-pavement noise testing measured 94 dB(A) and increased to 97 dB(A).

11.7 References

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CHAPTER 12 VIRGINIA DEPARTMENT OF TRANSPORTATION COLD CENTRAL PLANT RECYCLING AND STABILIZED BASE EXPERIMENT

12.1 Background and Objectives

Cold central plant recycling (CCPR) is a process whereby the asphalt is milled from the roadway and brought to a centrally located recycling plant that incorporates recycling agents and additives into the material (*1*). The most common recycling agents and additives include foamed asphalt, emulsified asphalt, hydraulic cement, fly ash, and lime (*2*). This approach allows for material removal from the roadway so that the underlying foundation may be stabilized or replaced as needed. The reclaimed asphalt pavement (RAP) is used as a recycled layer with very little added virgin material. This process also allows for existing RAP stockpiles to be used in new construction or rehabilitation projects (*1*).

CCPR has not been widely used in rehabilitating asphalt pavements, especially on high volume roadways. To evaluate CCPR under accelerated traffic conditions, the Virginia Department of Transportation (VDOT) sponsored three test sections at the NCAT Test Track, complementing an existing field project using CCPR on I-81 in Virginia. The RAP for the CCPR layers in the Test Track sections came from the I-81 project where RAP was milled to a depth of 10 inches and scalped over a 19 mm sieve to provide enough material to perform a mix design. At the Test Track, the RAP was processed through a pro-sizer with 100% passing the 12.5 mm sieve. The primary objectives were to characterize the field performance and to estimate the structural characteristics and contribution of the VDOT CCPR sections placed at the track. The sections were built as part of the 2012 Test Track experiment and performed so well during that cycle that VDOT elected to leave them in place for another 10 million equivalent single axle loads (ESALs) during the 2015 cycle.

12.2 Test Sections

The test sections in this experiment are shown in Figure 1, while the as-built properties are listed in Table 1. Each section featured a stone matrix asphalt (SMA) surface and Superpave dense-graded asphalt concrete (AC) layers above the CCPR layer. The nominal maximum aggregate sizes (NMAS) are listed with the layer descriptions in Table 1. The CCPR was 100% RAP with 2% foamed performance grade (PG) 67-22 asphalt binder and 1% Type II hydraulic cement, as noted in Table 1. Sections N3 and N4 were constructed on top of a 6-inch crushed granite aggregate base layer, while S12 was built on a cement-stabilized base layer. The base stabilization was done in-place using a reclaimer to simulate full-depth reclamation (FDR). Approximately 6 in. of crushed granite aggregate base and 2 in. of the subgrade were treated in place with 4% (by weight) Type II hydraulic cement. The FDR layer had a seven-day average compressive strength of 256 psi, which was below the maximum allowable of 350 psi used during the mix design process. All three sections were constructed on the same subgrade native to the track and classified as an A-4 soil (*3*). Sections N3 and N4 were designed to evaluate the difference between 4 in. and 6 in. of AC over 5 in. of CCPR. Sections N4 and S12 were designed to determine the differences between 6 in. of aggregate base and 8 in. of cement stabilized base (CSB). It is important to note that Figure 1 represents the average as-built thickness of the entire test section, which reflects natural variation due to standard construction practices at the track. The average represents measurements taken at 12 distinct locations within each section.

As described by Timm et al. (*4*), it is possible to determine the expected service lives of the three Test Track sections by following procedures in the AASHTO 1993 Pavement Design Guide and using VDOT's typical design parameters for asphalt pavements (*5,6*). Structural numbers for Sections N3, N4, and S12 were calculated as 5.1, 4.2, and 5.5, respectively. The soil support value was assumed to be 9400 psi, which represents approximately one third of the falling weight deflectometer (FWD) measured values at the Test Track (*3*). Using inputs for designing an interstate-style pavement with the calculated structural numbers and the assumed soil support value yielded ESAL values ranging from about 3 to 16 million.

Figure 1 also shows the depth of instrumentation used in this investigation. Six horizontal asphalt strain gauges oriented in the longitudinal direction (parallel to traffic) were placed at the bottom of the CCPR layer to capture bending of the asphalt-bound layers. Six vertical strain gauges were installed to capture vertical deflection of the asphalt-bound layers. However, the vertical strain gauges were only functional for a short period of time (i.e., a few weeks), preventing the development of meaningful vertical strain data over time. Earth pressure cells were placed at the top of the base and top of the subgrade to capture vertical pressures transmitted through the sections. Temperature probes were installed after paving at the middle of the composite AC/CCPR to measure mid-depth temperature during testing.

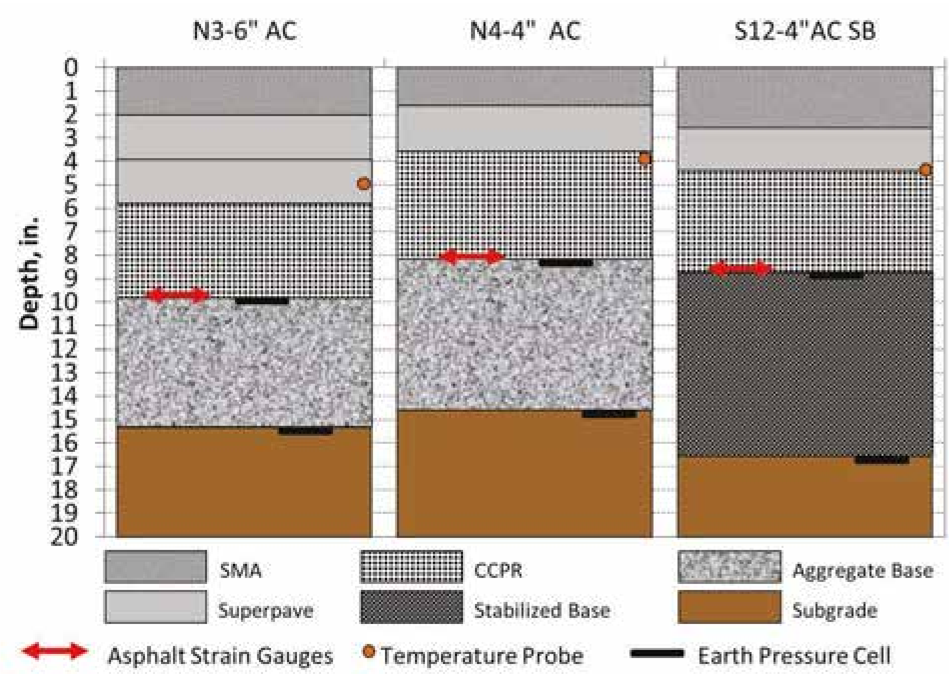


Figure 1 VDOT Experiment Average As-Built Thicknesses and Depth of Instrumentation

Table 1 VDOT Experiment As-Built Layer Properties

Section	N3: 6 in. AC	N4: 4 in. AC	S12: 4 in. AC SB
Layer Description	Lift 1-19 mm NMAS SMA with 12.5% RAP and PG 76-22 binder		
Binder Content, %	6.1	6.0	6.1
Air Voids, %	4.3	4.7	4.2
Layer Description	Lift 2-19 mm NMAS Superpave with 30% RAP and PG 67-22 binder		
Binder Content, %	4.6	4.6	4.7
Air Voids, %	7.1	7.4	6.7
Layer Description	Lift 3-19 mm NMAS Superpave with 30% RAP and PG 67-22 binder		
Binder Content, %	4.4	NA	NA
Air Voids, %	6.4	NA	NA
Layer Description	CCPR-100% RAP with 2% foamed PG 67-22 binder and 1% Type II hydraulic cement		
Layer Description	6 in. crushed granite aggregate base (CGAB)		6 in. CGAB + 2 in. subgrade both stabilized in-place with 4% Type II hydraulic cement
Layer Description	Subgrade – AASHTO A-4 Soil		

12.3 Performance

Through the end of trafficking (November 2017), no cracking was observed in the three CCPR test sections. Figure 2 shows the rutting performance through both research cycles. The increasing rut depths noted between 3 and 4.5 million ESALs and between 8 and 9 million ESALs correspond to the increasing temperatures experienced during the summer months of trafficking during the first test cycle. The sections did not exhibit significantly more rutting during the second test cycle and all sections have exhibited excellent rutting performance. The maximum rut depth was approximately 0.30 inches with very little practical differences between the three sections.

Pavement smoothness, expressed as the International Roughness Index (IRI), is shown in Figure 3. The data indicates relatively little change in smoothness over time through November 2017. Section S12, which included the stabilized base, had an initial roughness of nearly double the other sections. This was caused by a localized low spot approximately 40 to 60 feet into the section that was noted immediately after construction. However, the IRI did not change appreciably during the 2012 cycle. Between the 2012 and 2015 test cycles, the transition zone leading into S12 was milled and inlaid, which noticeably improved the ride quality data. Overall, it is important to note that the IRI of S12 has not changed significantly over time except for the improvement from milling between test cycles.

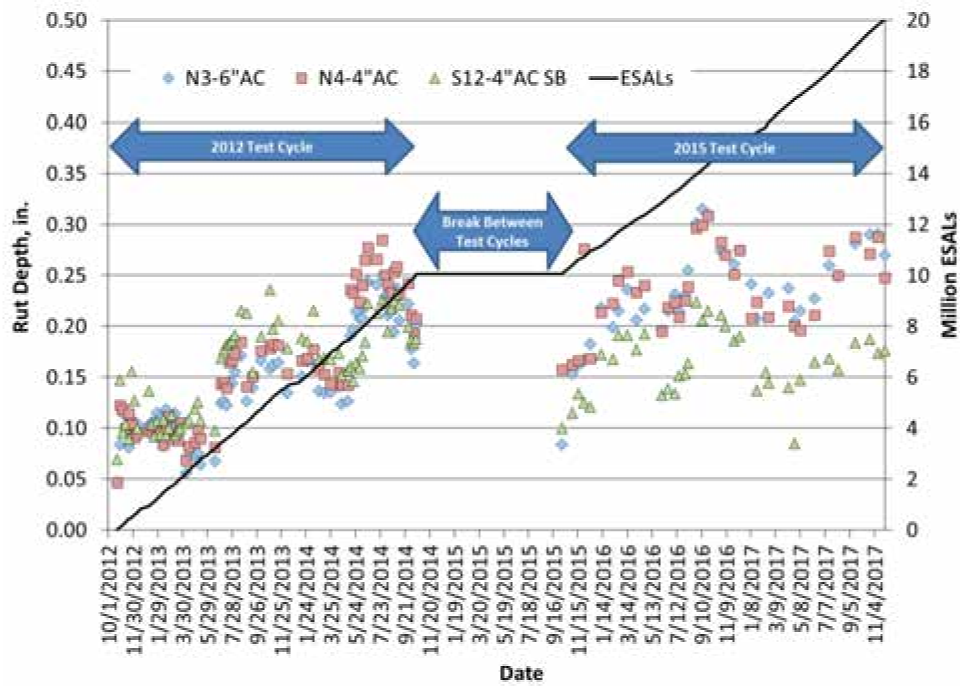


Figure 2 Rutting Performance

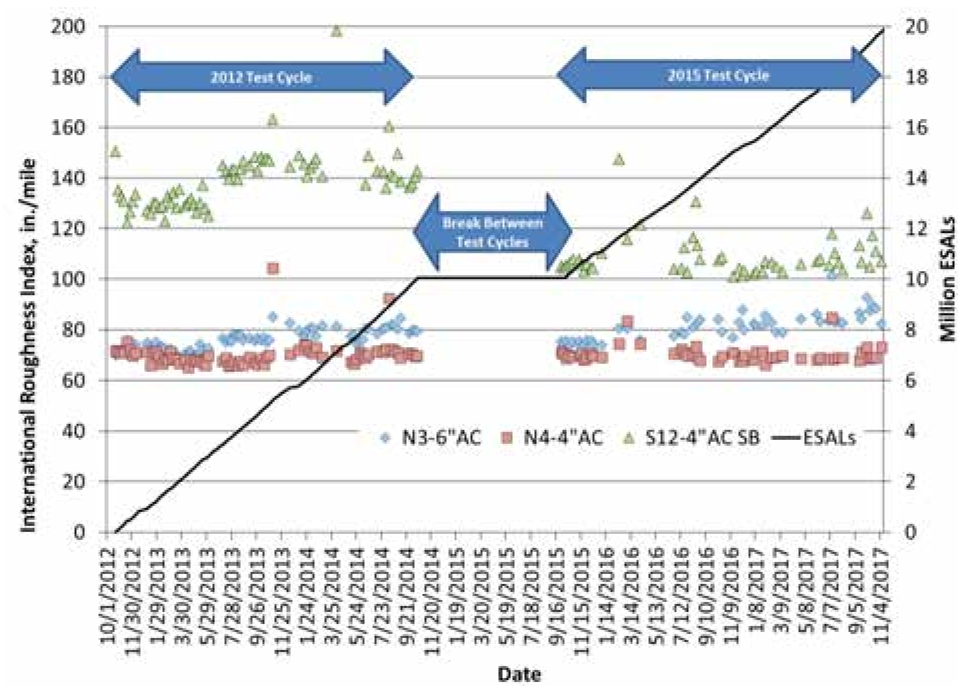


Figure 3 Ride Quality

12.4 Backcalculated Moduli

For backcalculation purposes, each pavement section was treated as a three-layer structure consisting of the AC/CCPR lifts as layer one, the aggregate base (N3 and N4) or stabilized base (S12) as layer two, and the subgrade as layer three. Previous research demonstrated that the CCPR would exhibit a behavior similar to that of AC (*7*), and therefore these layers were combined. Subsequent laboratory dynamic modulus testing of the CCPR by VDOT confirmed that the CCPR exhibited behavior consistent with AC materials and supported the combination of AC and CCPR for backcalculation (*8*).

Figure 4 shows the influence of measured mid-depth temperature on backcalculated AC/CCPR moduli during both test cycles. The sections having an unbound granular base layer (N3 and N4) show the strong influence of measured mid-depth temperature on the modulus, demonstrated by the exponential regression equations and corresponding coefficients of determination (R^2). Very similar behavior has been reported previously for AC materials at the Test Track (*9*). Therefore, it was further justified to consider the CCPR and AC materials as a single layer for backcalculation purposes. Interestingly, the thicker AC section (N3) appears to be slightly more temperature sensitive (i.e., steeper slope) than the thinner AC section (N4). This may be due to a higher percentage of RAP making up the backcalculated layer in N4. Previous studies at the Test Track that compared a virgin AC section to a 50% RAP section found the RAP section to be less temperature sensitive, presumably due to the presence of more aged binder (*10*).

Section S12, having the cement stabilized base, shows a higher modulus and much less temperature sensitivity than the other two sections, as shown in Figure 4. The exponential regression coefficient is lower than the other two sections and the corresponding R^2 is much lower, demonstrating little effect of temperature on the AC modulus. Although it seemed reasonable to expect the modulus of S12 to resemble that of N4 due to the similar thickness of the AC/CCPR layer, it may be inferred that the increased modulus is an artifact of the backcalculation process. Essentially, the AC/CCPR was given a higher apparent modulus to adjust for smaller measured deflections on the stabilized base section. Furthermore, the lower temperature sensitivity observed for Section S12 may be attributed to the presence of the cement stabilized base.

Figure 5 shows the normalized AC/CCPR modulus versus date over both test cycles. Linear trendlines were found for each data set. The sections having an aggregate base (N3 and N4) show virtually no change (i.e., very small negative slopes and extremely low R^2 values) in modulus over time, however, the modulus for the stabilized base section (S12) clearly increased over time. It appears that the cement stabilized layer is curing over time, as expected (*11*). This should result in reduced pavement response measurements that will be presented in the next subsection. In none of the three sections was there a clear, significant, reduction in modulus over time, which indicates good structural health for all three sections.

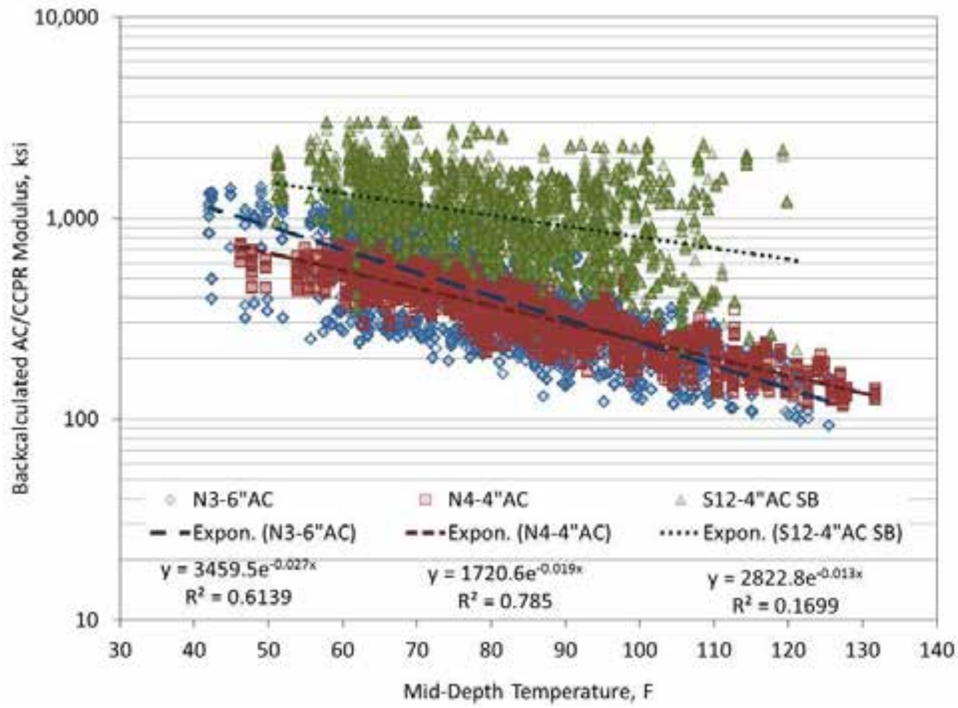


Figure 4 Backcalculated AC/CCPR Modulus vs. Temperature

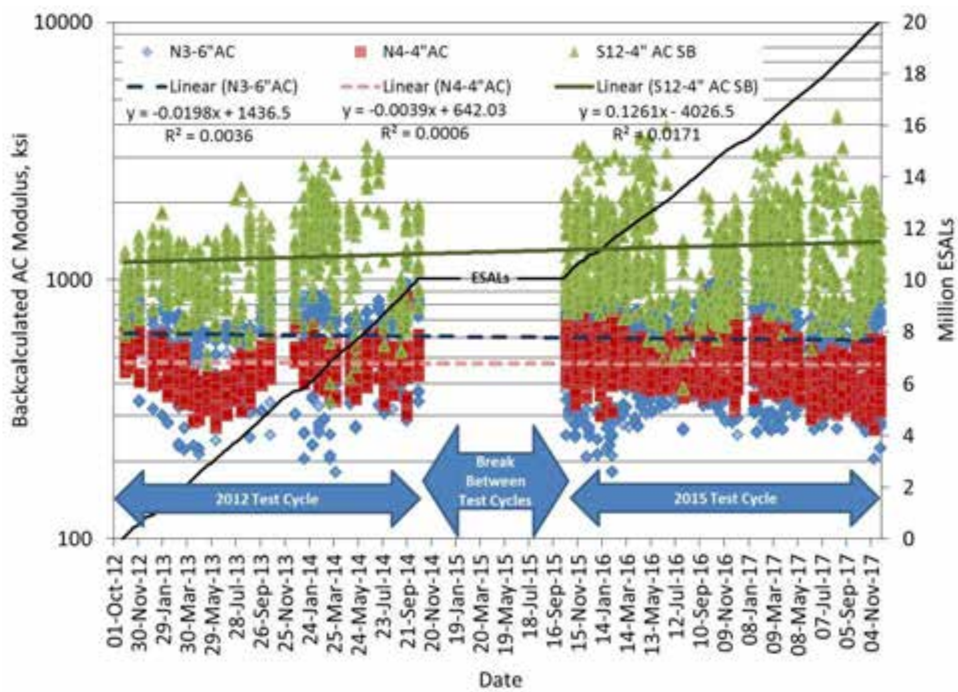


Figure 5 Backcalculated AC/CCPR Modulus at 68°F vs. Date

12.5 Pavement Response

Figure 6 shows the tensile strain response at the bottom of the CCPR layer versus temperature measured during both test cycles. Given the negative exponential relationship between backcalculated modulus and temperature presented above, the strain response was strongly correlated to temperature through exponential regression equations. As expected, the benefit of the additional 2 in. of AC in Section N3 (6 in. AC) as compared to Section N4 (4 in. AC) is clearly seen across the temperature spectrum. At 68°F, N3 had approximately 35% lower strain than N4. Both N3 and N4 exhibited similar temperature sensitivity as demonstrated by the very similar exponential coefficients in their respective regression equations.

Section S12 experienced relatively lower strains than the other sections and demonstrated less sensitivity to temperature. Both characteristics are clearly shown in Figure 6. The exponential regression coefficient of S12 is approximately half that of the other sections. Similar observations were made regarding the backcalculated modulus regression equations. The strain magnitude is also significantly lower in S12 than the other two sections. The differences are less pronounced at colder temperatures but increase as temperatures increase. This is the combined effect of low temperature sensitivity and the stiff base layer producing lower strain levels. The tensile strain is also a function of the underlying supporting layer. In S12, this material is a cement-stabilized base layer, while it is an unbound granular material in N4. Therefore, it is reasonable to expect less strain in the section that has the stiffer underlying material, which limits, to an extent, the tensile strain in the CCPR layer.

Following the temperature normalization procedure described previously, the strain measurements were corrected to a reference temperature of 68°F. The normalized strains at the bottom of the CCPR layer with respect to date are presented in Figure 7, where linear trendlines have been assigned to each data set. Both Section N3 (6 in. AC) and Section N4 (4 in. AC) show a slight increase in strain over time. The slope and relatively low R^2 corresponding to these sections indicate relatively little change in strain over time. The small slope and low R^2 corresponding to Section S12 (4 in. AC, SB) also indicate no appreciable change over time and a healthy pavement structure.

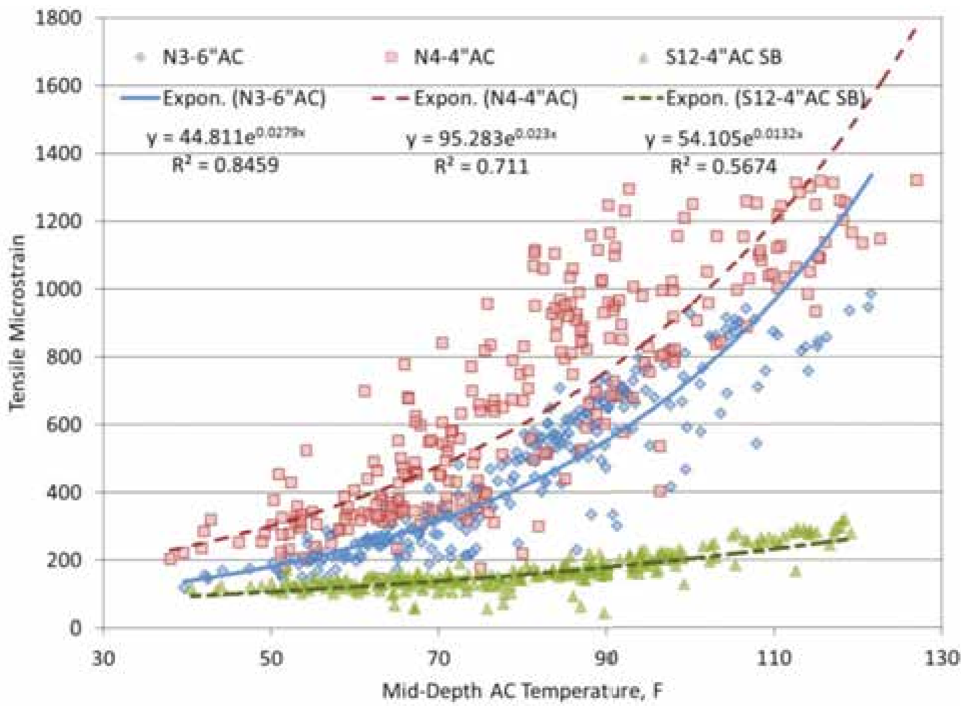


Figure 6 Tensile Strain at the Bottom of the CCPR Layer vs. Temperature

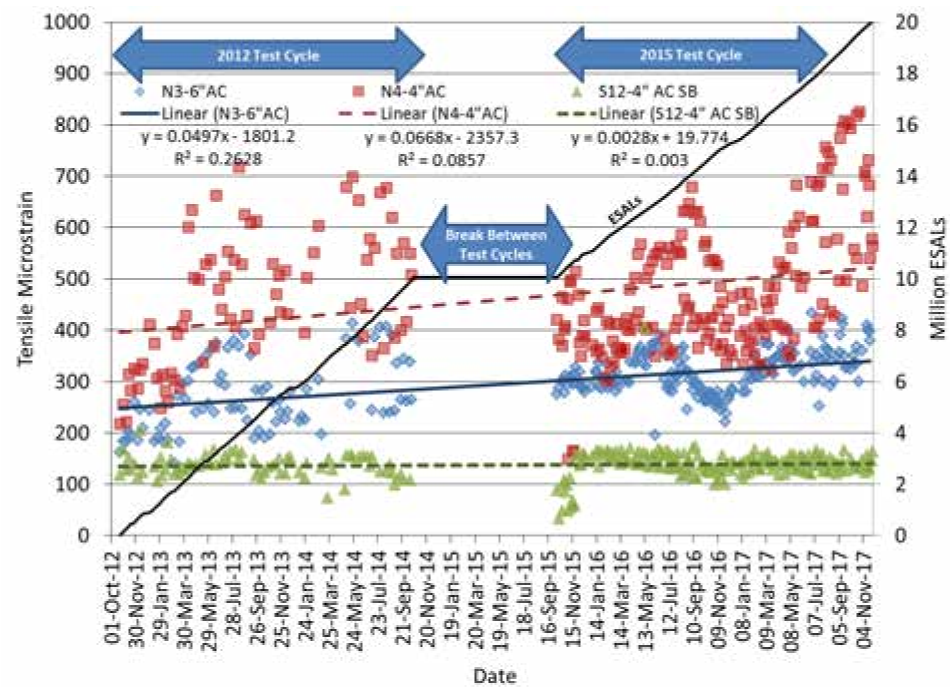


Figure 7 Tensile Strain at the Bottom of the CCPR Layer at 68°F vs. Time/Traffic

12.6 Perpetual Pavement Analysis

Since the CCPR sections at the Test Track were exhibiting excellent performance through two research cycles, analyses were conducted to evaluate whether these sections may be considered as perpetual pavements with respect to bottom-up fatigue cracking. Studying the fatigue performance of a pavement section subjected to high truck traffic volumes is a common practice for asphalt pavements constructed with non-CCPR materials. However, it is not yet known if the CCPR materials used in this study are expected to ultimately fail by fatigue or some other mechanism. The remainder of this subsection was first documented elsewhere (*4*).

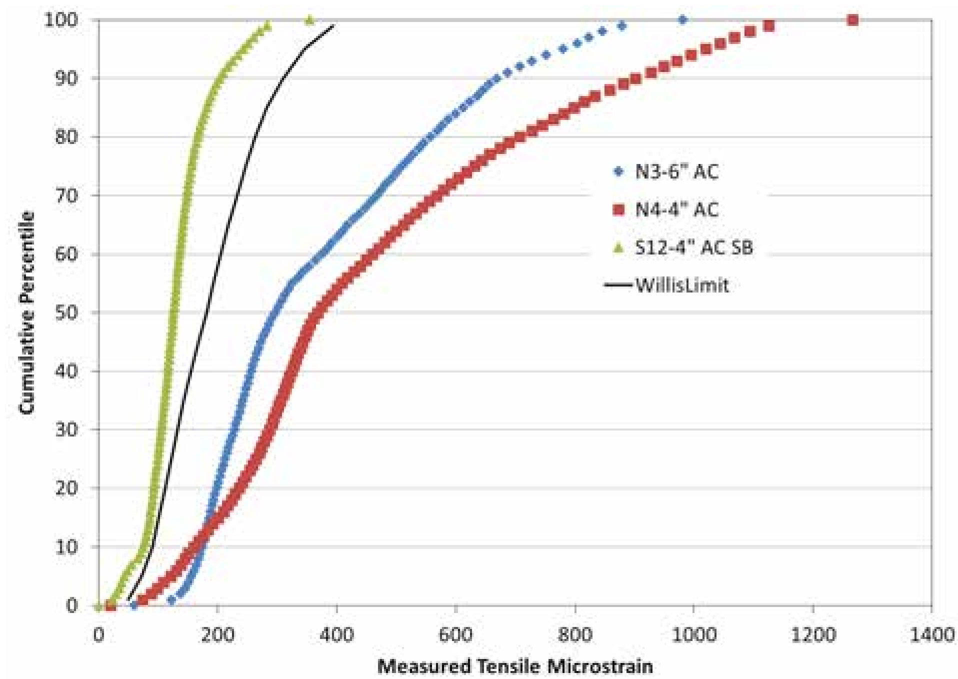
The Test Track sections were evaluated with respect to recently developed criteria that utilize control strain distributions in the evaluation process. The first, a measured strain distribution, was developed from Test Track data using strain data from a number of sections that either did or did not experience bottom-up fatigue cracking (*12,13*). This control distribution serves as an upper limit to strain response above which cracking may be expected and below which perpetual performance is expected. The second, a simulated strain distribution, was also developed from NCAT Test Track data but relied on simulated strain levels in the design software, PerRoad, and was further validated with field-documented perpetual pavements (*5,6,12*). It is important to emphasize that these criteria were not developed from CCPR or cement stabilized base pavement sections, so it is currently unclear whether they are truly applicable. However, they do serve as a checkpoint against conventional flexible pavements and further trafficking and monitoring of the sections will help further validate whether they may be applied, or new criteria are needed.

To evaluate the sections against the measured strain distribution criteria, a cumulative strain distribution was generated from measured data for each section and plotted with the control distribution as shown in Figure 8a. Simulated strain distributions were generated through the PerRoad software by entering relevant input parameters (i.e., layer moduli, layer thicknesses, traffic conditions) and utilizing the Monte Carlo features in PerRoad to produce cumulative distributions as illustrated in Figure 8b. Both analyses result in the same conclusion that Section S12, with the cement stabilized base layer, is expected to be perpetual since its strain distribution is less than the control distribution. The other two sections both exceed the control distribution, and according to the criteria, are expected to experience bottom-up cracking at some point. Furthermore, both analyses show the benefit of the additional 2 inches of AC resulting in lower strain levels for N3 relative to N4. At this point, it is unknown whether the sections exceeding the strain limits will truly develop bottom-up cracking since the criteria were not developed from CCPR sections. Likewise, cracking could develop in S12, perhaps from cracking of the cement stabilized layer reflecting through the CCPR and AC layers. However, application of the current perpetual criteria indicate that S12 is likely perpetual (i.e., no deep structural rutting and no bottom-up cracking) while the others are not.

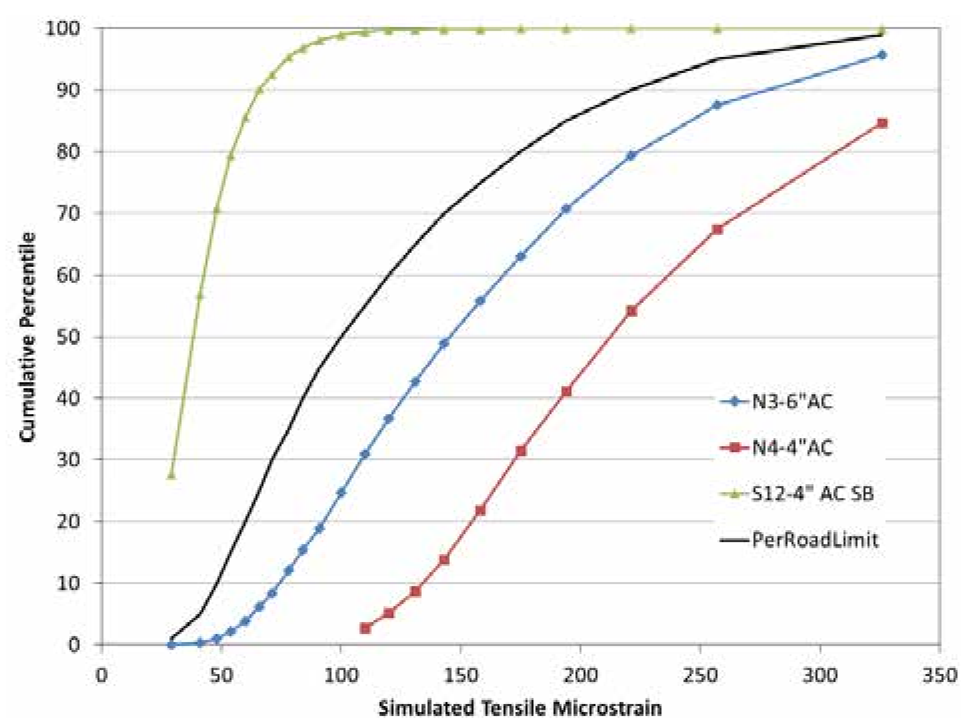
Further perpetual analyses were conducted with PerRoad to evaluate Sections N3 and S12 to quantify the changes in thickness needed to bring them closer to the perpetual design limit (i.e., optimized perpetual design). In Section N3, this required additional thickness while S12 required thickness reductions. Figure 9 summarizes the results of the analyses where the as-built cross sections were first analyzed followed by incrementally increasing (N3) or decreasing (S12) AC thickness. For example, the N3+2” series represents the simulation where an additional 2 inches were added to the as-built AC/CCPR layer thickness. The figure shows that an additional 2.5 to 3 inches of AC/CCPR, bringing the total AC/CCPR depth to 11.75 to 12.25 inches, will move N3 to the non-cracking side of the perpetual limit.

Figure 9 also shows that dramatic decreases in AC/CCPR thickness (up to 6 inches) still leave the strain distribution well to the left of the perpetual limit. An obvious concern with this cross section, which focuses only on controlling strain at the bottom of the AC/CCPR layer, is that covering the stabilized base layer with only 3 or less inches of AC/CCPR would potentially lead to cracking of the stabilized base layer. Additional mechanistic simulations were conducted with the layered elastic program, WESLEA for Windows, to examine the horizontal strain levels through the depth of varying S12 cross sections. These simulations, discussed in depth below, used the average modulus values from the PerRoad analysis and a single wheel load of 9,000 lb with 100 psi contact pressure.

Figure 10 summarizes the results for the three simulated S12 cross sections. In each case, the neutral axis (point of zero strain) of the cross section lies within the AC/CCPR layer with the bottom of the CCPR and stabilized base layers both in tension. The as-built cross section has nearly equivalent strain levels at the bottom of both layers, but the stabilized base layer experiences significantly higher strain levels as the AC/CCPR thickness is reduced. Changing from as-built to -6 in. results in a 40% increase in tensile strain at the bottom of the CCPR layer, but the stabilized base tensile strain increases by nearly a factor of 3. The strain (or stress) tolerance of the stabilized base layer is not currently known, but this analysis highlights the potential risk of placing too little AC/CCPR over the stabilized base layer where it would be forced to carry significantly greater tensile loadings. Measurement of the fatigue tolerance of the stabilized base should be studied in the future.



(a) Measured Strains



(b) PerRoad Simulated Strain

Figure 8 Perpetual Pavement Analysis Cumulative Strain Distributions (*4*)

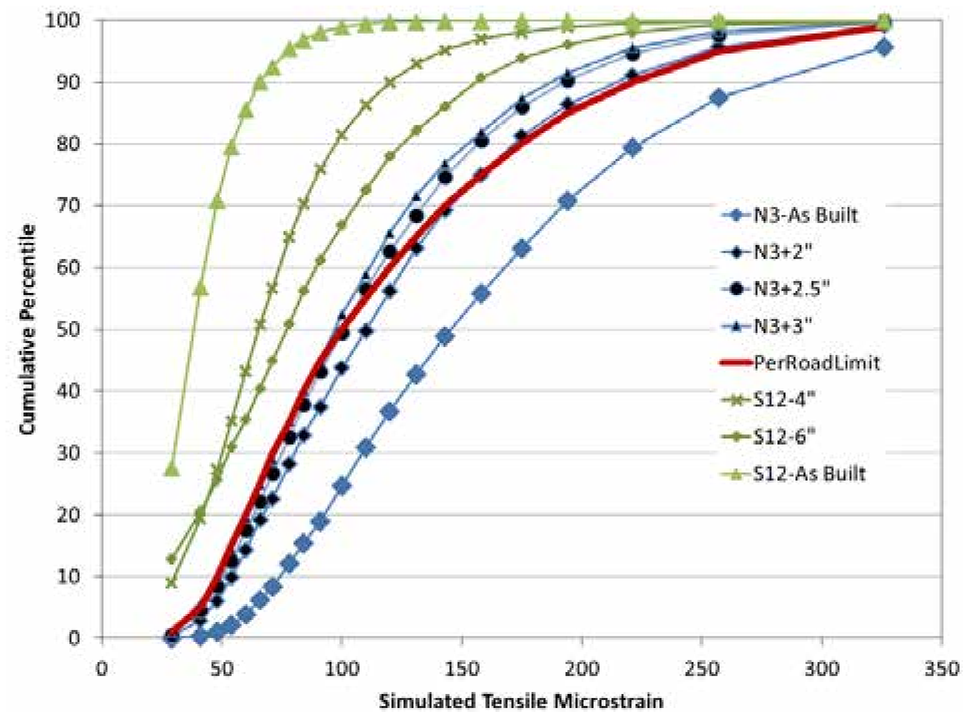


Figure 9 Additional Perpetual Strain Analyses (*4*)

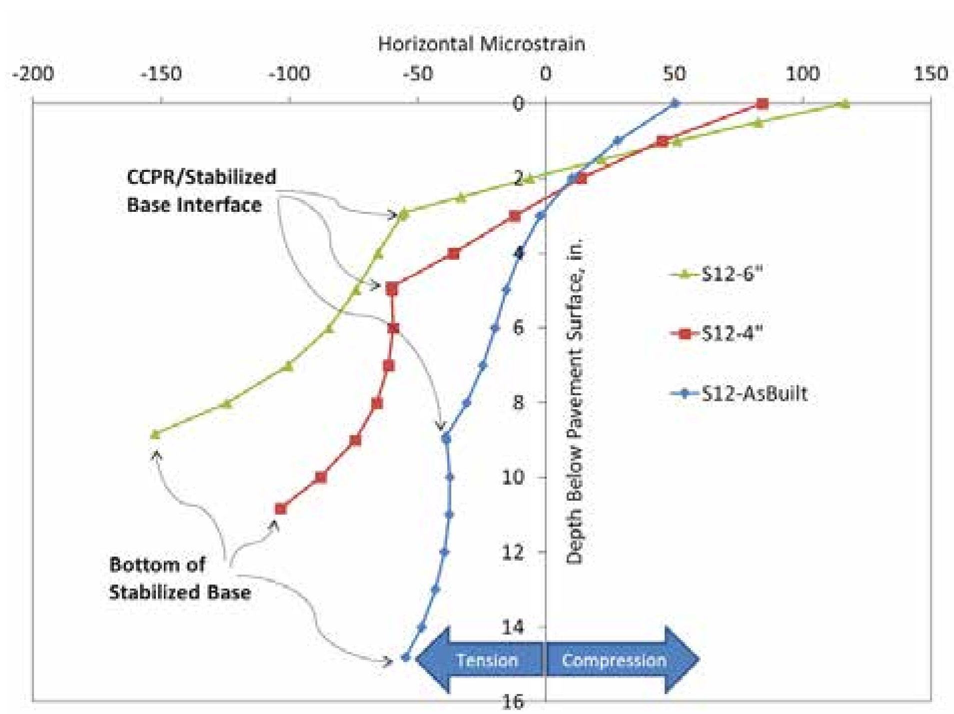


Figure 10 S12 Horizontal Strain Versus Depth Simulations (*4*)

12.7 Summary and Conclusions

This study was meant to investigate the field performance and structural characteristics of three CCPR sections at the NCAT Test Track under accelerated traffic loadings. Based upon the data presented above, the following conclusions and recommendations are made:

- The three recycled pavement test sections at the NCAT Test Track are examples of new or reconstructed pavement structures built with high percentages of recycled materials. The results of this study show that they have outperformed their designed service lives based on the current traffic level of 20 million ESALs and likely much longer based on performance and structural characterization.
- All three CCPR sections have exhibited excellent performance over the two research cycles. Very little difference was observed between sections in terms of rutting performance. Though S12 (4 in. AC, SB) was originally built rougher than the other sections, all three had very little change in smoothness over time. Cracking has not yet been observed in any of the sections. There were no distinguishable surface-observable performance differences between the three pavements.
- The backcalculated AC/CCPR moduli in N3 (6 in. AC) and N4 (4 in. AC) respond to changes in temperature similar to conventional asphalt materials, which was also observed in a laboratory study of CCPR mixtures (*8*). Future mechanistic modeling should treat CCPR with similar production characteristics as a bituminous material.
- The backcalculated AC/CCPR moduli in S12 (4 in. AC, SB) demonstrated much less temperature sensitivity and higher moduli than the other sections. It was believed to have resulted from the backcalculation process attributing some stabilized base properties to the AC/CCPR layer.
- Very little change in temperature-normalized modulus over time was found for N3 (4 in. AC) and N4 (6 in. AC). This indicates that the sections, in terms of modulus, do not appear to be curing or experiencing damage. Conversely, S12 (4 in. AC, SB) showed an increase in temperature-normalized modulus over time, which was again thought to be related to the stabilized base layer curing over time. Future investigations should focus on laboratory evaluation of the cement stabilized material to determine its curing characteristics.
- Tensile strain was measured at the bottom of the CCPR in all three sections. These data further supported treating the CCPR as a bituminous material for mechanistic modeling and design purposes. Further monitoring of the sections for cracking is needed to evaluate where and when fatigue cracking develops. Investigation is also needed to develop fatigue transfer function coefficients to predict cracking of CCPR.
- Based on perpetual strain analysis, Section S12 with the cement stabilized base layer is expected to be perpetual since its strain distribution is less than the control distribution. This assumes that the previously developed criteria may be applied to CCPR pavements with a cement-stabilized layer. Section S12 should be left in place for the next cycle of trafficking to validate this assumption and expectation.
- Based on perpetual strain analysis, Sections N3 and N4 exceed the control strain distribution and are expected to experience bottom-up cracking at some point. Again, it

is currently unknown whether the criteria apply to CCPR pavements and further trafficking and monitoring is warranted in the next test cycle.

12.8 References

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CHAPTER 13 EXECUTIVE SUMMARY

13.1 Overview

Located on a 309-acre site, the 1.7-mile NCAT Test Track is a world-renowned accelerated pavement testing facility that combines full-scale pavement construction with live heavy truck trafficking for rapid testing and analysis of asphalt pavements. Since its original construction in 2000, six cycles of research have been conducted, and findings from the Test Track have helped sponsors refine their materials specifications, construction practices, and pavement design procedures for asphalt pavements.

In 2015, the sixth Test Track cycle began a new chapter in full-scale pavement research through a partnership with the Minnesota Department of Transportation's MnROAD facility. One of the major NCAT and MnROAD partnership efforts is the ongoing Cracking Group (CG) Experiment with test sections built in both facilities to validate asphalt mixture cracking tests for future routine use in mix design and acceptance testing. The sixth research cycle also included individual experiments addressing research needs specific to the respective sponsors. The research topic and objective of each experiment conducted in the sixth research cycle are included in Table 1 followed by a summary of research and key findings for each experiment.

Table 1 Research Topics, Objectives and Sponsors of Sixth Cycle Experiments

Research Topics	Sponsors	Objectives
Cracking tests	FHWA, *ADEM, and *DOTs for AL, FL, IL, MI, MN, NY, NC, OK, and WI	Validating asphalt mixture cracking tests for future routine use in mix design and acceptance testing
Porous friction course (PFC)	Alabama Department of Transportation (ALDOT)	Continued evaluation of PFC test sections built in 2012 to improve PFC specifications
Bio-based rejuvenator	Collaborative Aggregates	Evaluating a surface mix containing bio-based Delta S rejuvenator and 35% RAP
Enhanced friction surface	Federal Highway Administration (FHWA)	Evaluating friction performance of asphalt bound surfaces
Softer binder for high RAP mix	Florida Department of Transportation (FDOT)	Evaluating the viability of using a softer binder for mixtures with a higher RAP content
Stress absorbing interlayer	Georgia Department of Transportation (GDOT)	Comparing two treatments for mitigating reflective cracking
Longitudinal joints and durability	Kentucky Transportation Cabinet (KYTC)	Evaluating the longitudinal joints and durability for two mix designs
High RAP thin overlay	Mississippi Department of Transportation (MDOT)	Continued evaluation of a Thinlay surface containing 45% RAP
Friction and bond strength	Oklahoma Department of Transportation (ODOT)	Assessing friction and the effect of tack coat application rate on bond strength of a PFC surface
4.75 mm thin overlay	Tennessee Department of Transportation (TDOT)	Evaluating the performance of a 4.75 mm NMAS thin-lift surface
CCPR and FDR	Virginia Department of Transportation (VDOT)	Continued evaluation of three sections built in 2012 with cold central-plant recycling (CCPR) and full depth reclamation (FDR) bases.

*ADEM: Alabama Department of Environmental Management; DOTs: Departments of Transportation.

13.2 Cracking Group Experiment: Validation of Cracking Tests for Balanced Mix Design

There have been increasing concerns over the past few years that volumetric properties are not sufficient to ensure the long-term durability of asphalt mixtures, especially those with higher contents of reclaimed asphalt pavement (RAP) and/or recycled asphalt shingles (RAS). As a result, laboratory tests have been developed to evaluate a mixture's resistance to various modes of cracking. However, there are very limited field validation data available to help DOTs select asphalt mixture tests that can address the most common types of cracking seen in their mixtures.

Recognizing this need, the CG Experiment was developed and led by the NCAT and MnROAD partnership to validate and assist state DOTs in implementing asphalt mixture cracking tests for future routine use in mix design and acceptance testing. The experiment includes (1) seven new test sections built on the Test Track to validate tests for top-down cracking and (2) eight rebuilt test sections on MnROAD's main-line test road for validating tests for low-temperature cracking. Preliminary results from the Test Track experiment are presented in this report with results from the MnROAD experiment to be reported in the future.

The seven CG sections on the Test Track are evaluated under the same traffic and environmental conditions and have similar pavement structures except for the surface mixes, which were designed with a range of recycled materials contents, binder types and grades, and in-place densities to achieve various levels of cracking performance. They were constructed as 1.5-inch surface lifts over highly polymer-modified intermediate and base layers. The target thickness for both the intermediate and base layers was 2.25 inches per layer. The asphalt pavement cross-section was relatively thin for the heavy loading on the Test Track so that the surface layers would experience significant stress and strains but avoid bottom-up fatigue cracking by using the highly modified mix for intermediate and base layers. The construction of these sections was completed in the summer of 2015.

After two years of trafficking with ten million accumulated ESALs, only one of the seven test sections (i.e., Section N8) had a substantial amount of top-down cracking in nearly seventeen percent of the lane area. The surface mix of this section has 20% RAP and 5% RAS with a PG 67-22 binder. Limited coring showed that the cracking was confined to the surface layer with no evidence of debonding between layers. Analyses of backcalculated asphalt mixture moduli also indicated that the wheel path cracking in this section has resulted in damage to the pavement structure. Three other test sections also showed evidence of very low-severity cracking on their surfaces. Based on the cores extracted at the cracked locations in these sections, there was no visible evidence that the observed hairline surface cracking in these sections had propagated into the surface layer. Analyses of backcalculated moduli indicated no damage to these pavement structures.

The plant mix for the seven surface layers was sampled during construction for laboratory testing. Testing of plant mix samples that were reheated just enough to fabricate the specimens has been completed and analyzed in this report. Additional work is underway to test plant mix samples that have been laboratory-aged to simulate approximately four years of field aging in

Auburn/Opelika, Alabama as well as testing of laboratory-prepared mixtures that have been aged to represent mix production aging and four years of in-service aging of surface mixtures. Based on the completed laboratory test results, the preliminary observations of the cracking tests evaluated are as follows.

The Energy Ratio (ER) method has several significant shortcomings. In its current procedure, it is not possible to properly characterize the variability of the ER parameter. The equipment cost and test complexity also render it impractical for routine use. The test results do not appear to properly separate the surface mixture with a substantial amount of cracking in Section N8 from the other surface mixtures that have had no signs of cracking. Although the field results are limited, and results of aged mixtures are yet to be reported, the Energy Ratio method does not seem suitable for specification use in routine practice.

The overlay test (OT) results (both the Texas method and the NCAT-modified method) ranked the mixtures largely in accordance with their anticipated level of field cracking. Results of the two test methods were highly correlated. Both methods predicted that the surface mixture in Section N8 would be the most susceptible to cracking, as was confirmed in the field. However, one of the disadvantages of the OT methods is their relatively high variability. For the results of this study, the pooled coefficient of variation for the Texas method was approximately 45%, and for the NCAT-modified test, it was approximately 35%. These are similar to results reported in the literature for these methods. This diminishes the power of the test to distinguish mixtures with significant differences in composition. Furthermore, higher equipment costs and longer time to complete the tests are substantial disadvantages. On the other hand, both OT methods appear to appropriately rank the mixtures with different density levels. The mixture with a higher density level had higher cycles to failure than the control mixture with a lower density level.

The semi-circular bend (SCB) and Jc criteria (Louisiana method) were able to identify the surface mixture in Section N8 as susceptible to cracking, but very similar results were also indicated for four of the other mixtures, two of which have shown no signs of cracking to date. More field performance data are needed to judge the validity of the Jc criteria. Despite the relatively large number of specimens needed to obtain the Jc parameter, the test can be completed within a few days. Like the ER parameter, a disadvantage of the SCB method is the inability to assess variability of the Jc parameter with traditional statistical analyses.

The Illinois Flexibility Index Test (I-FIT) yielded a relatively large spread of Flexibility Index (FI) results for the seven mixtures. This kind of statistical spread in results for different mixtures would allow users to better assess how to improve mix designs and adjust field mixtures. The FI results indicated that the surface mixture in Section N8 was the most susceptible to cracking, as was confirmed on the Test Track. Based on a similar calculation method, the indirect tensile asphalt cracking test (IDEAL-CT) data showed the same trends as the I-FIT data in most respects. More field performance data are needed to better judge the validity of the test and potentially set criteria for specification use. One concern with both the I-FIT and IDEAL-CT methods is the impact of specimen density. Counter to the expected outcome, higher density specimens have lower FI and CT_{Index} results than lower density specimens. The results of the I-FIT, IDEAL-CT, and the two OT methods were highly correlated for the mixtures in the CG experiments as they had similar aggregate gradations. The I-FIT and IDEAL-CT have the lowest equipment cost and fastest testing time of the six cracking tests in the experiment, but the IDEAL-CT offers faster specimen fabrication than the I-FIT since no specimen saw cutting is required.

Since a minimum amount of cracking was monitored in the seven sections, they will remain in place for continuing traffic and performance monitoring for another research cycle to increase the amount and severity of cracking in several test sections in order to accomplish the experiment objectives.

13.3 Alabama Evaluation of Open-Graded Friction Course Mixtures

Open-graded friction course (OGFC), also known as Porous Friction Course (PFC), has been used as the wearing surface in Alabama for many years. However, ALDOT has limited its use due to premature raveling issues occurring after approximately six or seven years in service (or between 10 to 20 million equivalent single axle loads). A typical OGFC mix in Alabama consists of a 12.5-mm nominal maximum aggregate size (NMAS), 0.3 percent cellulose fiber, and 6 percent PG 76-22 asphalt modified with styrene-butadiene-styrene (SBS).

In 2012, ALDOT sponsored three test sections (E9A, E9B, and E10) on the Test Track to evaluate potential changes in its mix design procedure to improve the durability of OGFC mixtures in Alabama. The following changes for an OGFC mixture were evaluated.

1. A finer 9.5-mm NMAS gradation (instead of a typical 12.5-mm NMAS gradation) was designed with a cellulose fiber and SBS-modified asphalt binder for the OGFC mixture in Section E9A.
2. A synthetic fiber (instead of a cellulose fiber) was utilized in the OGFC mix design for Section E9B with a typical 12.5-mm NMAS gradation and SBS-modified asphalt binder.
3. A ground tire rubber (GTR) modified binder was used in place of SBS-modified binder for the OGFC mixture in Section E10 with a typical 12.5-mm NMAS gradation but without cellulose fiber.

Prior to the construction of the three test sections at the Test Track, the OGFC mixtures were designed in 2012 based on a 12.5-mm OGFC mix design previously approved by ALDOT. These mixtures were designed with a design compaction effort of 50 gyrations to have minimum air voids of 15 percent, a maximum Cantabro loss of 15 percent, and a minimum conditioned splitting tensile strength of 50 psi.

Sections E9A, E9B, and E10 were milled and inlaid with the OGFC mixtures in 2012. All the mixes were placed 0.75 inches thick with in-place air voids immediately after construction at approximately 20 percent. Except for the changes made in the mix designs, the sections were paved following common construction practices for OGFC mixtures in Alabama. While a state-approved OGFC mix design was referenced when designing the three mixtures, it was not paved on the Test Track for this experiment, as previous in-service pavements on a nearby portion of Interstate 85 were considered the control for this experiment.

The 9.5-mm mixture in Section E9A experienced an increase in roughness toward the end of the 2012 research cycle, but this increased roughness level stayed the same throughout the 2015 cycle. The roughness of the 12.5-mm mixes was consistent throughout the two research cycles. The three mixtures performed well without cracking and experienced minimum field rutting of approximately 0.05 inches after 20 million ESALs from 2012 through 2017. There was no sign of raveling nor significant difference in the performance between the three OGFC mixes on the Test Track. Since the three OGFC test sections still performed well, ALDOT has decided to continue trafficking these test sections for another research cycle until 2021.

13.4 Collaborative Aggregates Delta S Rejuvenator Study

It is a common practice among asphalt paving producers to use RAP as a component in new asphalt mixtures to help lower production costs, conserve natural resources, and/or save landfill and stockpiling areas. While the use of RAP offers economic, environmental, and social benefits, there are concerns that using higher proportions of RAP in asphalt mixtures could result in stiffer mixtures that are likely prone to cracking and would result in higher maintenance and rehabilitation costs. To address these concerns, several methods have been proposed to reduce the potential effects of oxidized binders from RAP on the field performance of asphalt mixtures. One method is to use petroleum-based or bio-based rejuvenators to restore rheological properties of oxidized asphalt binders in recycled mixtures.

In 2015, Delta S, a bio-based rejuvenator, was used in an asphalt mixture with recycled materials placed in the surface layer of Section N7 for field performance evaluation on the Test Track. The field performance of Section N7 is compared with that of Section N1, which is the control section for the CG Experiment.

The surface layer of Section N7 was originally built in July 2015 using a 9.5-mm NMAS mixture with 20% RAP and 5% RAS. The Delta S rejuvenator was added to the virgin PG 67-22 binder at a dosage of 10% by weight of the recycled binders available in the RAP and RAS materials. After approximately four months of truck trafficking (1.8 million ESALs), the original surface showed slippage cracks.

A forensic investigation determined that the original surface mixture was produced and hauled to the Test Track (approximately 10 minutes away) for immediate paving without any silo storage. Because of the short haul and no silo storage time, the interaction between Delta S (blended with the virgin binder) and the recycled binder, especially in the RAS, may not have been completed, leaving a higher proportion of Delta S in the virgin binder than originally intended. This caused a decrease in stiffness and splitting tensile strength, leading to slippage cracking problems in the original surface mixture.

After the forensic investigation, it was decided that the interaction between Delta S and the aged binder in the RAS should be further studied and that the wearing course of Section N7 would be replaced with a mixture containing only RAP materials. This mix would have 35% RAP with a recycled binder ratio similar to that of the original surface mixture in Section N7. Because this mix design did not include RAS (even though it had a similar recycled binder ratio), the Delta S dosage was reduced to 5% by weight of the aged RAP binder, which was 5% lower than the dosage used in the original N7 surface mixture with 20% RAP and 5% RAS. The N7 surface mixture was redesigned to compare directly with the N1 surface mix with 20% RAP, which is the control mix for the Cracking Group experiment. Finally, it was determined that the mixture would be kept in a silo for two hours before paving so that the rejuvenator could interact with the RAP binder.

The corrective actions taken for the new surface mixture in Section N7 helped address the problems identified in the original surface mixture. To the end of the 2015 research cycle, the re-designed surface mixture with 35% RAP and 5% Delta S in Section N7 endured around 7.5 million ESALs (compared to 1.8 million ESALs for the original surface mixture), and no slippage failures were observed. In addition, the re-designed surface mixture showed good ride quality and rutting performance. The cracking performance of the re-designed surface mixture in Section N7 was comparable to that of the surface mixture with 20% RAP in Section N1, the control section for the CG Experiment, at the end of the sixth research cycle. Both sections (N1 and N7) will be kept in place for continuing traffic through another research cycle to allow for a thorough field performance evaluation.

13.5 FHWA Development of Asphalt Bound Surfaces with Enhanced Friction Properties

A high friction surface treatment (HFST) can enhance pavement friction for safe driving in critical braking and cornering locations such as horizontal curves, deceleration ramps, and intersection approaches. Currently, the standard HFST specified in AASHTO PP 79-14, *Standard Practice for High Friction Surface Treatment for Asphalt and Concrete Pavements*, is often a thin thermosetting polymer resin bound layer of calcined bauxite aggregate.

The standard HFST has shown the highest friction and high macro-texture characteristics for skid resistance. However, it requires polymer resin binder and imported aggregate materials and can only be placed by a limited number of contractors with specialized equipment, making it much more expensive than other commonly used pavement materials.

A previous FHWA-sponsored study conducted at the Test Track used regionally available friction aggregates in place of the calcined bauxite. The results concluded that polymer resin bound surfaces with other regionally available friction aggregate sources did not provide the same level of surface friction as those of the calcined bauxite. Based on these results, another study was conducted in the sixth research cycle to evaluate asphalt (instead of polymer resin) bound surfaces, specifically micro-surfacing and thin overlays, with calcined bauxite as the primary friction aggregate. These surfaces were placed using conventional asphalt construction technologies instead of the specialized application equipment required to place the standard HFST. The study included two sections (W7A and W7B) placed in 2015, one section (W3) placed in 2017, and three HFST sections (W8A, W8B and W9) placed in 2011, as follows:

- Section W7A was resurfaced with a micro-surfacing layer using (1) a 50:50 aggregate blend of calcined bauxite and limestone sand and (2) a CSS-1HP emulsion processed from a highly polymer modified asphalt (HiMA) to improve surface durability and aggregate particle retention.
- Section W7B was also resurfaced with a micro-surfacing layer, but the mixture consisted of a 100% sandstone blend and the same emulsion utilized in the micro-surfacing layer of Section W7A.
- Section W3 was resurfaced with a Stone Matrix Asphalt (SMA) layer. The SMA was designed as a 4.75 mm NMAS mixture to expose the calcined bauxite particles on the surface. The aggregate blend, including 40% calcined bauxite, 59% granite, and 1% filler, was used with a PG 76-22 binder in the mix.
- Three HFST sections (W8A, W8B and W9) remaining in place from the previous study were used for comparison. They were constructed using polymer resin bound surfaces with different aggregates. W8A used granite aggregate, W8B used calcined bauxite aggregate, and W9 used flint aggregate.
- Among these sections, W8B is only one complying with the AASHTO PP 79-14 HFST standard, as it utilizes the calcined bauxite meeting the minimum 87% aluminum-oxide specification requirement, which is considered the “gold standard” for high friction. The calcined bauxite used in Sections W7A and W3 has only 84% aluminum-oxide.
- All test sections were located in the super-elevated portion of the west curve. Also, since the sections were placed at different times, they received different levels of cumulative traffic by the end of the sixth research cycle.

Both micro-surfacing sections performed well based on the measured friction and macro-texture after receiving 10 million ESALs for two years of traffic. The friction (skid number at 40 mph or SN40R) and macro-texture (mean profile depth or MPD) measurements are 55 and 0.70 mm for W7A and 50 and 0.90 mm for W7B. However, these measurements are lower than those of the standard HFST (W8B), which has been tested for five years with over 23 million ESALs of traffic polishing.

The SMA section (W3) performed well based on measured friction (SN40R = 55), but its surface macro-texture value (MPD = 0.35 mm) was low, which increases risk for hydroplaning. This section was only eight months old and received 3.4 million ESALs at the end of the sixth research cycle.

Furthermore, since the safety of the pavement surface is influenced by both micro-texture (friction) and macro-texture (surface texture), more research is needed to study the combined impact of both surface features on reduction of crash rates. For example, is there a measurable reduction in crash rate when the micro-texture is greater than 50 SN40R and the macro-texture is greater than 0.60 mm MPD?

13.6 Florida High RAP and Cracking Study

The FC-9.5 and FC-12.5 Superpave mixtures have been successfully used by FDOT for surface layers. These mixtures are designed with a PG 76-22 binder, and the maximum RAP content allowed is 20% by weight of total aggregate. In some cases, contractors can use more than 20% RAP by weight of aggregate, provided that there is no more than 20% binder replacement.

Contractors in the state are currently not allowed to use more RAP in these mixes due to concerns that the additional amount of RAP will make these mixtures susceptible to cracking. To address these concerns, FDOT has considered specifying a softer modified binder and/or requiring additional performance testing during mix design to evaluate a mixture's cracking resistance if more than 20% RAP is allowed.

To evaluate their options under consideration, FDOT sponsored four subsections that were resurfaced with approximately two inches of four FC-12.5 Superpave mixtures. The four surface mixtures were designed with a design compaction effort (N_{des}) of 100 gyrations. The main differences between these mixtures are the amount of RAP used and base binder's performance grade (PG 64-28 and PG 58-28). The quality control data showed 20.0% RAP for E7A mix, 23.9% RAP for E7B, 28.9% RAP for E8A, and the same PG 76-22 polymer-modified binder was used in the three mixtures. The E8B surface mix had 28.8% RAP, similar to the E8A mix, but had a softer polymer-modified PG 64-28 binder.

The four subsections were trafficked for two years to evaluate their field performance. In addition, laboratory testing was conducted on plant mix and asphalt binder to determine if additional testing can be specified during mix design to evaluate the rutting and cracking performance FC-12.5 Superpave mixtures with more than 20% RAP.

The four mixtures performed well after approximately 10 million ESALs with low severity cracking. The roughness (based on International Roughness Index, IRI) remained unchanged throughout the research cycle. The four sections showed almost no rutting after 10 million ESALs with rut depths being below 2 mm. Some changes were observed in macro texture data (based on mean texture depth, MTD) after 5 million ESALs. These changes may be related to the removal of asphalt film on the surface due to trafficking.

All of the mixtures exhibited only low severity cracks, and they were found to be reflective cracking based the crack maps surveyed before placing these mixtures. The area of lane with cracks did not change much after 5 million ESALs but had an increase in severity close to the point of being classified as medium severity. The final area of lane with (all) low severity cracks was approximately 13% for E7A, 11% for E7B, 7% for E8B, and 4% for E8A. In addition, results of several laboratory cracking tests (i.e., SCB, I-FIT, ER, and OT) conducted on plant mix were compared with the field performance, but they did not show good correlations.

The performance data observed after two years of trafficking at the Test Track did not show the effect of increasing the RAP content from 20% to 30% nor using a softer polymer-modified asphalt binder on the mixture field performance. These sections will remain in place for another research cycle to further evaluate their field performance and correlations between lab and field cracking performance.

13.7 Georgia Interlayer Study for Reflective Crack Prevention

To reduce reflective cracking, GDOT has used a cracking relief interlayer to provide discontinuity between the existing surface and overlay so that cracks are not as easily reflected to the overlay. Currently, the interlayer is composed of a single chip seal treatment over the existing surface. It is then leveled with 75 to 80 lbs/sy of asphalt mixture before placing an overlay. This method, however, has not been as effective as desired. Therefore, GDOT has sponsored a study at the Test Track since 2012 to evaluate two alternative methods including a double chip seal treatment with a sand seal top layer (Section N12) and an open-graded interlayer (OGI) (Section N13).

To simulate cracking, deep saw cuts were made in both test sections and filled with sand to avoid self-healing. Section N12 was then covered with a cracking relief interlayer consisting of a double chip seal and a sand seal placed about 0.7 inches thick and surfaced with a 1.5-inch thick layer of a 9.5-mm NMAS dense-graded mix. Section N13 was covered with a 1.1-inch thick OGI mixture and a 1.1-inch thick overlay using the same mix as Section N12. The OGI was similar to a 12.5 mm NMAS PFC but with lower asphalt content and no fibers (the mix temperature was lowered to prevent drain-down). Both sections have the same combined thickness of about 2.2 inches above the existing saw-cuts surface.

At the end of the 2012 research cycle and after approximately 10 million ESALs, cracking was beginning to develop in both sections, so these sections were kept in place for another research cycle. After trafficking for two more years and approximately 20 million ESALs of loading, the amount of cracking in Section N13 (with the OGI interlayer) increased significantly with 50% of the saw cut area having reflected through to the surface. For Section N12, reflective cracking was observed for only 6% of the saw cut area. Cracking in both sections is still at low severity (≤ 6 mm).

There was rutting in both sections with Section N12 (with the surface treatment interlayer) having higher rut depths at the end of the 2015 research cycle. The maximum rut depth in Section N12 was approximately 0.75 inches (21 mm) while it was only 0.25 inches (6 mm) in Section N13.

13.8 Kentucky Longitudinal Joints and Mix Durability Experiment

The durability of longitudinal joints in asphalt pavements is one of the major concerns by KYTC. While poor compaction of the mix at a longitudinal joint is often considered the main cause leading to its deterioration, use of a coarse-graded asphalt mixture currently specified by KYTC can also make its compaction more challenging at the joint. For this reason, KYTC sponsored a study at the Test Track to determine if a finer mixture can be specified to improve the performance of longitudinal joints and overall mix durability without compromising rutting performance. Past research conducted at the Test Track showed that fine-graded mixtures perform as well as coarse-graded mixtures in terms of rutting performance.

Two subsections (S7A and S7B) were planned for this study. Both the inside and outside lanes of these subsections were milled and inlaid, and the outside lane was paved a few days after the inside lanes had been placed to simulate staged construction in Kentucky. Standard construction practices were followed to construct the longitudinal joints for these sections.

Both lanes of Section S7A were paved with a coarse-graded 9.5-mm NMAS surface mix. This mix was approved by KYTC with a N_{des} of 100 gyrations and an SBS-modified PG 76-22 binder. The two lanes of Section S7B were paved using a fine-graded mixture designed to meet KYTC's volumetric requirements with a lower N_{des} of 65 gyrations. The changes resulted in a higher binder content (0.6% higher) for the fine-graded mixture. The two mixtures used the same aggregates, NMAS, and asphalt binder. Laboratory testing was conducted to make sure the mixtures would not fail because of rutting.

After two years of trafficking with 10 million ESALs, no cracking was observed in the two sections. Rut depths for both sections were less than 3 mm, indicating similar rutting performance for the two mixtures. Roughness based on IRI was significantly higher for Section S7A than for Section S7B. The field permeability taken directly on the longitudinal joint of S7B (fine-graded) was 20% lower than that measured on the longitudinal joint of Section 7A (coarse-graded), which could affect the joint performance.

Based on the results of this study, it was recommended that KYTC considers allowing fine-graded mixtures with a lower design compaction effort. The sections are being left in place for another research cycle to continue assessing their performance.

13.9 Mississippi Evaluation of Thinlay Mix with 25% RAP and Local Aggregates

To address the need to maintain more lane miles with limited budgets, state agencies have considered various options for keeping their roadways in good condition. Thin asphalt overlays or Thinlays, which can be paved as thin as 5/8 of an inch, are one of these options.

Thinlays have been evaluated on the Test Track for more than 15 years. In 2003, a low volume road 4.75-mm NMAS mix was placed ¾-inch thick in Section W6 for a study sponsored by MDOT. The mix was expected to last for a half-million ESALs; however, this section has supported over 50 million ESALs to date with no cracking, rutting, or raveling. This mix consisted of 69% imported limestone screenings, 30% hard sand, 1% hydrated lime antistrip agent, 6.1% polymer modified liquid asphalt, and no reclaimed or recycled materials.

In 2012, MDOT redesigned this Thinlay mix by adding RAP, changing from polymer modified to neat asphalt, eliminating imported stone screenings, and relying completely on locally available surplus sand stockpiles in Mississippi. The result of this effort was a new Thinlay mix placed on the surface layer of Section S3 in 2012. The redesigned Thinlay surface consisted of hard sand locally available in Mississippi, RAP, Portland cement filler, hydrated lime antistrip, and neat virgin liquid asphalt. The target lift thickness on the track was 1 inch, but this mix could be used for preservation in lifts as thin as ¾ inch. Both W6 and S3 Thinlay mixtures were placed on the originally constructed Test Track sections with approximately 2 feet of asphalt structure.

The objective of Section S3 for the 2012 research cycle was to evaluate the rutting performance of the redesigned Thinlay mix. After the application of 10 million ESALs at the end of the 2012 cycle, no significant rutting or surface cracking was observed. MDOT chose to continue trafficking through the 2015 research cycle in order to expand the scope of the mix evaluation to include cracking and durability.

At the end of the 2015 research cycle and approximately 20 million ESALs of heavy truck traffic, no cracking, rutting, roughness, raveling, or friction deficiencies were noted for the redesigned Thinlay mix. A low cost per mile can be achieved as a result of the use of all local materials, RAP, and neat liquid asphalt in a relatively thin surface layer (i.e., at a low spread rate).

13.10 Oklahoma Open-Graded Friction Course and Surface Friction Experiment

A good friction surface is needed in critical braking and cornering locations for safe driving. The current standard HFST requires premium thermosetting polymer resin and imported calcined bauxite aggregate, making it an expensive surface treatment. Therefore, state agencies are interested in finding an alternative.

In 2015, ODOT sponsored a study at the Test Track to determine the highest surface friction an asphalt surface mixture can achieve using aggregates available in Oklahoma. The surface mixture selected was OGFC as it had the best macro-texture to improve surface friction. In addition, since the performance of OGFC is significantly affected by the interface bond between the OGFC and the underlying surface, the second objective of the study was to determine the tack coat application that can improve the interface bond.

Based on a laboratory study conducted prior to construction, sandstone aggregate had the best friction characteristics among four regionally available aggregates evaluated and was selected for further evaluation on the Test Track. In addition, a higher tack application rate yielded a higher interface shear bond strength. Based on these results, a 12.5-mm NMAS OGFC mixture was designed with 20.1% air voids using 6.4% SBS-modified PG 76-22 binder and sandstone aggregate. The OGFC mix was tacked with a hot-applied tack coat (Ultrafuse) at two application rates for evaluation.

During construction, the existing surface of Section N9 was first micro-milled. It was then tacked with UltraFuse at a residual rate of 0.05 gal/yd^2 for the first 100 feet and at 0.10 gal/yd^2 for the second 100 feet. Finally, the sandstone OGFC was placed 0.75 inches thick on the tacked surface

Friction performance measured periodically by a ribbed-tire skid trailer and a dynamic friction tester showed a slight decline surface friction over the two-year period. The highest SN40R values of 57 were measured in April through June 2016, and the final SN40R values of 53 were taken in the last three months of truck traffic. The measured friction values were higher than the typical SN40R of 45 to 35 for other dense-graded asphalt surfaces placed on the Test Track but lower than the SN40R for the standard HFST, which were above 65 at the end of the same cycle (but with more than four years of Test Track traffic).

Pavement surface texture was measured with a high-speed laser mounted to a survey van and with a circular texture meter. Both values indicated very good macro-texture, as expected from

OGFC surfaces in comparison to dense-grade mix surfaces. High-speed laser macro-texture values started at 1.2 mm MPD and dropped slightly to 1.1 mm over the two years of traffic.

Since the conventional shear test method for evaluating the interface bond strength would leave many core holes in the short test sections and the thickness of the OGFC lift (approximately 0.75 inches thick) would not be sufficient for shear testing across the interface, a pull-out test was adapted for use on this project. The test results, both visual and measured values, show that the higher tack coat rate created better tensile bond between the OGFC and the milled surface.

After approximately 10 million ESALs of heavy truck traffic at the end of the research cycle, no rutting was observed. Measured cracking remained constant at 2.4% and was limited to reflective cracking from the underlying pavement. In addition, the ride quality of the two sections did not change during the traffic period after accounting for sample core damage at the beginning of the section.

13.11 Tennessee Evaluation of 4.75-mm Mix for Thicker Lift

The 4.75-mm mix is routinely placed as thin-lift (5/8-inch) surfaces in Tennessee. With its satisfactory performance over the years, TDOT considers placing this mix in a thicker (i.e., 1.25-inch) surface lift to achieve better in-place density but is concerned with the mix's ability to resist rutting. For this reason, TDOT evaluated a 4.75-mm mix design placed in a thicker lift in Section S4 with the focus on its rutting resistance.

The mix was designed with 75 blows by TDOT according to the Marshall mix design method. The aggregate gradation consisted of limestone aggregates and natural sand from Tennessee and 16% fine (minus 5/16-inch) RAP from a stockpile at the Test Track. The optimum binder content for this mix was 6.8% by weight of the total mix and was a blend of 87% PG 64-22 new binder and 13% RAP binder.

During construction, Section S4 was milled and inlaid with the 4.75-mm mix. The as-built thickness for the surface layer was 1.50 inches. There were also noted differences between the mix design and production test results, which were most likely due to the differences between the aggregates sampled in Tennessee for mix design and those delivered for construction at the Test Track.

With over 10 million ESALs of heavy truck traffic, the 4.75-mm mixture showed good smoothness, no cracking, and low rutting (i.e., less than 1.2 mm). The field rutting performance agreed with the Hamburg test results with an average rut depth of 2.6 mm and no sign of stripping. The mixture also maintained a stable friction value during truck traffic polishing and showed increasing macrotexture as the asphalt mastic was wearing off the surface.

13.12 Cold Central Plant Recycling and Stabilized Base Experiment

Cold central plant recycling (CCPR)—a method of combining RAP with foamed or emulsified asphalt and additives in a central recycling plant without the application of heat—has been used for rehabilitating low- and medium-volume roadways. To determine the viability of this technology for high volume roadways, the VDOT sponsored three test sections, complementing an existing project on I-81 in Virginia, to evaluate field performance of CCPR material and characterize its structural contribution. Sections N3 and N4 were designed to evaluate the difference between 6 and 4 inches of asphalt built on top of the same underlying layers, including 5 inches of CCPR material and 6 inches of aggregate base. Sections N4 and S12 were designed to evaluate the difference between underlying base materials, 6 inches of aggregate base vs. 8 inches of cement-stabilized base (CSB), in supporting the same upper layers, including 4 inches of HMA and 5 inches of CCPR material.

After two research cycles and over 20 million ESALs, all three sections have performed well with no cracking, minimal rutting, and no appreciable change in ride quality. Structural evaluations showed that CCPR material responds to temperature changes like a conventional mix, which makes it appropriate to model CCPR material as a bituminous material in a mechanistic design. Compared to Sections N3 and N4, the backcalculated AC/CCPR moduli in Section S12 had less temperature sensitivity and higher moduli, likely due to the backcalculation process attributing some of the CSB properties to the AC/CCPR layer. Section S12 also showed an increase in temperature-normalized modulus over time, possibly due to the CSB curing.

Section N3, with an additional two inches of AC, had lower strain levels than Section N4, and the CSB in Section S12 yielded much lower strain magnitudes and less temperature sensitivity. Strains normalized to 68°F showed that Sections N3 and N4 had an increasing trend over time, while Section S12 was relatively constant. Thus, using a stabilized base may help control tensile strains and help eliminate bottom up fatigue cracking provided the stabilized base has been properly designed to mitigate cracking that could propagate through the asphalt in that layer.

Based on perpetual strain analysis, Section S12 with the CSB layer is expected to be perpetual as its strain distribution is less than the threshold distribution, while Sections N3 and N4 are expected to have bottom-up cracking in the future as its strain distribution exceeds the threshold distribution. Sections N4 and S12 are remaining in place for another research cycle to validate the assumption and criteria used in this analysis.