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Phase VIII (2021-2024) NCAT Test Track Findings: Implementation Synopses

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DISCLAIMER

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1. IMPLEMENTATION SYNOPSES

1.1 Introduction

The sponsorship of the eighth Test Track research cycle (2021 to 2024) was a partnership of fourteen highway agencies, the Federal Highway Administration (FHWA), and private companies. These sponsors participated in individual and group experiments to advance asphalt pavement technologies. The key findings of these experiments, supported by the field performance on the Test Track, are synthesized in this chapter for implementation. These findings will help highway agencies refine their specifications for materials, construction practices, and pavement design, ultimately leading to an improvement in asphalt pavement quality. The findings also enable the private sponsors to validate and promote their innovations.

1.2 Additive Group Experiment

The goal common to most asphalt additives is to enhance the overall performance and longevity of flexible pavements, often while utilizing sustainable, recycled materials. Several additive categories have gained popularity over the years; these technologies include post-consumer recycled (PCR) plastics, recycled ground tire rubber (GTR), and fibers. These additive technologies are marketed to state agencies and contractors with the promise of performance enhancements, cost reductions, or both. Full-scale testing of asphalt mixtures incorporating additives can be used to validate these claims. However, it is both cost-prohibitive and time-consuming and, therefore, not a practical method for evaluating prospective additive technologies rapidly. In many cases, once agencies have become comfortable enough with an additive technology to allow its widespread use, new technologies have already emerged and are awaiting evaluation and implementation.

Therefore, an industry-wide need for a more rapid system to accurately evaluate new, and existing, additive technologies was recognized. The Additive Group (AG) experiment aims to addresses this need by providing a comprehensive laboratory analysis and full-scale field evaluation of a range of additive technologies selected by the state DOT sponsors of the AG experiment. A “framework” for the rapid evaluation of additive technologies that utilizes the results of the battery of laboratory tests, calibrated to the fatigue damage findings from the full-scale paved test sections is one of the primary goals of the experiment. The intent of this framework is to provide state agencies with a method of predicting the in-field fatigue cracking performance of asphalt mixtures modified with additives, without the need for the construction of full-scale test sections; only requiring the battery of laboratory tests to be conducted.

Phase I of the AG experiment was limited entirely to the laboratory setting. It involved a preliminary investigation of multiple additive technologies, the development of the dense-graded mixture design using a balanced mix design (BMD) approach, and eventual selection of five additive technologies (plus one control mixture) for evaluation in Phase II of the experiment. The technologies selected for evaluation in Phase II included a wet and dry process recycled ground tire rubber (GTR), aramid fibers, and wet and dry process post-consumer recycled (PCR) plastic (and wet process SBS-modified control mixture).

Phase II of the additive group saw a comprehensive laboratory evaluation of the six AG mixtures. The stiffness and fatigue characteristics of the asphalt mixtures were evaluated utilizing the dynamic modulus (E*), direct tension cyclic fatigue, and bending beam fatigue tests. Simultaneously, full-scale instrumented test sections were constructed at the NCAT Test Track to evaluate the field fatigue cracking performance and general structure condition and response of each modified asphalt mixture. The initial cycle of accelerated trafficking (approximately 10 million ESALs) concluded in April 2024. Performance data (IRI, rutting, and cracking), backcalculated layer moduli (from FWD testing), and measured strain responses were collected at regular intervals throughout this period of accelerated trafficking.

Further analyses of the field data, coupled with forensic analyses of some of the AG test section(s) has commenced following the recent conclusion of trafficking. Preliminary development of the AG analysis "framework" will coincide with these analyses, though the majority of the AG test sections require additional accelerated trafficking in order to induce fatigue cracking distresses.

Based on the current results of the NCAT Additive Group Phase II experiment, the following findings and recommendations have been provided:

Principle Laboratory Findings—

- The dry process plastic-modified mixture had significantly increased stiffness relative to the control mixture, especially at intermediate and high temperatures. Both rubber-modified mixtures had decreased stiffness at low and intermediate temperatures relative to the control mixture. The fiber-modified and wet process plastic-modified mixtures were found to have similar stiffness to the SBS-modified control mixture.
- The wet process rubber-modified mixture displayed the best fatigue performance in terms of its fitted S_{app} parameter value and was found to have drastically increased strain tolerance over the other AG mixtures, based on the relative positioning of its fatigue life transfer function. The dry process rubber-modified mixture was found to have the second highest fitted S_{app} parameter value, but a statistically similar strain tolerance to the control mixture. The dry process plastic-modified mixture displayed the worst overall fatigue performance, both in terms of its fitted S_{app} parameter value and the relative positioning of its fatigue life transfer function. The fiber-modified and wet process plastic-modified mixtures were found to fatigue performance similar to the SBS-modified control mixture.

Principle Field Findings—

- None of the AG test sections displayed an increase in their average IRI through the initial period of accelerated trafficking, despite the detection and measurement of cracking in multiple sections.
- All of the AG test sections displayed similar increases in their average rut depth through the initial period of accelerated trafficking, which was well below the allowable 0.5" threshold.
- The dry process rubber-modified test section was found to have the greatest amount of fatigue cracking (16.5% of the wheelpath area), followed by the dry process plastic (0.6%) and wet process rubber-modified (0.3%) test sections (both of which had much less cracking by comparison).
- The dry process rubber-modified test section had the highest average measured strain response amongst all of the AG test sections, followed by the wet process rubber-modified test section. The high levels of strain, coupled with the dry process rubber-modified mixture's average strain tolerance (displayed by its laboratory fatigue transfer function), likely led to the cracking observed in the test section. By contrast, the wet process rubber-modified mixture was found to have excellent strain tolerance in laboratory testing. This likely helped it resist fatigue cracking distresses for longer. The dry process plastic-modified test section developed fatigue cracking despite it having the second lowest average strain response overall. The observed cracking could be explained by this mixture's poor strain tolerance (as evaluated and observed in laboratory testing) relative to the other AG mixtures.
- Granular base and subgrade moduli differ significantly between the AG test sections, adding a confounding variable to that must be properly accounted for when modeling the predicted fatigue performance of the test sections utilizing the laboratory test results.

1.3 Alabama Department of Transportation High-Performance Open-graded Friction Course (OGFC) Mixture Design

The Alabama Department of Transportation (ALDOT) has used open-graded friction course (OGFC) mixtures for many years due to their significant safety benefits. However, the premature raveling in the OGFC layer has been a recurrent issue in Alabama. The premature raveling of OGFC mixtures also seems to be more prevalent when using certain aggregate types, like sandstones and slags. In Alabama, the majority of OGFC mixtures typically have an optimum binder content (OBC) of 6.0% regardless of aggregate types with the current ALDOT design method (ALDOT-259-97), which is the minimum permissible binder content for OGFC mixtures per ALDOT Specifications. Therefore, a more rigorous performance-based OGFC design methodology is needed to improve durability by considering the mixture performance during the design phase. This adjusted design method uses the Superpave Gyratory Compactor (SGC) for laboratory compaction and determines the OBC based on mix properties that include air voids, Cantabro loss, permeability, and moisture susceptibility.

The high-performance OGFC mixture design procedure consists of three steps: 1) designing the aggregate blend to meet the gradation and voids in coarse aggregate (VCA) requirements; 2) determining the OBC based on air voids (using the vacuum sealing method per AASHTO T331) and Cantabro loss results; and 3) verifying the mixture draindown and moisture susceptibility (using Tensile Strength Ratio (TSR) test per AASHTO T 283) at the OBC. All the mixture tests were performed on SGC design pills compacted with 50 gyrations except for the draindown testing, and the loose mixtures were conditioned at 275°F for four hours prior to compaction. The proposed volumetric and performance criterion included an air voids range of 15-20%, a maximum Cantabro loss of 15%, a maximum draindown of 0.3%, and a minimum TSR and conditioned splitting tensile strength of 0.70 and 50 psi.

In 2021, the high-performance design methodology was used to develop two OGFC mixtures with two local aggregate types including granite and sandstone/slag. The OBC of 7.0% was determined for both designs, which was approximately 1.0% higher than the typical values determined based on the current ALDOT design method. To better determine the effectiveness of the high-performance design method, the OGFC mixture designed with the challenging sandstone and slag was produced and constructed on the section E9 of 2021 NCAT Test Track.

The QC test results of the plant-produced mixtures showed a slightly higher binder content (i.e., 7.2%) and a finer blend gradation compared to the original job mix formula (JMF) design, which resulted in lower air voids and Cantabro loss while higher TSR and conditioned splitting tensile strength. The resultant finer gradation during production was mainly caused by the high Los Angeles (LA) abrasion and segregation of the aggregate stockpiles. In addition, this section was paved following common construction practices for OGFC mixtures in Alabama except for the changes made to the mix designs. The as-built lift thickness was 1.3 inches, which was greater than the planned thickness of 1.0 inch.

After 10 million ESALs of trafficking from 2021 to 2024, the OGFC section exhibited excellent smoothness, rutting resistance, and cracking resistance. During this research cycle, all the field performance generally remained consistent except for permeability, which indicated excellent durability without the occurrence of raveling. The field permeability of the section reduced with increasing trafficking, and the OGFC section still has limited permeability (38m/day) after 10 million ESALs of trafficking. Based on the preliminary results, the high-performance OGFC design method is recommended to replace the current ALDOT design method to improve the durability of OGFC mixtures in the field.

1.4 Alabama Department of Transportation Long-Term Performance Evaluation of High-Performance Thinlays

In the 2018 Test Track research cycle, the Alabama DOT (ALDOT) sponsored Sections N10 and N11 to evaluate two thinlay mixes. This experiment assessed the field performance of thinner overlays to provide ALDOT with alternatives suitable for pavement preservation on high-volume roads. At the end of the 2018 research cycle, both sections showed excellent performance with no cracking and minimal rutting. Based on the performance of the test sections, trafficking of the sections continued in the 2021 Test Track research cycle to assess long-term performance.

Section N10 used a 4.75 mm NMAS Stone Matrix Asphalt (SMA) mix, and Section N11 used a 4.75 mm NMAS dense-graded Superpave mix. The SMA mix used a blend consisting of 62% limestone, 13% granite, 5% fly ash, and 20% fine fractionated RAP, and included a PG 76-22 SBS-modified binder with a total binder content of 6.0%. The dense-graded mix used a blend of 58% limestone, 22% sand, and 20% fine fractionated RAP, and used a PG 67-22 binder with a total binder content of 6.1%. ALDOT does not allow high percentages of carbonate aggregate, such as limestone, in surface mixes due to pavement friction concerns. ALDOT granted a waiver for this project to use higher percentages of limestone than currently allowed due to a desire to use locally available materials.

The as-built lift thicknesses of sections N10 and N11 were 0.8-inch and 0.5-inch, respectively. To assess the performance of the thinlays without being impacted by the performance of the underlying layers, a structurally sound pavement was needed underneath. Therefore, a 7-inch asphalt base layer produced with a highly modified binder (HiMA) binder was placed and compacted in one lift beneath the thinlays.

Sections were subjected to an additional 10 million ESALs of heavy truck traffic in the eighth research cycle and surface cracking, rutting, and smoothness in terms of the international roughness index (IRI), and friction were monitored. Rutting performance was excellent, with less than 0.2 inches of rut depth for both sections. Although IRI numbers for both sections were high from the beginning of the cycle because of the uneven finish of the underlying thick base layer placed in one lift, roughness remained stable for the cycle’s duration. Both sections had very low severity cracking of less than 2.4% of the lane area. Friction numbers of both sections remained stable with a slight reduction over time, but still above the safety threshold of 30 that has been established at the NCAT Test Track. Both thinlays constructed with high percentages of limestone were able to maintain an acceptable friction performance after 20 million ESALs.

The results of this study indicate that it is feasible to construct alternative thinlays on structurally sufficient pavements in Alabama and achieve satisfactory performance.

1.5 Florida Department of Transportation In-Place Density Study

In 2018, the Florida DOT initiated a field study to evaluate the impact of in-place density on the field performance of an asphalt overlay. A common axiom in the research literature suggests that a 1% reduction in in-place density from the standard 93% of G_{mm} could diminish pavement life by approximately 10%. This experiment was designed to provide evidence to support the use of in-place density as a key measure of the quality of asphalt pavement construction.

The Test Track experiment involved building four 100-foot subsections of an asphalt overlay with a range of target in-place densities ranging from 88% to 94% of G_{mm}. The mix design is a typical 12.5 mm NMAS Superpave surface mix containing 20% RAP and a PG 76-22 SBS-modified binder. Over the past five years, the study consisted of collecting field performance data and conducting numerous laboratory performance tests on the mixtures.

The field performance of the subsections has been measured weekly for rutting, IRI, mean profile depth, and cracking. Through two cycles of heavy trafficking, the subsections are still performing very well, as summarized in Table 1. Rutting is very low in all subsections, and IRI has changed very little. As expected, cracking is greater in the sections with lower in-place densities. However, the cracking extent and severity of all subsections are still low at this time.

TABLE 1 In-Place Density Experiment Summary of Field Performance Data at 20 Million ESALs

Subsection (Avg. In-Place Density)	Rut Depth, mm	Change in IRI, in./mi.	Mean Profile Depth, mm	Cracking, % of Lane Area	Cracking, % of Wheelpaths
E5A (93.6% of G_{mm})	0.9	9.8	1.13	2.6	3.2
E5B (92.0% of G_{mm})	1.3	13.3	1.20	5.1	4.0
E6A (87.8% of G_{mm})	1.2	-0.7	1.40	6.8	7.2
E6B (89.7% of G_{mm})	1.2	14.0	1.31	5.6	7.2

Laboratory performance tests were performed on specimens made from re-heated plant-produced mix. The mixtures were evaluated for cracking potential, rutting potential, and durability using seven different tests: Illinois Flexibility Index Test (I-FIT), Indirect Tensile Asphalt Cracking Test (IDEAL-CT), Cantabro Mass Loss, Energy Ratio, Dynamic Modulus (E*), Hamburg Wheel Tracking Test (HWTT), and High-Temperature Indirect Tension (HT-IDT). Performance test specimens were compacted to the in-place density targets of the test subsections: 6.5% air voids for E5A, 8.0% air voids for E5B, 12.0% air voids for E6A, and 10.0% air voids for E6B.

Results of I-FIT and IDEAL-CT reveal that lower densities yield higher results for these index tests, which is the opposite of the expected behavior. This issue has been reported by numerous other studies and highlights the importance of using the standard specimen air void target of 7.0 ± 0.5% for these tests. Otherwise, the results of the tests can be confusing and not indicative of actual pavement performance.

Hamburg Wheel Track results generally followed the expected trend of higher rut depths as specimen air voids increased. HT-IDT strengths decreased as specimen air void contents increased, as expected. The rutting test results were consistent with the excellent field rutting performance of the subsections.

Cores from each subsection were obtained after five years of service to evaluate the recovered binder properties. Results show that binders recovered from the subsections with lower in-place densities have aged more, becoming stiffer, and had a greater decline in relaxation properties than binders recovered from the higher-density subsections. Also, binders in the upper half of the cores have aged substantially more than the lower half of the cores, especially for the lower-density test subsections.

This ongoing study is poised to offer valuable data for in-place density criteria for asphalt pavement construction, potentially influencing future QA practices for state DOTs. With the test sections slated to remain until 2027, the research will continue to provide updated findings on the long-term effects of in-place density on pavement performance.

1.6 Georgia Department of Transportation Interlayer Study for Reflective Crack Prevention

The Georgia Department of Transportation (GDOT) aimed to evaluate innovative interlayer techniques for minimizing reflective cracking in pavements. The study utilized two experimental sections (N12 and N13) at the NCAT Test Track to compare different treatments over four trafficking phases. In 2012, two initial techniques were tested:

- A 0.70-inch double surface treatment with a sand seal coat.
- A 1.10-inch open-graded interlayer (OGI).

Following exposure to 20 million ESALs, the double surface treatment section exhibited reflective cracking in only 6% of the saw cuts. In contrast, the OGI section showed 50.5% of saw cuts reflecting through to the surface. The crack width in both sections was less than 0.24 inches (6 mm), which is considered low severity. Moreover, the OGI section demonstrated a lower final rut depth of 0.24 inches (6 mm), in contrast to 0.83 inches (21.0 mm) observed in the double surface treatment section.

In 2018, these sections were subdivided and expanded to six subsections to test additional treatments, including geosynthetic interlayers and chip seals with varying materials, which included:

- N12-A, GlasGrid® CG100 interlayer by ADFORS Saint-Gobain,
- N12-B, PETROMAT® fabric interlayer,
- N12-C, chip seal using No. 7 stone,
- N13-A, chip seal with reclaimed asphalt pavement (RAP),
- N13-B, Arizona-style asphalt rubber gap-graded (ARGG) interlayer, and
- N13-C, open-graded Interlayer (OGI).

The six subsections were exposed to 20.08 million ESALs, allowing for a detailed analysis of each treatment method's effectiveness. Performance monitoring included weekly ride quality evaluations, rut depth, surface texture, and cracking. Key findings revealed some variations in International Roughness Index (IRI), with minimal changes across subsections and differences in rut depth changes and cracking patterns, indicating similar efficacy of treatment methods.

Key observations—

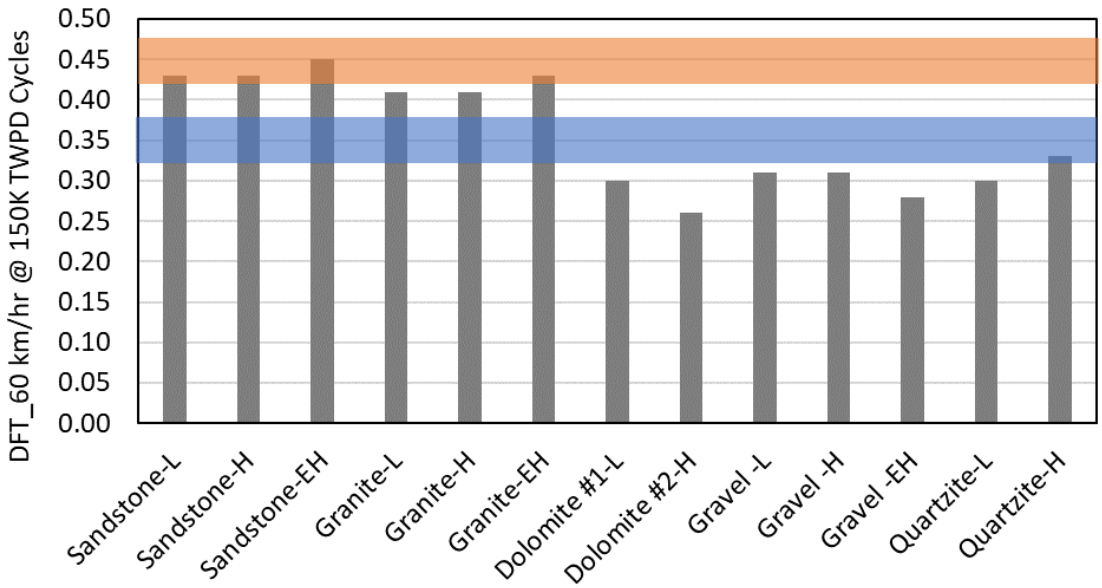
- A range of interlayer approaches can mitigate reflective cracking.
- An economic analysis should be conducted to evaluate similar approaches.
- This study highlights the importance of tailored treatment strategies to address reflective cracking, considering each pavement structure's specific conditions and demands.
- The ARGG used is likely too flexible for non-surface mix application.

GDOT's research provides valuable insights into the complexities of mitigating reflective cracking in asphalt pavements. By systematically evaluating a range of treatment methods, the study identifies promising solutions and underscores the necessity for ongoing innovation and testing in pavement maintenance and rehabilitation strategies.

1.7 Kentucky Transportation Cabinet Balanced Mix Design and Friction Experiment

In 2021, the Kentucky Transportation Cabinet (KYTC) began investigating implementing a Balanced Mix Design (BMD) specification using the Hamburg Wheel-Tracking Test (HWTT) for rutting and the Indirect Tensile Asphalt Cracking Test (IDEAL-CT) for cracking. This move aimed to extend pavement lifespans which led to the anticipation of increased traffic loading and the potential for more surface aggregate polishing. Concerned about skid resistance, KYTC included laboratory friction measurements into the BMD framework, prompting the concept of "BMD + Friction." The goal was to design for long-term skid resistance using lab tests alongside traditional BMD tests during mix design. Two half-sections (S7A and S7B) were placed in 2021, aiming for "medium" and "high" friction, respectively. Tests included the Three-Wheel Polishing Device (TWPD) and the Dynamic Friction Tester (DFT) for friction measurement, aiming to relate lab friction results to field performance within the BMD framework.

Following initial trials, KYTC found the proposed mix properties, volumetric criteria, and BMD threshold values for the S7A and S7B mixes to be overly stringent when combined with the friction targets. Due to a limited selection of aggregates suitable for achieving higher friction, the volumetric criteria were relaxed. Samples from numerous KYTC-approved aggregate sources were tested for their potential to meet friction requirements, shown in Figure 1. A total of 26 design blends with six different coarse aggregates were tested. Sandstone aggregate was chosen for the mix targeting high friction (mid 0.40's with the DFT after 150K cycles of TWPD polishing), while quartzite reached the medium friction range (mid-to-high 0.30's after 150K cycles of TWPD polishing). The selected BMD criteria were met in the laboratory design phase with relaxed volumetric requirements.



Note: L = low aggregate proportion in blend; H = high aggregate proportion in blend; EH = extremely high aggregate proportion in blend.

FIGURE 1 Initial aggregate blends to identify suitable friction aggregates.

Performance testing of the mix sampled during production yielded results different from mix design. The laboratory cracking resistance of the S7A "Medium" mix dramatically improved while the S7B "High" mix had decreased cracking results, falling below the BMD criterion of CTIndex of 90. Both mixes exhibited good rutting resistance. There was slight decrease in the DFT results from design to production, but not a statistically different result. After 10M ESALs of traffic on the Test Track, no cracking and very little rutting were recorded.

Locked-Wheel Friction Testing was conducted monthly on these sections using the ribbed tire and the skid numbers were reported (SN40R). The field friction performance for the two mixes is shown in Figure 2. Generally, the two sections met the friction expectations from the design, consistently yielding a difference of approximately 5 SN units apart (similar to the differences in the DFT results in design) and by finishing the 10M ESALs with SN40R results within the expected ranges from the mix design. Therefore, this study demonstrated that the DFT has the ability to identify mixes with superior friction performance in the mix design process and can be utilized to predict field friction from lab results.

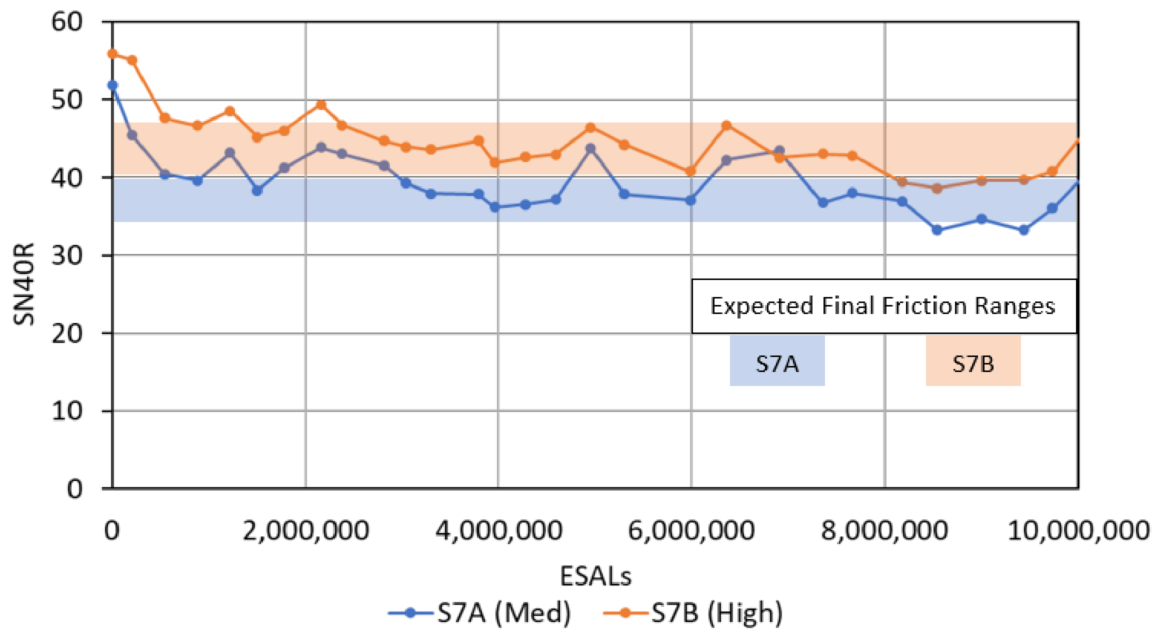


FIGURE 2 SN40R performance of S7A and S7B.

Key Takeaways—

- DFT results from laboratory-produced asphalt slabs can relate to field performance of the same mix and discriminate between mixes with varying friction characteristics.
- The TWPD/DFT system can be used in a mix design framework similar to BMD to eliminate mixes with poor friction properties and create mixes that will meet minimum friction levels in the field.
- Friction is primarily affected by aggregate type and gradation, and locking in an aggregate type or gradation to meet friction requirements will limit mix design options. Strict volumetric requirements will further restrict mix design options and prevent mixes with a potential for acceptable performance from being produced.

1.8 Mississippi Department of Transportation Stabilized Foundation Pavement

The Mississippi DOT (MDOT) primarily constructs semi-rigid pavements that often include both a cement or lime stabilized subgrade layer and a cement stabilized material (CSM) base layer. When MDOT began developing a plan to locally calibrate the performance models in the Mechanistic-Empirical Pavement Design Guide (MEPDG), neither the damage or distress transfer functions related to this pavement type had been calibrated at the national level (*1*). Therefore, there was a need to build a section at the Test Track that would reasonably replicate conditions in Mississippi to provide Level 1 inputs with well documented structural and performance data sets to enable future design and analysis of these pavement types using the MEPDG. To that end, MDOT sponsored section S2 which was constructed for the 2018 research cycle.

The construction of MDOT's section began by excavating approximately 5.5 ft of existing Test Track materials from Section S2. Soil classified as AASHTO A-6 (20) was imported from Mississippi and placed in eight lifts totaling approximately 56 inches. The top 6 inches of the MS soil was treated and mixed in place with dry hydrated lime, now called the lime treated soil (LTS) layer. Several weeks later a silty-sand base material, also imported from MS and classified as AASHTO A2-4, was placed on top of the lime-stabilized soil. This material was treated and mixed in place with cement and was allowed to cure for several more weeks and is now called the cement treated base (CTB) layer. Once the LTS and CTB layers were sufficiently cured, paving of the asphalt layers began. Four Superpave lifts were placed achieving a total asphalt concrete (AC) depth of 9.25". During construction, earth pressure cells, asphalt strain gauges and temperature probes were installed to provide mechanistic response and environmental condition data during trafficking, which has now reached approximately 20 million ESALs after two cycles of trafficking.

The surface performance of the stabilized foundation pavement has been excellent. Total rutting after the first 10 million ESALs was less than 0.20", no cracking was observed at the surface and smoothness did not change appreciably over time. Structural health monitoring, through frequent falling weight deflectometer testing and backcalculation, stress measurements, and strain measurements showed no significant changes over time indicating good structural health during the first 10 million ESALs.

During the second 10 million ESALs, rutting did not increase appreciably with rut depths still around 0.20" after 20 million ESALs. Some minor longitudinal cracking did appear between wheelpaths at approximately 15.3 million ESALs but was relatively minor and would, at times, disappear during different times of the year. At the conclusion of the 2021 Test Track, the cracking represented 1% of the total lane area but did not increase in severity since it was first observed. Like the first test cycle, ride quality did not change significantly during the second test cycle.

Measured vertical stresses in the section under truck loading were as expected, with stress decreasing with depth and increasing exponentially with a temperature of the asphalt concrete. Stresses deeper in the structure were less affected by temperature.

Strain measurements made at the bottom of the AC were very low (less than 100 microstrain in tension) which was expected given the thickness of the AC resting on the stabilized foundation. The very low tensile strain levels suggest that bottom-up fatigue cracking of the AC should not occur in this section. However, a surprising trend was observed where the tensile strain decreased with increasing temperature which is opposite of more conventional flexible pavements having unstabilized foundation layers. A critical AC temperature of 75°F was found where the mode changes from tension to compression. Below 75°F, the primary mode is tensile while above 75°F the bottom of the AC experiences compression. Simulations using the customized MASTIC software that follows layered elastic theory confirmed the measured pavement responses and also identified another critical zone at the mid-AC depth where tensile strains could reach a peak level. Repeated tension at this depth could lead to middle-up cracking in the wheelpaths. Further trafficking of the section should be conducted to evaluate this hypothesis.

To further investigate and validate the findings from S2, MDOT constructed an instrumented test section in Mississippi on State Route (S.R.) 76 that includes asphalt strain gauges located at two intermediate depths within the AC (*1*). These measurements will help to confirm the theoretical findings of strain reversal at these intermediate depths during hot weather months and investigate the occurrence of mid-level AC cracking in this type of pavement structure (*1*). It is anticipated that the findings may result in changes to the approach for designing intermediate lift mixes to consider crack-resistance in these layers (*1*).

While the S.R. 76 section is similar to S2, a key difference is that S2 included a lime stabilized subgrade while S.R. 76 includes a cement stabilized subgrade (*1*). Similar to S2, S.R. 76 will provide long-term in-situ modulus of both the cement stabilized subgrade and cement stabilized base layers to evaluate current design approaches recommended in the MEPDG (*1*). As with S2, this section will be monitored until it requires mill and fill, AC overlay, or some other rehabilitation (*1*).

1.9 Mississippi Department of Transportation Spray-On Rejuvenator Experiment

1.9.1 Overview and Objective

Asphalt binder near the pavement surface tends to become stiffer and more brittle over time due to age hardening from oxidation. This results in surface deterioration, such as non-load-associated distresses and top-down fatigue cracking. To combat this, spray-on rejuvenators, also known as rejuvenating seals, are used on asphalt surfaces to preserve their functional and structural integrity. These treatments can mitigate binder stiffness and brittleness by penetrating the top layer of the asphalt and renewing the aged binder. They may be combined with other materials, like emulsions or polymers, to seal surface cracks and prevent raveling.

These cost-effective treatments are typically reapplied every three to four years to extend the pavement's life. However, they are only recommended for certain surfaces and are not advisable for those with structural deficiencies. A critical consideration is that applying spray-on rejuvenators decreases friction and skid resistance, necessitating traffic control measures post-application.

This experiment, sponsored by the Mississippi Department of Transportation since the seventh research cycle (2018–2021), aimed to determine the short- and long-term effectiveness of two spray-on products in rejuvenating asphalt surfaces and evaluate their impact on surface friction following application. To ensure a thorough evaluation, the research cycle was extended from 2021 through 2024, allowing for ongoing monitoring and assessment. Performance was measured using rheological parameters and surface friction data.

1.10 Summary of Test Results and Research Findings

Two spray-on rejuvenator products were applied over the surface of Section S3, a 1.5” mill and inlay asphalt pavement section constructed in 2012, after the section experienced around 20 million equivalent single axle loads (ESALs) of traffic without presenting rutting and cracking distresses. The hot mix asphalt (HMA) of Section S3 was a dense-graded mix with sand and gravel containing 25% reclaimed asphalt pavement (RAP) and an asphalt content of 6.8%. The asphalt binder was a neat binder with a performance grade (PG) 67-22. The spray-on rejuvenator products utilized in this study were ReGenX® and Delta Mist™ , listed in this report as S3-A and S3-B, respectively. ReGenX® was applied at 0.07 gal/yd^2 (0.014 gal/yd^2 residual rate) for the first 97.5 feet (Section S3-A), and Delta Mist™ was applied at 0.10 gal/yd^2 (0.020 gal/yd^2 residual rate) for the final 75.3 feet (Section S3-B). The subsections were trafficked for 20 million ESALs from 2018 through 2024. The sections were monitored for rutting, cracking, ride quality, and surface macrotexture. Key takeaways from the research include the following—

- A decrease in the continuous true grade (i.e., pass/fail temperature) of the extracted binders present on the surface of the section was observed at high, intermediate, and low temperatures, indicating binder softening on treated sections.
- After a 48-month field aging interval, S3-B treatment resulted in lower continuous true grade at high and low temperatures compared to S3-A, with equal grades at intermediate temperature.
- Treatments improved cracking resistance, evidenced by less negative ΔT_c values even after a 48-month field aging interval, with maximum improvement observed at 6 months.
- Both spray-on rejuvenator products increased $J_{nr3.2}$ up to the 24-month field aging interval. However, the control and treated binders had almost negligible $J_{nr3.2}$ values and traffic rating “E” (E = extreme traffic loading).
- After a 30-month field aging interval, the treated sections presented $|G^*|_{60°C,\ 10\ rad/s}$ values less than the S3 control before treatment application: product S3-A showed a decrease in $|G^*|$ of 6.9%, while product S3-B showed a decrease of 14.5%. When comparing both treatments after a 48-month field aging interval, the S3-B product resulted in a binder with much lower stiffness (427.0 kPa) than the S3-A product (743.2 kPa). The same trend was observed for complex viscosity (η*).
- *G-R* parameter values for the S3-A product were lower than S3-B product values, capturing the higher softening effect of the S3-A treatment. As field aging progressed, a change occurred, and the binder treated with product S3-B presented lower *G-R* parameter values than S3-A up to 48 months after treatment application. The same trend was observed for PGH, $J_{nr3.2}$, and $|G^*|_{60°C,\ 10\ rad/s}$ data.
- When comparing the first obtained friction value after treatment application, product S3-A showed the highest decrease in friction (29.6%), while product S3-B showed the smallest decrease in friction (3.7%). Two weeks after treatment application, products S3-A and S3-B showed friction values equal to (0.27) and higher (0.30) than the S3

control section, respectively. Between 6 and 60 months after treatment application, S3-A and S3-B presented similar friction values, regardless of field aging interval.

- Rut depths were similar for Sections S3-A and S3-B (5.3 and 3.7 mm, respectively) and below the typical maximum field rut depth threshold of 12.5 mm.
- After approximately 65 months of treatment application and 20 million ESALs of truck traffic, the two treated sections remain below the maximum lane area cracked limit of 20%. The section treated with product S3-A presented 8.5% of lane area cracked, while the section treated with product S3-B presented 8.2%. The type of cracking observed was classified as block cracking.
- IRI values were 93.1/mile for S3-A and 79.2 in/mile for S3-B.
- At the end of truck trafficking for the eighth research cycle, MPD results were almost identical for the two subsections: 1.20 mm for S3-A and 1.25 mm for S3-B.

This NCAT field study has shown that the restoration capacity of a spray-on rejuvenating treatment increases rapidly after application due to a decrease in asphalt binder stiffness. However, it then begins to slowly decrease with oxidative aging due to binder embrittlement. Therefore, the 30-45 days aging time proposed in the FAA P-632 procedure, described in Section 1.3.2 of this report, can be misleading in assessing a spray-on rejuvenator product's long-term effectiveness.

In summary, the spray-on rejuvenator products assessed in this study are a good option for preventing or slowing the surface deterioration of pavements. They are convenient to use since they don't require specialized equipment and can effectively restore the surface condition of an existing pavement.

1.11 North Carolina Department of Transportation Effects of Tack Coat Type and Rate on Interface Shear Bond Strength

The enduring performance of asphalt pavements is intrinsically linked to the bonding between individual pavement layers. Ideally designed as a unified, flexible layer when subjected to load, the actual multi-layered construction of asphalt pavements depends on the bond strength at these interfaces to prevent distress. Tack coats are thus vital for pavement integrity, serving as the bonding agent between layers.

Different tack coat materials require different application rates. Over-application of tack coat can be as detrimental as under-application. Thus, their precise estimation and application are essential to ensure optimal bonding and, by extension, the longevity and performance of asphalt pavements.

The experiment evaluated the effects of tack coat type and rate on interface shear bond strength initially and over time under the influence of aging and traffic loading. It also aimed to assess the adequacy of the North Carolina Department of Transportation's (NCDOT) mixture specifications to produce stable, high recycled content mixes under realistic high-shear stress conditions.

1.12 Summary of Test Results and Research Findings

The experiment divided a 1.5" milled asphalt pavement section (Section W4) into three subsections (W4-A, W4-B, and W4-C). Different types and rates of tack coat were applied. A PG 67-22 binder was applied at 0.06 gal/yd^2 (0.060 gal/yd^2 residual rate) for the first 100 feet (Section W4-A), then CRS-1h was applied at 0.06 gal/yd^2 (0.036 gal/yd^2 residual rate) for the next 50 feet (Section W4-B) and at 0.10 gal/yd^2 (0.060 gal/yd^2 residual rate) for the final 50 feet (Section W4-C). The mix used a PG 58-28 neat binder, a blend of granite and sand, 25% RAP, and 4% RAS. The subsections were trafficked for 10 million ESALs from November 9, 2021, through April 3, 2024. The sections were monitored for rutting, cracking, ride quality, and surface macrotexture. Key takeaways from the research include the following—

- Section W4-B (CRS-1h applied at 0.06 gal/yd^2 with 0.036 gal/yd^2 residual rate) showed the highest interface shear bond strength across all aging intervals.
- Section W4-A (PG 67-22) and Section W4-B (CRS-1h) used 0.06 gal/yd^2 as the tack coat application rate but had different residual rates (W4-A, 0.060 gal/yd^2 residual rate; W4-B, 0.036 gal/yd^2 residual rate). Throughout 3 to 24 months post-construction, Section W4-A recorded a slightly higher bond strength increase of 38.4% compared to W4-B's increase of 35.9%.
- Section W4-C (CRS-1h applied at 0.10 gal/yd^2 with 0.060 gal/yd^2 residual rate) demonstrated the most significant increase in bond strength over time.
- All 3 subsections presented similar friction values, regardless of the field aging interval.
- Rut depths were less than 2.0 mm, which is below the typical 12.5 mm maximum field rut depth threshold.
- W4-A, W4-B, and W4-C have performed well, with no cracking reported during this research cycle, where approximately 10 million ESALs were applied by the end of fleet operations in 2024.
- The three subsections showed good ride quality (despite core extractions in the wheel path for bond strength evaluation affecting overall smoothness) and had almost identical, consistent surface macrotexture measurements throughout the research cycle.

In summary, tack coat type and rate can influence interface shear bond strength over time. Additionally, the experiment shows NCDOT's mixture specifications for high recycled content mixes are adequate under high-shear stress conditions. Field performance remained robust after significant traffic loading, indicating good durability and stability. The study supports maintaining these subsections for traffic continuation, allowing for a thorough field cracking and rutting performance evaluation.

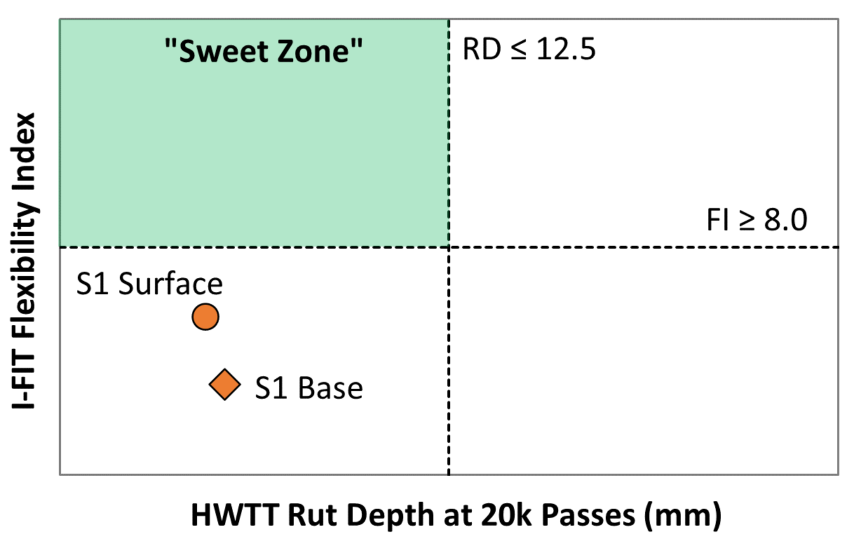
1.13 Oklahoma Balanced Mix Design Experiment

The Oklahoma Department of Transportation (ODOT) started implementing balanced mix design (BMD) in 2017. ODOT sponsored three BMD experiments at the NCAT Test Track, including S1 in the 2018 research cycle and N8 and N9 in the 2021 research cycle. The overall objective of these sections was to support ODOT with the implementation of BMD performance tests while exploring the responsible use of reclaimed asphalt pavement (RAP) and recycled tire rubber (RTR) in surface mixes using BMD. For section N9 in particular, an additional objective was to assess the performance of the rubber-modified mix and its potential to prevent reflection cracking from the underlying layer. The recurrence of reflective cracking with the new rubber-modified BMD is compared to the previous Test Track cycle conventional BMD mix to quantify the impact of the rubber additive.

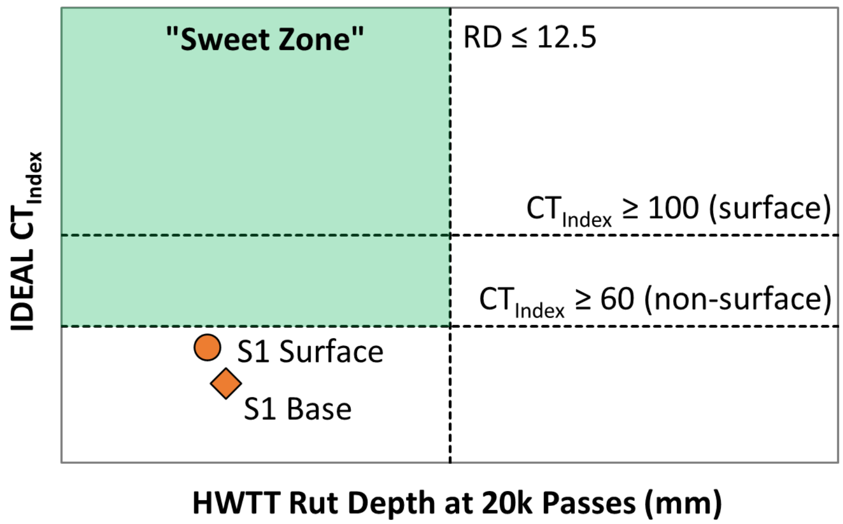
Summary of Test Results and Research Findings

<u>Section S1 – BMD with Low RAP</u>

Section S1 was built as a 5.0-inch mill-and-inlay section over an existing perpetual pavement with 24 inches of asphalt mixes. The section was placed in two layers, with the surface layer using an ODOT S4 mix [12.5mm nominal maximum aggregate size (NMAS)] with 12% RAP and the base layer using an ODOT S5 mix (19mm NMAS) with 30% RAP and a tall-oil-based recycling agent (RA). Both mixes were designed with the *Performance Modified Volumetric Design* approach per AASHTO PP 105, using the Hamburg Wheel Tracking Test (HWTT) for rutting evaluation and the Illinois Flexibility Index (I-FIT) for cracking evaluation. Quality control testing showed that both mixes met ODOT's production tolerance for asphalt binder content, air voids, and voids in mineral aggregate (VMA). Both mixes passed ODOT's HWTT criteria but failed the previous I-FIT criterion [i.e., minimum flexibility index (FI) of 8.0] and the current Indirect Tensile Asphalt Cracking Test (IDEAL-CT) criteria [i.e., minimum cracking tolerance index (CT_{Index}) of 100 for surface mixes and 60 for non-surface mixes]. As a result, they fell outside the "sweet zone" of the performance diagram due to inadequate cracking resistance, as shown in Figure 1. Despite the failing I-FIT and IDEAL-CT results, the section performed well, with minimal rutting, no cracking, and a steady international roughness index (IRI) after 20 million ESALs. The discrepancy between the I-FIT/IDEAL-CT results and field cracking performance data suggests that ODOT's BMD cracking criteria may be conservative for asphalt pavements with a robust underlying condition.



(a)



(b)

Figure 1. BMD Performance Diagram of Section S1 Surface and Base Mixes from Production Testing: (a) HWTT-versus-I-FIT, (b) HWTT-versus-IDEAL-CT

<u>Section N8 – BMD with High RAP and Recycling Agent</u>

Section N8 was built as a 5.5-inch mill-and-inlay section over an existing pavement from the 2015 Cracking Group experiment, which had 6.5 inches of asphalt mixes and 6 inches of aggregate base over the Test Track subgrade. Like Section S1, N8 was also placed in two layers. The surface layer used an ODOT S4 mix (12.5mm NMAS) with 30% RAP and a bio-based RA, and the base layer used the same ODOT S5 mix (19mm NMAS) in Section S1 from the 2018 research cycle. The N8 surface mix was designed at NCAT to match the volumetrics of the S1 surface mix from the 2018 research cycle. Quality control testing of the surface mix showed 1.3% more dust and 0.1% less asphalt binder at production compared to the job mix formula (JMF). As a result, the production mix had notably lower air voids and VMA, but the volume of effective binder (V_{be}) remained consistent with the JMF. Nevertheless, BMD performance testing showed that both the hot-compacted and reheated specimens passed ODOT's HWTT and IDEAL-CT criteria (i.e., minimum CT_{Index} of 80 in 2020 and 100 in 2023 for surface mixes) and thus, fell inside the "sweet zone" of the performance diagram, as shown in Figure 2. The section performed extremely well, with minimal rutting, no cracking, and steady IRI after 10 million ESALs, which agreed with the BMD test results. The outstanding laboratory results and field performance of this mix indicated that volumetric requirements could be relaxed when designing high RAP mixes using BMD provided that the performance test requirements can be met.

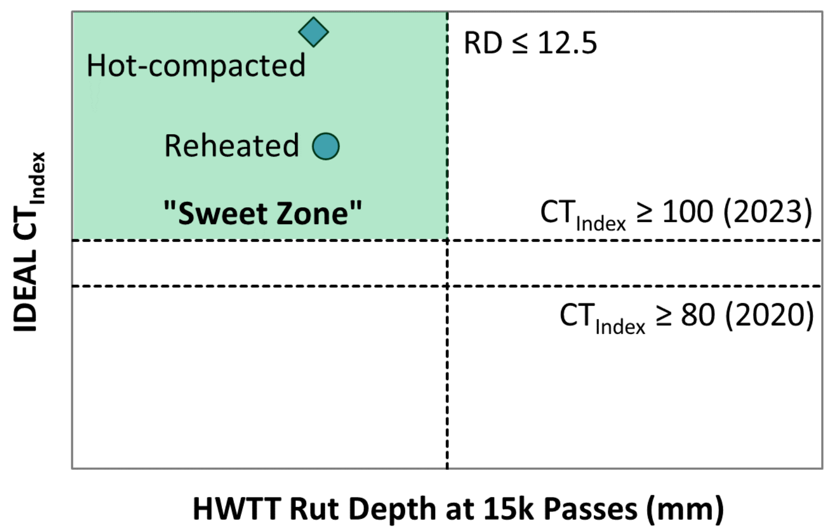


Figure 2. HWTT-versus-IDEAL-CT Performance Diagram of Section N8 Surface Mix from Production Testing

<u>Section N9 – BMD with Dry RTR</u>

N9 was built as a 1.5-inch mill-and-inlay section on a 14-inch asphalt pavement. The mix was designed as a 9.5mm NMAS mix with 15% RAP and 12% RTR added dry during production. The mix was intended to replicate the N9 (2018) surface mix using a similar aggregate structure and stockpiles, but replacing the PG 76-28 SBS binder with a PG 58-28 plus the RTR. The underlying structure was a perpetual pavement buildup where a small amount of top-down cracking still existed after milling. This cracking on the milled section was also present during the reconstruction activities of the 2018 cycle. As recommended by the RTR supplier, laboratory-prepared mixtures were required to be conditioned at 325°F for 4 hours to achieve the target compaction by “activating” the rubber. However, conditioning the mix at a higher temperature than the standard short-term oven aging temperature reduced the cracking resistance mix resulting in low IDEAL-CT CT_{Index} results. Therefore, the short-term aging procedure was modified to achieve compaction without sacrificing cracking performance. The modified procedure consisted of 30 min conditioning at aging at 325°F, followed by 3.5 hours of aging at 275°F. Upon completion of short-term aging, the mixture was brought back to 315-325°F range for compaction. A mix trial was developed at 5.8% binder content that met the preliminary ODOT CT_{Index} criterion of 80 but failed the rutting criterion due to early stripping. An additional mix trial that incorporated 0.5% liquid antistrip agent met the minimum HWTT criterion of 15,000 passes for a PG 70-xx, but the CT_{Index} dropped to 65. Considering that the N9 mix with this aggregate did not experience stripping failure in the field in the 2018 research cycle, and the special modifications needed to the aging procedure when incorporating the dry rubber, ODOT decided to move forward with this design at an optimum AC content of 5.8% AC and incorporating an antistrip agent. BMD testing conducted on plant-produced mix showed that the mix passed ODOT’s preliminary IDEAL-CT criterion, but failed the HWTT criterion due to stripping. Despite the discrepancies in the production mix results, no signs of moisture damage or significant rutting were evident in the field for this section. The cracking measured at the end of the research cycle was only 0.1%. When compared to the N9 (2018) section, the section had 3.3% cracking at the end of the research cycle. These results indicated that the current N9 mix has exceeded the cracking performance of the 2018 N9 mix.

Key Takeaways of the Experiment

The superior field performance of Sections S1 and N8 suggests that 1) it is feasible to design good-performing surface mixes with RAP using BMD; 2) ODOT's current IDEAL-CT criteria seem conservative for asphalt pavements with a robust underlying condition; and 3) to achieve full innovation benefits of BMD, volumetric requirements should be relaxed when designing high RAP mixes provided that the performance test requirements can be met.

In addition, although the N9 (2021) mix design incorporating RTR did not meet the preliminary BMD thresholds suggested by ODOT potentially due to differences in the mix conditioning procedure, the field performance results showed that the 2021 N9 mix outperformed the N9 mix from the previous cycle demonstrating that the dry rubber additive delayed the appearance of reflective cracking.

1.14 South Carolina Full-Depth Rapid Rebuild

The South Carolina DOT (SCDOT) sponsored a full-depth, thick-lift, rapid rebuild section in the 2018 research cycle. Their motivation was to further develop a rapid construction method that utilizes reduced lane closure lengths on major highways and primary routes resulting in less traffic disruption (*2*). They were also interested in developing an easy-to-compact and durable pavement that could be used as a riding surface that has less risk for failures at individual lift interfaces compared to conventional rebuilds (*2*). Developing knowledge of how the deep sections will respond to heavy loading on the Test Track in a short period of time was also of high interest to the SCDOT (*2*).

The section (S9) was constructed in August 2018 with an asphalt concrete (AC) depth of 8.05" placed in a single lift on a prepared crushed granite aggregate base. The AC was a dense graded 12.5 mm NMAS mix with a PG 64-22 binder and 25% RAP. The target mix design air voids was around 2.5 to 3% to make them easier to compact in the field as recommended for these mixes (*3*). The AC was produced as a warm mix and mat temperatures were monitored during construction. Paving initiated at 10 AM and it was found that the section required nearly 6 hours to cool from 242°F to 175°F. Simulations of pavement cooling were conducted with MultiCool software which were not very accurate for such a thick lift so caution should be exercised making future predictions with the software. However, paving at more advantageous times of day (i.e., early evening or night) will significantly reduce the cooling time to under two hours. This was confirmed both through simulation and measurements of a similar trial section placed at the Test Track.

Achieving density with an 8-inch lift was a non-issue. Density exceeding 95% of the theoretical maximum was accomplished with standard rollers and roller patterns. No specialized processes or equipment were needed. However, as-built smoothness may be an issue with thick-lift paving and certainly was with this section. The problem was rectified somewhat with diamond grinding. It is anticipated that paving crews, given more opportunities to pave thick-lifts, could greatly improve as-built smoothness. In practice, the SCDOT has found that having an additional lane to stage material transfer vehicles, trucks, and rollers is beneficial to minimize dips in the longitudinal pavement profile (*3*).

The thick-lift section exhibited excellent performance over the first 10 million ESALs. Rutting was less than 0.25”, very little cracking (likely top-down) developed, and smoothness did not change. Premature or excessive rutting, which was a potential liability for this construction technique, was not evident and should not be a problem provided that adequate compaction is achieved during construction. Further, Hamburg Wheel Tracking Test results of the AC confirm that rutting of the mix should not be a problem in this experiment.

The second 10 million ESALs did see a slight increase in rutting from 0.25” up to 0.30”. The slight top-down cracking seen during the first cycle increased in extent but not in severity growing from 0.7% of lane area at the end of the first cycle to approximately 10% of lane area. The ride quality demonstrated a slight downward trend from the first test cycle. These measures all indicated a section performing very well over 20 million ESALs.

The thick-lift section behaved in much the same fashion as other conventional multi-lift sections. The influence of pavement temperature was evident, and expected, in the measured pavement responses (i.e., stress and strain) and backcalculated AC moduli. The temperature-corrected data were remarkably consistent over time, indicating good structural health despite the small amount of cracking observed in the section.

Laboratory determination of AC dynamic modulus produced data consistent with backcalculated moduli. Either data set could be used for future mechanistic modeling of the test section. Additional trafficking and cracking development will be required for calibration and this section will be trafficked in the 2024 research cycle.

One area not yet explored with this section is the possible advantage of single lift construction not having lift interfaces that could suffer slippage and loss of structural integrity. This feature is inherent to the method of construction and could help alleviate this type of distress in addition to providing for a more rapid rebuild of the cross-section.

The SCDOT has used thick lift paving on the order of 4.5” to 6” of AC on several projects over the past few years (*2*). Project selection for this type of rapid rehabilitation involves identifying roads that can be reconstructed using thick lift paving in lieu of doing other rehabilitations methods that often take longer to construct and provide a reasonable cost and in turn provide an adequate pavement design (*2*). Through 20 million ESALs, the thick-lift section at the Test Track has shown their special mix design can be placed up to 8 inches and have very good long-term performance which means more projects can be considered for this type of rehabilitation.

1.15 Tennessee Department of Transportation Balanced Mix Design Using a Gyratory Compactor with Marshall Compactor Comparison

Marshall mix design is still one of the prominent mix design approaches globally and remains the mix design method of choice for the Tennessee Department of Transportation (TDOT). Meanwhile, TDOT is also exploring Balanced Mix Design (BMD) as a way to continually improve their mixes. However, most BMD test protocols were developed using the Superpave Gyratory Compactor (SGC) to compact specimens. TDOT wanted to know: (1) If proposed BMD test thresholds were appropriate and (2) whether the proposed BMD tests could be conducted on specimens compacted with the Marshall hammer as opposed to the SGC.

After a benchmarking study, TDOT selected two tests with subsequent thresholds to evaluate. First, the Hamburg Wheel Tracking Test (HWTT) was selected to evaluate the rutting susceptibility of mixes with a criterion of 12.5 mm max rut depth after 20,000 passes. With respect to cracking, TDOT selected the Indirect Tension Asphalt Cracking Test (IDEAL-CT) with a minimum threshold of CT_{Index} = 100. TDOT elected to investigate an accepted mix design in Tennessee that has achieved a CT_{Index} of 165. NCAT's first task was to adjust the mix design to get as close to the minimum CT_{Index} as possible. Raw materials were collected, and the following steps were taken to:

- Increase the RAP content from 28% RAP by weight of aggregate (original design) to 35% RAP, the maximum allowed by TDOT in base layers. Note that this is a surface layer design, which has a lower maximum RAP percentage.
- Decrease the asphalt content (6.1%) to the minimum asphalt content allowed by TDOT of 5.7%.
- Reduce the liquid antistrip (LAS) to the lowest value given that it may affect performance beyond the antistripping benefits.
- Reduce that total asphalt content to just below the minimum allowed by TDOT.

The selected design for construction had a CT_{Index} of 117 and included 35% RAP, 5.7% total binder content, and 0.3% LAS. Any further steps investigated caused the mix design to fall below the minimum CT_{Index} criteria of 100.

Test Section S4 was selected for paving this mixture. This was a mill and inlay section that contained 6 inches of aggregate base, 22.4 inches of existing hot mix asphalt, and a 1.6-inch overlay of the modified TDOT mix design that met a design CT_{Index} of 110 and met the HWTT criteria. Samples were taken during construction to check volumetrics and BMD tests, and showed the CT_{Index} dropped to 85.1, below the threshold of 100. The production mixture was

0.1% binder lower than the design, but well within production tolerance. However, this would reduce CT_{Index}. The HWTT passed with only 3.7 mm of rutting after 20,000 passes. Trafficking commenced with no cracking nor appreciable rutting or IRI change after 10 million ESALs.

But what about the Marshall vs SGC question for BMD tests? The theory is that the CT_{Index} equation will still apply since thickness and diameter are accounted for in the computation. Samples were produced both during mix design and during production using the SGC with a 6-inch diameter and Marshall hammer at a 4-inch diameter. In both cases, the Marshall compacted specimen yielded a lower CT_{Index} than its SGC counterpart. Given the limited dataset, it is only possible to state that they are in fact *not* the same. Further study is underway to evaluate the impact of specimen size on CT_{Index} results and that study will be released soon. The findings and conclusions from this study are as follows: (1) The TDOT BMD mix design thresholds yielded acceptable results on the NCAT Test Track after 10 million ESALs; (2) CT_{index} results collected during production (CT_{Index} = 85) were lower than the design (CT_{Index} = 110) and threshold (CT_{Index} = 100). This is not surprising considering the work of Wright et al. (*4*) which showed currently accepted production mixture variability can result in test values outside BMD thresholds; and (3) For this mixture, use of a 4-inch Marshall specimen for IDEAL-CT testing does not provide the same result as the standard 6-inch SGC specimen. Marshall compaction results were lower than the companion 6-inch SGC specimens.

TDOT plans to continue to investigate BMD on the test track while also looking to understand whether there is a connection between 6-inch SGC and 4-inch Marshall Specimens to enable their use for the IDEAL-CT.

1.16 Texas Balanced Mix Design Experiment

The Texas Department of Transportation (TxDOT) is one of the leading agencies in implementing balanced mix design (BMD). In the 2018 research cycle, TxDOT sponsored a BMD experiment consisting of sections S10 and S11 for field performance evaluation on the NCAT Test Track. A supplemental section, N6, was added to the experiment in the 2021 research cycle. TxDOT's experiment aimed to compare the field performance of asphalt mixes designed using a BMD approach (S10 and N6) versus the traditional volumetric approach (S11). All three sections were built as 2.5-inch mill-and-inlays over existing asphalt pavements with a challenging underlying condition to assess the surface mixes against reflective cracking.

Summary of Test Results and Research Findings

The S10, S11, and N6 mixes were designed by adjusting a TxDOT-approved 12.5 mm SP-C surface mix design. All three mixes used a PG 70-22 modified binder, fractionated reclaimed asphalt pavement (RAP), a blend of granite and dolomitic limestone aggregate, and had the same recycled binder ratio (RBR) of approximately 20%. The major differences among the three mixes were the gradation and the resultant optimum binder content (OBC) at 4.0% design air voids. The S10 BMD mix had a slightly coarser gradation and significantly higher OBC and voids in mineral aggregate (VMA) than the S11 volumetric mix, and the gradation and VMA of the N6

BMD mix were between those of S10 and S11. Performance testing of the mix design samples showed that all three mixes passed TxDOT's Hamburg Wheel Tracking Test (HWTT) requirements, but only the S10 and N6 mixes passed TxDOT's Overlay Test (OT) requirements.

Performance testing of the production samples showed consistent trends with the mix design samples. As shown in Figure 1, the S10 BMD mix passed both the OT and HWTT requirements and, thus, was expected to have balanced rutting and cracking resistance. The S11 volumetric mix passed HWTT but failed OT; as a result, it fell outside the "sweet zone" of the BMD performance diagram. The N6 BMD mix passed HWTT but had marginal OT results compared to TxDOT's criterion. For all three mixes, critical aging for 8 hours at 135°C was found to have a significant impact on the OT and Indirect Tensile Asphalt Cracking Test (IDEAL-CT) results. The mixes after critical aging were more susceptible to cracking due to increased mix embrittlement and reduced relaxation properties. Nevertheless, the S10 BMD mix showed significantly better cracking resistance than the S11 volumetric mix after critical aging, and the N6 BMD mix demonstrated laboratory cracking resistance between the S10 and S11 mixes.

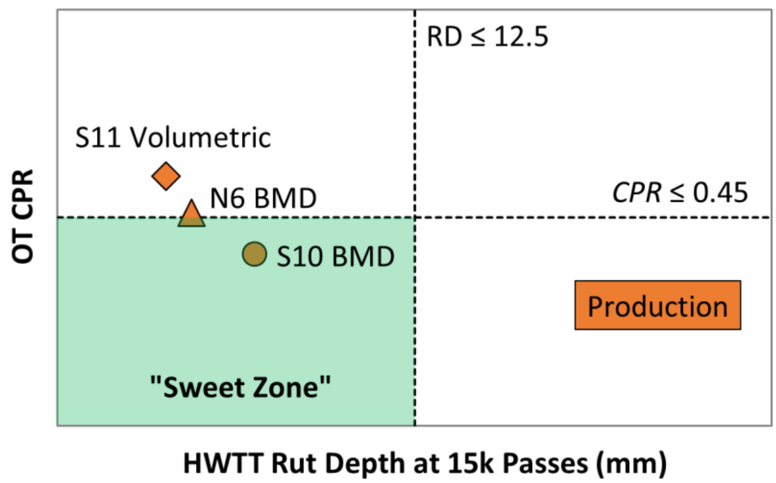
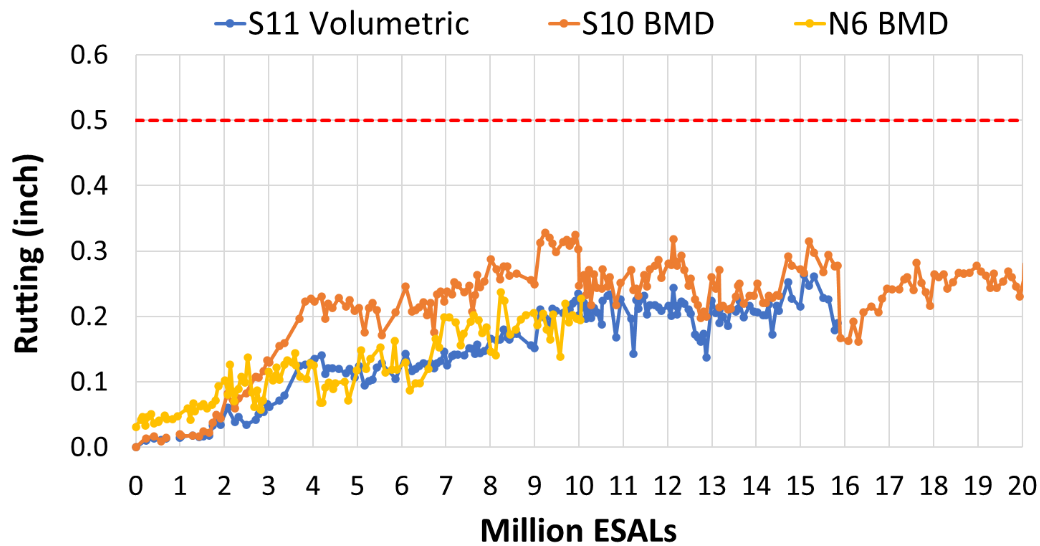
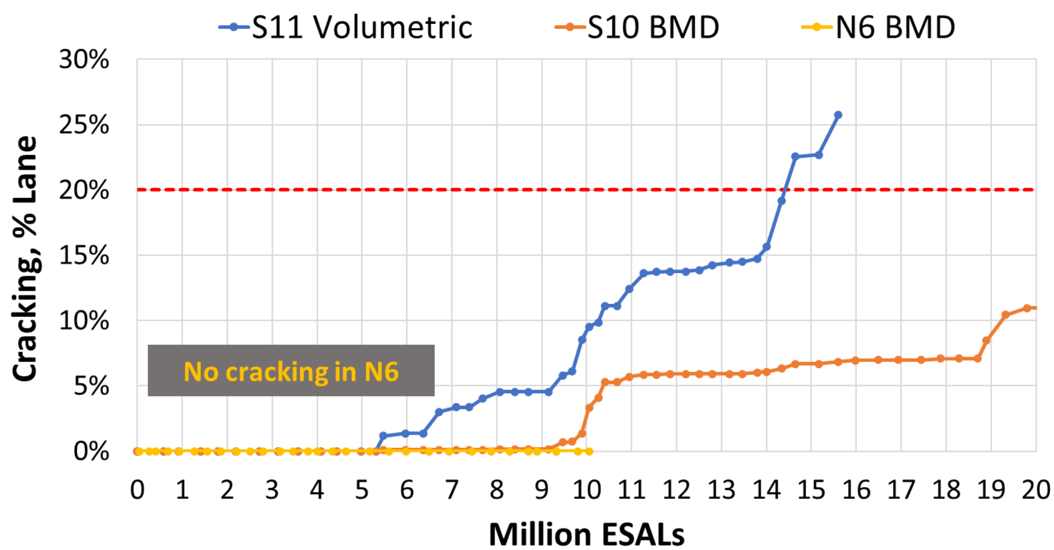


Figure 1. BMD Performance Diagram of S10 BMD, N6 BMD, and S11 Volumetric Mixes from Production Testing

Figure 2 presents the field rutting and cracking performance at the NCAT Test Track. Section S10, constructed in 2018, performed well with approximately 0.3 inches of rutting and 11.0% of cracked lane area after 20 million ESALs. Section S11, also built in 2018, had good rutting performance but reached over 20% of cracked lane area at around 14.5 million ESALs. The cracking in S11 progressed to approximately 25.7% before it was milled off at around 16.0 million ESALs. Section N6, constructed in 2021, performed well with minimal rutting and no cracking after 10 million ESALs. Overall, the field performance data of the three test sections agreed with the BMD performance test results. The cracking performance of Section S10 versus S11 demonstrates the benefit of BMD in extending the lifespan of asphalt overlays despite having a challenging underlying pavement condition.



(a)



(b)

Figure 2. Field Performance Data at the NCAT Test Track; (a) Rutting, (b) Cracking

Key Takeaways of the Experiment

- BMD allows performance enhancement of asphalt mixtures over the traditional mix design approach based on volumetric analysis. By incorporating mixture performance test requirements, BMD can effectively identify and eliminate asphalt mixtures that meet the volumetric requirements but lack performance properties.
- Mixture performance test results from both mix design and production testing agree with the field performance of pavement test sections on the NCAT Test Track, which indicates that HWTT, OT, and IDEAL-CT are good candidate tests for use in BMD for mix design approval and production acceptance.
- The cracking performance of Section S10 versus S11 demonstrates the benefit of BMD in extending the lifespan of asphalt overlays despite having a challenging underlying pavement condition.

1.17 Virginia Department of Transportation Long-Term Performance of CCPR Section N4

In 2012 the Virginia Department of Transportation initiated an experiment at the Test Track to evaluate Cold Central Plant Recycled (CCPR) asphalt pavements under three different scenarios: 5 inches CCPR with a 4-inch overlay over aggregate base, 5 inches of CCPR with a 6-inch overlay over aggregate base, and 5 inches of CCPR with a 4-inch overlay over an 8-inch cement stabilized layer. Just a year prior, VDOT had constructed similar sections on Interstate-81, just outside of Staunton, Virginia. While early performance on I-81 was promising, VDOT wanted to investigate (1) how the sections would perform under heavy traffic loads, (2) understand the stress and strain distribution within the sections (I-81 was not instrumented) (3) see how long the pavement would perform under accelerated loading, and (4) identify the failure mechanism and rate to inform future maintenance and rehabilitation strategies. After two test cycles (20M ESALS) all three sections were performing exceptionally well. VDOT decided to continue trafficking on section N4 (4" overlay on CCPR) and section S12 (4" overlay on CCPR on cement stabilized layer) and discontinue trafficking on N3 (6" overlay on CCPR). This would allow VDOT to take both sections to the equivalent levels of trafficking to previous perpetual pavement sections to support the notion that the recycled sections are, in fact, performing as perpetual pavements. This was confirmed during the 2018 Test Track Cycle for section S12, which performed perpetually. After 29.9 million ESALs high-resolution crack mapping images of Section N4 indicated that there was 0.1% of the wheelpath area cracked and 0.5% of the lane area cracked, indicating that there is actually more cracking outside of the wheelpath than within. It was decided to continue trafficking section N4 to yield insights into failure mechanisms of CCPR in a high-traffic environment, allowing VDOT to know what to look for and project an expected rate of deterioration for pavement management planning purposes.

The section far surpassed its expected performance with excellent rutting, IRI, and virtually no cracking through the first 30 million ESALs. All three performance measures showed degradation during the final 10 million ESALs, but this should not detract from the overwhelmingly positive performance history. In fact, the primary reason for leaving the section in place for the final 10 million ESALs was to gain a better understanding of how a CCPR pavement would deteriorate.

The strain sensors were not working for much of the 2021 test cycle. Although it is difficult to draw any definite conclusions, the section experienced significant strain levels (e.g., exceeding 1,000 $\mu\varepsilon$ in the summer) throughout the four test cycles, which contributed to the eventual fatigue cracking in the section. The vertical pressure data indicated a significant increase during the third and fourth test cycles. This was especially noticeable during the fourth cycle, where the measured pressures were approximately double that of the first cycle. This was attributed to structural pavement damage.

The backcalculated AC/CCPR moduli, much like the pressure data, showed clear signs of pavement damage during the third and fourth test cycles. When normalized for temperature to 68°F, the AC/CCPR moduli decreased by about 50% from the first test cycle to the last.

A forensic investigation will be conducted on the test section, along with additional laboratory testing, to better understand the deterioration mechanism of CCPR. It is notable that the section did not fail rapidly once cracking commenced, despite heavy loading. Overall, the test section performed remarkably and provides an implementable solution for high volume routes in addition to lower volume routes.

1.18 Virginia Department of Transportation Thin Overlay on Re-Recycled Cold Central Plant Recycled Asphalt Mixtures

Interest has continued to grow around the country regarding the use of Cold Central Plant Recycled asphalt mixtures (CCPR), especially in light of the influx of funding through the Bipartisan Infrastructure Law (BIL) and the programs like the FHWA Climate Challenge. CCPR is the process of using up to 100% RAP in addition to a recycling agent (foam or emulsified asphalt binder) and an active filler, often lime or cement, at ambient temperatures. Experience at the NCAT Test Track, Lee Road 159, and US 280, among others, has shown that these mixtures are high performers. Calculations based on Virginia bid price data found that the use of CCPR can significantly reduce the cost of an equivalent pavement structure (*5*). A study of Virginia's Interstate 81 recycling project noted that energy savings were estimated to be reduced by 50-70% and global warming potential by 40-70% compared to a conventional design when CCPR and other recycling techniques were used (*6*).

Recycling and achieving good performance is a strong case for using CCPR, but what happens when the CCPR eventually meets its expected service life? Can CCPR then be recycled *again*? This is a question that the Virginia Department of Transportation (VDOT) was wanting to answer, along with what happens when thinner overlays (2 inches) are placed on top of CCPR under heavy traffic conditions. Thus, the 2021 Test Track cycle was commenced by recovering the old CCPR from section S12 on the test track, stockpiling it, and then using it to produce a re-recycled CCPR mixture. The mix design process found the milled CCPR had a finer gradation than the original CCPR, which was not surprising considering the milling process used to pulverize a material that was not as stiff as the hot mix asphalt (HMA) that was used in the original (2012) CCPR being used as source material. Ultimately, the mix design found that the optimum foamed PG 67-22 asphalt content was identical to the S12-2012 CCPR at 2.0%, along with 1% Portland cement as an active filler. The final indirect tensile strength, both dry and wet, were reduced compared to the 2012 design but ultimately passed the specification criteria. The final structure consisted of a 6-inch granular aggregate base above the subgrade, with 5 inches of re-recycled CCPR, then a 2 inch stone matrix asphalt (SMA) mixture that contained a PG 76-22 binder.

Paving commenced by placing the CCPR on top of the aggregate base. However, it had rained previously, and the aggregate base had some retained moisture. Dynamic Cone Penetrometer testing was conducted to ensure adequate stiffness of the aggregate base, and the research team was comfortable with the results. The CCPR was placed and achieved target density, and the SMA overlay was placed the following day, again receiving adequate density. After 0.8 million ESALs visible cracking began to appear in the SMA. At 1.3 million ESALs significant deterioration had occurred with 14.1% cracking in the wheelpaths and 0.24 inches of rut depth. Coring was initiated after the initial cracking was observed (0.8 million ESALs) but yielded unsuccessful results, with the CCPR falling completely apart. This is indicative that the CCPR had not cured. It is believed that when the CCPR was being compacted, the vibratory roller pulled excess moisture up from the aggregate base into the CCPR layer. The SMA was placed the next day, and this likely only further exacerbated the problem as the vibratory roller would have been used once again only now an impervious "cap" was placed on the surface due to the overlay. This is believed to have led to the failure.

With the lessons learned from this experiment, the research team went back to the drawing board and decided to once again re-recycle the CCPR, henceforth referring to this section as the 3rd generation re-recycled CCPR. Materials were collected during the forensic period of the experiment, crushed using a laboratory jaw crusher, and a mix design was conducted. It was decided to use 15% SMA in the new blend to account for any additional materials lost through the milling and forensic process. Thus, the final mix design consisted of 85% RAP, 15% recycled SMA, 2.1% foamed asphalt, and 1% cement active filler. The wet and dry strengths were once again reduced, but still passed the mix design criteria. Low strength values are believed to be caused by the excess of binder in the mixture, both from the original RAP, the first round of CCPR, the re-recycling of CCPR, and then the 3rd generation re-recycling of CCPR. Prior to construction, care was taken to ensure that the aggregate base was dry. After the 5 inches of 3rd generation re-recycled CCPR was placed it was surface with two inches of SMA.

The 3rd generation re-recycled CCPR was placed around 3.2 million ESALs, and since then the performance of the mixture has been stable. No cracking has formed, and some rutting has occurred but stabilized. This rutting has been attributed to the SMA layer due to some variability in production. IRI has remained consistent. Using FWD data, the average backcalculated AC/CCPR modulus at 68F is 254 ksi with data collected from construction of the 3rd generation re-recycled CCPR through 10 ESALs.

There are a few key findings from this study: First, it is possible to re-recycle CCPR and get good early performance. This opens up opportunities for continual recycling of our roadway infrastructure in cost effective, environmentally conscious ways. Second, surfacing with only two inches of SMA has provided good early performance. Finally, lessons were learned regarding inadequate curing of the CCPR and, namely, the potential impact of putting a roller in vibratory mode on top of the CCPR and overlay when the aggregate base may still be damp. For the 2024 Test Track Cycle, VDOT will resurface this section without compromising the 3rd generation re-recycled CCPR to continue studying the impact of both re-recycling CCPR as well as using a thin overlay over CCPR under heavy truck traffic.

1.19 Dense Graded Asphalt Thinlay on CCPR and Rejuvenated CCPR

What happens when you place Cold Central Plant Recycled asphalt mixtures (CCPR) beneath a thin overlay and subject it to heavy truck loading? How about if you add a rejuvenating agent to the mixture? This was the premise of the study conducted on the off ramp at the NCAT Test Track. Six 100-foot sections were constructed, each with a different base mixture (HMA or CCPR) along with a one inch thick 4.75 mm Nominal Maximum Aggregate Size (NMAS) overlay. CCPR consists of Reclaimed Asphalt Pavement (RAP) millings, often 100% RAP, along with a recycling agent (foam or emulsified asphalt binder), and an active filler (lime or cement) if needed. There has been increasing curiosity in what the impact of using rejuvenating agents would be on CCPR, especially in light of their use in hot mix asphalt mixtures. While research on typical CR pavements containing foamed asphalt binder or emulsion and their behavior is prevalent, little work has been published on the performance of CR pavements containing rejuvenators. Of the limited work, Bowers et al. (*7*) found that a rejuvenated CCPR mixture produced dynamic modulus (stiffness) values between that of hot mix asphalt and conventional cold recycled mixtures.

To that end, the offramp was constructed with three conventional asphalt products, an HMA base and both a foamed and an emulsified CCPR, along with three rejuvenated CCPR products to compare their performance to the conventional methods. The sections each had 6 inches of aggregate base with 5 inches of the following (section designation follows mix type): HMA control (R5), foamed asphalt with active filler (cement) (R6), engineered emulsion (R7), emulsified bio-based rejuvenator (R8), CR rejuvenator (R9), and anionic emulsion with a bio-based rejuvenator (R10). Three plants were used to produce the mixtures. A conventional HMA plant was used to produce the HMA mixture, a Wirtgen KMA for the foamed and emulsified CCPR, and a portable pugmill provided by Pavement Restorations, Inc. was used for the rejuvenated sections. The CCPR plants were selected by the research sponsors based on their expected application.

The CCPR mixes were all produced according to designs from either NCAT (foamed asphalt, CR rejuvenator, and anionic emulsion + bio-based rejuvenator) or the product manufacturer (engineered emulsion and emulsified bio-based rejuvenator). Materials designed with rejuvenator surpassed requirements for emulsified asphalt cold recycled mixtures, which is based on Marshall stability testing, with the exception of the emulsified bio-based rejuvenator which passed the foamed asphalt cold recycled mixture specifications, requiring indirect tensile testing. All mixes were placed without problems with exception for the anionic emulsion with bio-based rejuvenator (R10), which has bleeding/flushing issues during compaction. This is believed to be caused by excess water in the mixture from the hydroscopic moisture content of the RAP and the water in the anionic emulsion. The mixture was allowed to sit for a few hours and was then compacted without any issues. The asphalt overlay was applied after at least two days of curing.

Field performance has been section dependent. The HMA, foamed asphalt, engineered emulsion asphalt, and emulsified bio-based rejuvenator sections have exhibited no cracking and consistent IRI, and rutting has remained below 4 mm over the life of the pavement. The CR rejuvenator section has exhibited very minor cracking (0.4% of total lane area) and the anionic emulsion with bio-based rejuvenator section has shown cracking around 3% of the total lane area. The increase in IRI and rutting began to accelerate around 21,000 equivalent standard axle load (ESAL) applications for the Anionic Emulsion with Bio-Based Rejuvenator section. It is hypothesized that the increased IRI and rut depth for the anionic emulsion with bio-based rejuvenator section could be a result of the elevated moisture contents at the time of placement, as mentioned previously, combined with a slower cure process due to the anionic emulsion. This would cause a lack of stability in the base beneath the thinlay, which would lead to both rutting and IRI issues, as well as cracking, which appeared around 46,000 ESALs. The increase in rutting for the CR rejuvenator began around the same time as the anionic emulsion with bio-based rejuvenator, but it was not as intense. The IRI did not start to increase until approximately 37,000 ESALs. While curing may have also been an issue for this section, the rejuvenator did not include any emulsion or other recycling agent which would add additional water to the mixture. This section has stabilized and maintained an average rut depth of 7.2 mm since approximately 54,000 ESALs.

The major findings and conclusions of this study are as follows: (1) The plant production and construction of the rejuvenated CCPR mixtures was like a typical CCPR mixture; (2) some, but not all, rejuvenated CCPR mixtures can perform at least equivalent to typical CCPR mixture and DGA mixture with a thin overlay; (3) The IRI and rutting increased for both the anionic emulsion with bio-based rejuvenator section as well as the CR rejuvenator section. This is believed to be a byproduct of high moisture levels in the mixture during compaction.

1.20 Cargill Evaluation of BMD Mixture with High RAP and Anova Asphalt Rejuvenator in Comparison to Lower RAP Mix with Warm Mix/Compaction Aid Additive

Approximately 100 million tons of asphalt materials are removed from roads annually. These materials can be reused as reclaimed asphalt pavement (RAP) in new asphalt mixtures, which can help reduce material expenses, conserve natural resources, and save landfill space. Despite the potential for increased RAP content, the national average RAP content in new asphalt mixtures has remained around 22% in recent years as state departments of transportation

(DOTs) are hesitant to permit higher RAP contents due to concerns about the performance of asphalt pavements, which can result in higher maintenance costs.

One of the concerns for high RAP mixtures is related to the effect of oxidized asphalt binder from RAP on mixture resistance to cracking. To minimize the impact of oxidized asphalt binder on the quality of asphalt mixtures produced with high RAP contents, especially their resistance to cracking, several approaches have been evaluated over the years. One of these approaches is to use a recycling agent to help achieve the desired performance grade (PG) or rheological properties of the RAP binder or the total binder blend. In addition, the optimum contents of the recycling agent and virgin binder can be determined based on a balanced mix design (BMD) approach to further improve the overall mixture resistance to rutting and cracking.

Following these approaches, a high (45%) RAP surface mixture was designed using Cargill's Anova™ 1815 asphalt recycling agent following a BMD approach. The Anova recycling agent was used to restore the performance properties of the RAP binder, which helped mitigate the impact of high RAP content on the long-term performance of the asphalt mixture. This mixture was compared to a lower (30%) RAP surface mixture previously approved for paving in Virginia, which contained Cargill's Anova™ 1501 additive (as an adhesion promotor/warm mix/compaction aid). Both mixtures were paved on the same pavement structure and tested under the same traffic and climatic conditions in two Test Track research cycles from 2018 through 2024. Their field performance, including rutting, cracking, ride quality, and surface macrotexture, was monitored weekly.

The 45% RAP mixture was also produced without the recycling agent for laboratory testing only. The three plant-produced mixtures were sampled during construction and tested using several performance tests, and the data were analyzed to assist the field evaluation at the Test Track.

Before the construction, two asphalt mix designs were conducted according to the BMD provisional specification of the Virginia Department of Transportation (VDOT). The BMD provisional specification requires three laboratory tests to evaluate the resistance of asphalt mixture to rutting, cracking, and raveling. These tests include the Asphalt Pavement Analyzer (APA) with a maximum rut depth of 8.0 mm after 8,000 cycles, the Indirect Tensile Asphalt Cracking Test (IDEAL-CT) with a minimum CT_{Index} of 70, and the Cantabro abrasion test with a maximum mass loss of 7.5%.

The mix design process includes three steps. First, two volumetric mix designs were developed with 30% RAP and 45% RAP. These mixtures were then tested to determine their APA rutting and CT_{Index}. While their APA rut depths met the BMD provisional specification, their CT_{Index} values did not, and the CT_{Index} of the 45% RAP mixture was lower than that of the 30% RAP mixture. Second, the recycling agent was added to the 45% RAP mixture to achieve APA rutting and CT_{Index} results similar to those of the 30% RAP mixture. Finally, the binder contents of both mix designs were increased to meet the minimum CT_{Index} requirements set by VDOT. The 30% RAP mixture met the CT_{Index} threshold at a total binder content of 5.5% with the Anova™ 1501 additive as a compaction aid (CA), referred to as 30% RAP + CA mixture, while the 45% RAP mixture met the requirement at a total binder content of 5.8% using Anova™ 1815 asphalt recycling agent (RA), referred to as 45% RAP + RA mixture. These mixtures met the BMD provisional specification, with their CT_{Index} and APA rut depth results being almost the same and meeting the VDOT maximum Cantabro loss threshold of 7.5%.

The BMD mixtures were produced, placed, and compacted to achieve good in-place density (i.e., 96.2% for 30% RAP + CA and 96.8% for 45% RAP + RA) on the Test Track. Based on the laboratory performance test results of reheated plant mixtures, the two mixtures would have similar rutting and cracking resistance. Furthermore, the laboratory cracking test results showed a significant decrease in I-FIT, OT, and IDEAL-CT results after critical aging, representing approximately five years of field aging at the Test Track.

The 45% RAP mixture was also produced without the recycling agent to compare with the 45% RAP + RA mixture. The difference in cracking test results for the 45% RAP mixtures produced with and without the recycling agent showed the positive impact of the recycling agent on the cracking resistance of the reheated plant mixtures. However, after critical aging for eight hours at 135°C, which is representative of four to five years of field aging at the Test Track, the cracking test results of the three plant mixtures (i.e., 30% RAP + CA, 45% RAP + RA and 45% RAP) were reduced significantly and similar to each other.

Field performance data collected from Sections N3A (30% RAP + CA) and N3B (45% RAP + RA) on the Test Track showed that both sections had similar ride quality and surface macrotexture at the end of the eighth research cycle. During the 8th research cycle, there was a slight increase in rutting in Section N3B, which rose from 5.0 mm at the end of the 7th research cycle to approximately 7.5 mm. However, in Section N3A, the rutting remained almost the same as it was at the end of the 7th research cycle, at approximately 5.0 mm. These rut depths are below the typical maximum field rut depth of 12.5 mm, which agrees with the good APA and HWTT results for the reheated plant-produced mixtures sampled during construction.

In addition, hairline cracks were observed in both sections towards the end of the seventh research cycle. These cracks continued to grow slowly in both sections in the eighth research cycle, with 2.5% of the lane area observed in Section N3A and 1.3% in Section N3B. Both sections will be kept in place for traffic continuation to continue monitoring the cracking performance of these test sections in the next research cycle.

Key takeaways from the research include the following—

- The BMD approach was successfully used to design both 30% RAP + CA and 45% RAP + RA mixtures to improve its resistance to cracking and raveling without causing a detrimental effect on its resistance to rutting.
- Both sections showed good field rutting performance, with rut depths below the typical maximum field rut depth of 12.5 mm, which agrees with the APA and HWTT results on the reheated plant-produced mixtures.
- Both sections exhibited good cracking performance, with hairline cracks observed in 2.5% of the lane area in Section N3A and 1.3% in Section N3B at the end of the eighth research cycle. Lab cracking tests also showed a significant decrease in I-FIT, OT, and IDEAL-CT test results after critical aging. Thus, it is essential to continue monitoring the cracking performance of these test sections in the next research cycle.
- Both sections showed good ride quality and almost identical, consistent surface macrotexture measurements throughout the two research cycles.
- When comparing the experimental mixtures produced with and without the rejuvenator, the cracking test results were more different on the reheated plant-produced mixtures than on the critically aged plant mixtures.

In summary, the Anova asphalt recycling agent can improve the cracking resistance of a high RAP mixture within the BMD framework without affecting the mixture resistance to rutting. Sections N3A and N3B showed good field performance after 20 million ESALs without significant distress. They will be kept in place for traffic continuation in the next research cycle to allow for a more thorough field cracking and rutting performance evaluation.

1.21 Soylei Bio-polymer Modified Asphalt Mixture

Polymers used in asphalt paving materials have been traditionally made from petroleum-based products, but they can now be made from bio-based feedstocks. Iowa State University has developed a bio-derived polymer through the transesterification of glycerol from triglycerides found in epoxidized soybean oil using benzyl alcohol. This innovative biopolymer, which includes epoxidized benzyl soyate (EBS), can enhance asphalt binder resistance to oxidative aging as the epoxide rings within EBS react to form crosslinks, effectively blocking nucleophilic sites where asphalt oxidation occurs. The objective of this experiment was to evaluate the new biopolymer in a field experiment on the Test Track.

For this study, the biopolymer modified binder was blended at an asphalt terminal and transported to East Alabama Paving's plant in Opelika, Alabama. The biopolymer modified binder was utilized to produce an asphalt mixture that was paved in the surface layer of Section W10 at the Test Track on November 16, 2018. This test section was compared with Section E5A, which was surfaced with a conventional styrene-butadiene-styrene (SBS) polymer modified mixture on August 29, 2018. The target lift thickness for both mixtures was 1.5 inches, and the mix temperature was around 320°F. The in-place density was comparable for both sections, with Section E5A having a density of 93.2% of G_{mm} and Section W10 having a density of 93.3% of G_{mm}.

Both test sections were monitored for rutting, cracking, smoothness, and macrotexture on a weekly basis under the same heavy truck traffic loading conditions at the Test Track. Additionally, samples of asphalt binder and loose mix were collected during construction for laboratory testing to assist the field performance evaluation.

The binder used in Section W10 was modified with the biopolymer to achieve a performance grade similar to that of the conventional SBS modified binder used in Section E5A. The biopolymer modified binder had a true grade of 73.6 – 21.6 with ΔT_c of -3.0, while the conventional SBS modified binder had a true grade of 76.3 – 24.7 with ΔT_c of -2.6.

The Dynamic Modulus test results showed that the biopolymer modified mixture was stiffer than the SBS modified mixture across most temperatures and frequencies tested. This observation was consistent with the other laboratory test results for rutting, cracking, and mix durability.

Both mixtures showed good rutting resistance in the Hamburg test, with rutting below 2 mm and no signs of stripping. Therefore, neither mixture was expected to show significant rutting on the Test Track. The I-FIT and IDEAL-CT cracking tests, conducted at 25°C, showed that the SBS modified mix may have better cracking resistance than the biopolymer modified mixture. Additionally, the DCT test, which measures low temperature cracking resistance, conducted at -12°C), indicated that the SBS modified mixture may have better low temperature cracking resistance than the biopolymer modified mixture.

After 20 million ESALs of trafficking at the Test Track, both test sections have shown good field performance thus far. They showed good rutting with final rut depths below 5.0 mm. At the end of the eighth research cycle, low-severity cracking was observed in both sections, with 2.7% of the lane area in Section E5A and 3.4% of the lane area in Section W10.

The key takeaways of the experiment include—

- The new biopolymer can be used by itself or in combination with other modifiers to modify asphalt binders to achieve performance grades similar to conventional SBS modified asphalt binders.
- The laboratory test results indicated that the mixture modified with biopolymer was less resistant to cracking and stiffer. However, the field performance revealed different results. After a traffic load of 20 million ESALs at the Test Track, both mixtures showed good rutting, with final rut depths below 5.0 mm. The sections exhibited only low-severity cracking, with 2.7% of the lane area in Section E5A and 3.4% of the lane area.
- Both Section E5A and Section W10 will be kept in place for the ninth research cycle. Section E5A will be subjected to continued trafficking. However, Section W10 will be

split into two subsections. One of these subsections will be treated with a spray-on rejuvenator, while the other will be used as the control. This experiment aims to assess the effectiveness of the rejuvenator spray while also evaluating the long-term performance of the biopolymer modified asphalt binder compared to the conventional SBS modified asphalt.

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1.22 US Polyco Binder Formulation

In 2021, US Polyco sponsored a full-scale test section at the NCAT test track featuring a new binder formulation referred to as Sigmabond HP comprised of styrene-butadiene-styrene (SBS) polymer and Sigmabond Tire Rubber (TR). The objective of the research was to design, produce, and pave an asphalt mixture with US Polyco-modified binder Sigmabond HP on Section S8 of the NCAT Test Track and to compare its performance with that of a control mix with a conventional SBS-modified binder asphalt mixture. The control mix with a PG 76-22 SBS binder was built on Section N7 as part of the Additive Group Study. The design for both sections included a 5.5-inch asphalt layer over a 6-inch aggregate base on top of the existing Test Track subgrade. With this design, bottom-up fatigue cracking was anticipated as the failure mode.

The experimental plan was divided into two phases. Phase I consisted of a laboratory characterization using a dense-graded asphalt mix. The mixture was designed using a Sigmabond

HP-modified binder and an SBS-modified binder following a balanced mix design (BMD) approach to evaluate their rutting and cracking resistance using the Hamburg Wheel Tracking Test (HWTT) and IDEAL-CT. In addition, the mixtures were characterized to conduct theoretical structural pavement analysis in terms of fatigue performance using dynamic modulus (E*) and cyclic fatigue as inputs for WESLEA and FlexPave™ programs to predict equivalent pavement fatigue cracking performance and provisional asphalt layer structural coefficients. Phase II included the construction, trafficking, monitoring, and performance evaluation of the test sections for the duration of the test track cycle.

Key findings are summarized as follows:

<u>Phase I</u>

- The BMD evaluation showed that the US Polyco mix significantly exceeded the IDEAL-CT CT_{Index} of the control SBS-modified mix and had comparable rutting performance in the HWTT.
- The structural analyses conducted with WESLEA and FlexPave™ using dynamic modulus and cyclic fatigue test results showed improved structural capacity of the US Polyco mix compared to the control mix.

<u>Phase II</u>

- As constructed results showed, the US Polyco mix had comparable quality control data as the control mix regarding gradation and AC content, but the control section had a higher in-place density and was built 0.4in thicker.
- Performance test results conducted on plant-mixes showed that the US Polyco mix had lower IDEAL-CT CT_{Index} than the control and had comparable rutting performance in HWTT.
- The US Polyco experimental section experienced isolated cracking at approximately 35 feet into the section, which was attributed to lower than intended polymer content (degradation), possibly as a result of the tanker binder around the tube being heat damaged. Partial reconstruction of approximately 75 feet (section S8A) of the US Polyco section occurred in October 2022.

At the end of the cycle, after 10 million ESALs, sections S8A and S8B have comparable performance as the control mix with no cracking and comparable rutting. The excellent field performance supported the decision to leave approximately 2/3 of the original section in place for the remaining of the research cy