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Structural Study of Cold Central Plant Recycling Sections at the National Center for Asphalt Technology (NCAT) Test Track

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Abstract:

In 2012, the Virginia Department of Transportation (VDOT) contracted with the National Center for Asphalt Technology (NCAT) to install, instrument, and monitor three pavement test sections at the NCAT Test Track during the 2012-2014 track cycle. The work consisted of constructing, instrumenting, and trafficking the test sections with heavily loaded trucks until approximately 10 million 18-kip equivalent single axle loads (ESALs) were applied. Embedded instruments were installed to capture the temperature and pavement response from truck loading. The three test sections, having a length of 200 ft each, consisted of two different asphalt overlay thicknesses placed on top of a five-in cold central-plant recycled base. One of the three sections also contained a cement-stabilized base designed to simulate a full-depth reclaimed layer.

The purpose of this study was to evaluate the performance of the three test sections constructed using cold central plant recycling (CCPR) over the initial 2-year track cycle. The performance was documented by analyzing the results of laboratory testing from collected cores, as well as deflection testing from falling weight deflectometer, temperature, pressure, and strain measurements from embedded instruments, and surface-observable deterioration of the pavement sections.

The study found that none of the three sections showed any surface-observable deterioration after 10 million ESALs of loading. Throughout the cycle, the average measured strain from Section N3 (having a 6-in asphalt overlay) was 40% less at 68°F than that of Section N4 (having a 4-in asphalt overlay). The strain from Section S12 (having a 4-in asphalt overlay and a cement-stabilized foundation) was approximately 69% and 49% less than the strain levels for Sections N3 and N4, respectively, at 68°F. The structural layer coefficient of the CCPR material was estimated to range from 0.36 to 0.39 based on falling weight deflectometer testing. The temperature-normalized asphalt mixture/CCPR modulus of Section S12 was found to increase with respect to time. This indicates that the cement-stabilized foundation is increasing in strength over time, likely attributable to continued curing of the layer.

The study recommends that VDOT continue to emphasize the use of pavement recycling methods for new pavement construction and pavement rehabilitation projects. To this end, VDOT will work to identify locations for future pavement recycling projects where performance data suggest that maintenance activities take place more often than the average. VDOT will also review existing memoranda with district pavement management and design staff that state pavement recycling should be considered for projects where it is a viable option.

This study shows that the three pavement designs used in the three test sections constructed at the NCAT Test Track to be adequate for a minimum of 10 million ESALs and likely much longer. This report is an interim report in that the test sections are still being trafficked. A final report will be prepared upon the completion of testing.

FINAL REPORT

STRUCTURAL STUDY OF COLD CENTRAL PLANT RECYCLING SECTIONS AT THE NATIONAL CENTER FOR ASPHALT TECHNOLOGY (NCAT) TEST TRACK

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ABSTRACT

In 2012, the Virginia Department of Transportation (VDOT) contracted with the National Center for Asphalt Technology (NCAT) to install, instrument, and monitor three pavement test sections at the NCAT Test Track during the 2012-2014 track cycle. The work consisted of constructing, instrumenting, and trafficking the test sections with heavily loaded trucks until approximately 10 million 18-kip equivalent single axle loads (ESALs) were applied. Embedded instruments were installed to capture the temperature and pavement response from truck loading. The three test sections, having a length of 200 ft each, consisted of two different asphalt overlay thicknesses placed on top of a five-in cold central-plant recycled base. One of the three sections also contained a cement-stabilized base designed to simulate a full-depth reclaimed layer.

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The study found that none of the three sections showed any surface-observable deterioration after 10 million ESALs of loading. Throughout the cycle, the average measured strain from Section N3 (having a 6-in asphalt overlay) was 40% less at 68°F than that of Section N4 (having a 4-in asphalt overlay). The strain from Section S12 (having a 4-in asphalt overlay and a cement-stabilized foundation) was approximately 69% and 49% less than the strain levels for Sections N3 and N4, respectively, at 68°F. The structural layer coefficient of the CCPR material was estimated to range from 0.36 to 0.39 based on falling weight deflectometer testing. The temperature-normalized asphalt mixture/CCPR modulus of Section S12 was found to increase with respect to time. This indicates that the cement-stabilized foundation is increasing in strength over time, likely attributable to continued curing of the layer.

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This study shows that the three pavement designs used in the three test sections constructed at the NCAT Test Track to be adequate for a minimum of 10 million ESALs and likely much longer. This report is an interim report in that the test sections are still being trafficked. A final report will be prepared upon the completion of testing.

FINAL REPORT

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INTRODUCTION

Pavement recycling includes a series of technologies that can be used to rehabilitate distressed pavements or construct new pavements while reducing costs, construction time, and environmental impacts. For rehabilitation projects, in-place pavement recycling remixes the in situ or stockpiled pavement material and reuses it in the final pavement structure. For new construction projects, pavement recycling can reuse materials from existing stockpiles of reclaimed asphalt pavement (RAP) to build new pavements. Benefits of recycling techniques include reduced use of virgin materials, reduced fuel consumption, reduced time of lane closures, reduced emissions related to construction (Nataatmadja, 2001; Stroup-Gardiner, 2011; Thenoux et al., 2007), and large cost savings (Bemanian et al., 2006) that allow highway agencies to preserve their pavement networks better.

Despite the successful experiences of many highway agencies (Bemanian et al., 2006; Berthelot et al., 2007; Diefenderfer and Apeagyei, 2011a; Diefenderfer and Apeagyei, 2011b; Guthrie et al., 2007; Diefenderfer and Bowers, 2015; Hilbrich and Scullion, 2008; Lane and Kazmierowski, 2005; Lewis et al., 2006; Mallick et al., 2002; Maurer et al., 2007; Mohammad et al., 2003; Romanoschi et al., 2004; Saleh, 2004; Wen et al., 2004), pavement recycling techniques are still not widely employed in the United States. There are many reasons for this; a few reasons include the lack of long-term performance data from which the expected service life can be derived and a lack of understanding of the failure mechanisms of recycled pavement

structures. In addition, no distress prediction models exist for transforming the measured and/or predicted pavement responses into distress quantities within a mechanistic-empirical pavement design framework.

In an effort to address some of these deficiencies, the Virginia Department of Transportation (VDOT) contracted with the National Center for Asphalt Technology (NCAT) in 2012 to construct, instrument, and traffic three pavement test sections at the NCAT Test Track during the facility's 2012-2014 track cycle. The three test sections, having a length of 200 ft each, included two different asphalt overlay thicknesses placed on top of a recycled base. One of the three sections also contained a cement-stabilized base designed to simulate a full-depth reclaimed layer. The three test sections were trafficked with heavily loaded trucks for 2 years until approximately 10 million equivalent single axle loads (ESALs) were applied. In addition, embedded instruments were installed during construction to quantify the pavement response from truck loading. The designs of the track sections was similar to those of the pavement recycling project completed in 2011 on I-81 in Virginia (Diefenderfer and Apeagyei, 2014) and were used to study the performance of several design alternatives.

Specifically, the three test sections on the NCAT Test Track sponsored by VDOT, designated Sections N3, N4, and S12, included cold central-plant recycling (CCPR), cement stabilization of the aggregate and top of subgrade layers to simulate full-depth reclamation (FDR), and two asphalt surface mixture thicknesses. The CCPR materials were produced using foamed asphalt (produced using a performance grade [PG] 67-22 asphalt binder) as the recycling agent at a dosage rate of 2.0% and hydraulic cement as a chemical additive at a dosage rate of 1.0%. The FDR materials were produced using hydraulic cement as the stabilizing agent at a dosage rate of 4.0%. Construction occurred during July and August 2012 and trafficking began in October 2012; this report covers the results of trucking operations from October 2012 until October 2014.

Originally, the researchers considered using cold in-place recycling (CIR) to construct the recycled base on the track sections. However, the 200-ft length of each section was considered too short to produce a consistent material and thus CCPR was determined to be the most appropriate recycling technique. This decision led to one of the first studies of CCPR performance using instrumented pavement sections under full-scale accelerated loading.

PURPOSE AND SCOPE

The purpose of this study was to examine the structural performance of the three CCPR pavement sections described previously built at the NCAT Test Track to determine if the different cross sections exhibit different performance. The test sections were subjected to trafficking of approximately 10 million ESALs over 2 years. The performance of the sections was assessed by analyzing the results of field and laboratory testing.

The scope of this study consisted of the three pavement sections at the NCAT Test Track constructed during the 2012 track cycle: two used CCPR plus an aggregate base (having two

different asphalt surface thicknesses), and one used CCPR in conjunction with a cement-stabilized foundation simulating FDR. The three test sections each had a length of 200 ft. Laboratory testing conducted on laboratory-produced specimens fabricated from recycled asphalt materials produced during construction and on cores collected after construction included binder content, gradation, and dynamic modulus. Field-based testing included deflection measurements from a falling weight deflectometer (FWD); temperature, pressure, and strain measurements from embedded instruments; and surface-observable condition. The surface-observable condition was assessed by measuring the rut depth, ride quality (in terms of the International Roughness Index [IRI]), and surface cracking over the 2-year test cycle.

METHODS

Pre-Construction and Construction Summary

The pre-construction mix design and construction processes were summarized by NCAT and VDOT staff who were present during the test section construction in 2012. Details of construction activities, observations, and quality assurance tests follow.

Laboratory Performance Evaluation

Gradation and Binder Content

The binder content and gradation were measured on CCPR materials used during construction. The binder content was measured in accordance with AASHTO T 308-10, Standard Method of Test for Determining the Asphalt Binder Content of Hot-Mix Asphalt (HMA) by the Ignition Method (American Association of State Highway and Transportation Officials [AASHTO], 2013). Gradation analysis was conducted in accordance with AASHTO T 27-11, Standard Method of Test for Sieve Analysis of Fine and Coarse Aggregates (AASHTO, 2013). The gradation was determined on residual materials from the ignition oven test wherein all binder was removed.

Dynamic Modulus (|E*|) Specimen Fabrication and Testing

Dynamic modulus specimens were fabricated from loose materials collected at the mobile plant during construction of the test sections. As the moisture content at production is an important parameter for compaction, the sampled materials were placed in buckets and then sealed for transport to the on-site laboratory. The loose materials were then combined to create one approximately 25-kg batch. About 16 specimens were fabricated per batch. When additional materials were needed, additional buckets were unsealed and combined to create another approximately 25-kg batch.

Dynamic modulus test specimens were fabricated using a gyratory compactor such that the test specimens had approximately the same bulk density (approximately 135 to 137 lb/ft^3) as

the specimens used for strength testing during mixture design. For each dynamic modulus specimen, a calculated mass of material was added to a 150-mm-diameter gyratory compactor mold and compacted to a desired height of about 170 mm. The mass of material for each dynamic modulus specimen was calculated by multiplying the mass for an indirect tensile strength (ITS) specimen from the mix design process by the proportional change in volume between the mix design ITS specimen and the dynamic modulus specimen. The number of gyrations needed to fabricate the dynamic modulus specimen was not recorded but typically ranged from 30 to 50 gyrations. All fabricated specimens were cured in a forced draft oven for 72 hr at 104°F after compaction.

The fabricated specimens were stored at approximately 70°F and 70% relative humidity for approximately 6 months prior to testing. The long time between fabrication and testing was the result of a backlog of other testing in the laboratory rather than any predetermined curing or waiting period. Prior to testing, the fabricated specimens were cored and trimmed to produce test specimens having a 4-in diameter and a 6-in height. The dynamic modulus testing was conducted using an asphalt mixture performance tester in accordance with AASHTO TP 79, Standard Method of Test for Determining the Dynamic Modulus and Flow Number for Asphalt Mixtures Using the Asphalt Mixture Performance Tester (AMPT) (AASHTO, 2013) with slight modifications; a reduced set of temperatures was used. Testing was conducted in the axial mode at temperatures of 4.4, 21.1, 37.8, and 54.4 °C. At each temperature, testing was conducted at loading frequencies of 25, 10, 5, 1, 0.5, and 0.1 Hz. For each condition, five replicates were tested. Each specimen was re-used throughout the testing regime and tested in increasing order of temperature and a decreasing order of frequency at each temperature to minimize the potential for earlier tests to influence later ones negatively through permanent sample deformation.

Instrumentation

Instruments to measure pavement temperature, strain, and pressure were placed in the three test pavement sections during construction. In general, pavement engineers are interested in measuring the response of a pavement from traffic loads and environmental conditions at certain locations within the pavement structure. The temperature was measured at the top of the surface layer, at the mid-depth of the combined thickness of the CCPR and asphalt layers, and at the bottom of the CCPR layer. In addition, since the bending strain is highest at the bottom of the asphalt layers, strain gauges were included at this location. Prior to the start of the project, there was some discussion as to whether the maximum bending strain would occur at the bottom of the asphalt overlay or the bottom of the CCPR layer. Ultimately, the bottom of the CCPR layer was selected for placing the strain gauges. Pressure cells were used to measure the pressure at the top of the aggregate layer and at the top of the subgrade. Additional details on the instrumentation used at the NCAT Test Track are provided by Timm (2009).

Field Performance Evaluation

Rut Depth and Ride Quality

Rutting and ride quality data were simultaneously collected on a weekly basis with vehicle-mounted sensors on an inertial profiler operated by NCAT staff. Data were collected in accordance with ASTM E950-09, Standard Test Method for Measuring the Longitudinal Profile of Traveled Surfaces with an Accelerometer Established Inertial Profiling Reference (ASTM International [ASTM], 2013); AASHTO R 43-07, Standard Practice for Determination of International Roughness Index (IRI) to Quantify Roughness of Pavements (AASHTO, 2013); and AASHTO R 48-10, Standard Practice for Determining Maximum Rut Depth in Asphalt Pavements (AASHTO, 2013). Since the test sections were only 200 ft in length, the data were reported as average values over the length of the entire 200-ft section.

Structural Capacity

Deflection testing to assess structural capacity was performed on a weekly basis by NCAT staff using a Dynatest Model 8000 FWD in accordance with ASTM D4694-09, Standard Test Method for Deflections with a Falling-Weight-Type Impulse Load Device (ASTM, 2013). Testing was conducted at four locations within each test section (one of which coincides with the location of the instrumentation). The FWD was equipped with nine sensors at radial distances of 0, 8, 12, 18, 24, 36, 48, 60, and 72 in from the center of a load plate. Deflection testing was conducted at four load levels (6,000; 9,000; 12,000; and 16,000 lbf). Following two unrecorded seating drops, three deflection basins were recorded at each load level.

The deflection data were analyzed in accordance with the 1993 AASHTO *Guide for Design of Pavement Structures* (AASHTO, 1993). The results of the analysis included calculating the combined moduli of the asphalt/CCPR layers and calculating the structural layer coefficient for the CCPR in each section.

RESULTS AND DISCUSSION

Pre-Construction Summary

Mix Design

Prior to construction, a mix design for the CCPR material was completed at the laboratory at the NCAT Test Track by staff of the Virginia Transportation Research Council (VTRC) to find the optimum moisture content, the density at optimum moisture content, and the optimum recycling agent content for the CCPR material. Despite the materials being stockpiled for nearly 1 year and undergoing processing to break up large clumps that had formed from environmental exposure, the mix design parameters for the materials placed at the NCAT Test Track were similar to those for the materials placed on the I-81 project. Although it is typical to find PG 64-22 binder in Virginia, PG 67-22 binder is more common in the southeast United

States. Thus, several sources of PG 67-22 binder were tested to determine their foaming properties until one could be found to meet the desired half-life and expansion ratio (defined by Wirtgen GmbH [2010] as having a minimum expansion ratio of 11 and a half-life of 8 sec). Since multiple binders had to be tested, it was surmised that the rejected PG 67-22 binders might have been polymer modified; modification is known to produce binder that is inadequate for foaming (Wirtgen GmbH, 2010). The binders were never tested for the presence of polymer modifiers since a binder source having suitable foaming properties was identified.

From the mix design process, the mix design parameters were determined to be 2% foamed asphalt using PG 67-22 binder and 1% Type II portland cement as a chemical additive in accordance with AASHTO M85, Standard Specification for Portland Cement (Chemical and Physical) (AASHTO, 2013). A laboratory-scale pug mill was used to mix the foamed asphalt with the recycled asphalt pavement (RAP) materials to determine the mix design parameters in accordance with AASHTO T 180-10, Standard Method of Test for Moisture-Density Relations of Soils Using a 10-lb (4.4-kg) Rammer and an 18-in. (457-mm) Drop, Method D (AASHTO, 2013). ITS was also tested in accordance with AASHTO T 283, Standard Method of Test for Resistance of Compacted Hot-Mix Asphalt (HMA) to Moisture-Induced Damage (AASHTO, 2013). The average wet and dry bulk densities at the stated foamed asphalt and cement contents were 133.0 and 127.0 lb/ft^3, respectively; the average ITS value was 83 psi with a retained strength ratio of 76%.

A mix design for the cement-stabilized materials was performed by a third party assuming typical FDR construction practices. The only difference between the cement-stabilized process used at the NCAT track and an FDR process by definition is the presence of some existing bound asphalt pavement. Prior to sampling for the mix design, the asphalt overlay at the NCAT Test Track was removed for all sections. In hindsight, it would have been more technically correct to have included a thin layer of existing asphalt pavement and to have incorporated it into the blended materials. Because of this exception, the resultant mixture used at the track is described herein as a *cement-stabilized layer* simulating FDR. At the time of the mix design, the exact proportions of existing aggregate base and subgrade were unknown, so multiple mix designs assuming two proportions were completed: 1/3 subgrade with 2/3 aggregate base and 2/3 subgrade with 1/3 aggregate base. During construction of Section S12, the existing 6-in-thick aggregate layer was blended with approximately 2 in of the underlying subgrade. The tests used during the mix design were completed in accordance with ASTM D1633, Standard Test Method for Compressive Strength of Molded Soil-Cement Cylinders (ASTM, 2013) and AASHTO T 134, Method B, Standard Method of Test for Moisture-Density Relations of Soil-Cement Mixtures (AASHTO, 2013). The optimum Type II portland cement content was 4%, and the mixture had an average compressive strength 256 psi after 7 days, a maximum dry density of 130.0 lb/ft^3, and an optimum moisture content of 8.0%. A compressive strength of 350 psi after 7 days was used as a maximum limit during the design process.

Construction Description

The three sections constructed at the NCAT Test Track were designated Sections N3, N4, and S12, wherein the letter denotes the segment of the track according to cardinal directions and

the number denotes the sequential section number (e.g., N4 is the fourth section located on the North tangent) as shown in Figure 1. Construction of the three test sections occurred during July and August 2012. The cement-stabilized base in Section S12 was constructed in late July 2012 and cured for approximately 3 weeks before the CCPR layer was placed. Prior to construction of the CCPR layers, the existing asphalt and aggregate base in Sections N3 and N4 were removed. The aggregate layer was then replaced, with different thicknesses for the two sections so the final driving surface would be at the same elevation. The CCPR layer for all three sections was produced and placed during a single day in early August 2012.

Each of the three test sections featured a stone-matrix asphalt (SMA) surface layer and one or more dense-graded asphalt mixture layers above the CCPR layer. The SMA surface layer had a nominal maximum aggregate size (NMAS) of 12.5 mm and used a PG 76-22 binder. The second layer was a dense-graded intermediate mixture having a 19.0 mm NMAS and used a PG 67-22 binder. The CCPR was 100% RAP with 2% foamed PG 67-22 asphalt binder and 1% Type II portland cement.

Sections N3 and N4 were constructed on top of a crushed granite aggregate base layer, and S12 was built on the cement-stabilized base layer. All three sections were constructed on the same native subgrade, which is classified as an A-4(0) soil and described further by Taylor and Timm (2009). Sections N3 and N4 were designed to have 6 in and 4 in of asphalt materials over 5 in of CCPR, respectively. For Sections N3 and N4, the CCPR layer was placed on a 6-in-thick aggregate base. During construction of the CCPR layer, the CCPR material was produced at the mobile plant (shown in Figure 2) located on-site and hauled to the track using dump trucks. The trucks transferred the CCPR material directly into a conventional asphalt-paver hopper as shown in Figure 3. The CCPR was compacted using a 13-ton double-drum vibratory roller while the density was monitored by using a nuclear density gauge operated in backscatter mode. The CCPR material was paved at a depth of approximately 6 in and later profile milled to the desired thickness. The average measured dry density of the CCPR materials (measured using a nuclear density gauge in direct transmission mode the day after construction) was 130.4, 128.8, and 130.7 lb/ft^3 for Sections N3, N4, and S12, respectively. Based on the mix design optimum dry density of 127.0 lb/ft^3, the average density achieved was 102.7%, 101.5%, and 102.9% of design for Sections N3, N4, and S12, respectively. To calculate the dry density, the moisture content was measured during construction and removed from the readings. There will be a small error in moisture content measurement the day after construction as the mat continues to dry, thus making the density measurements conservative.

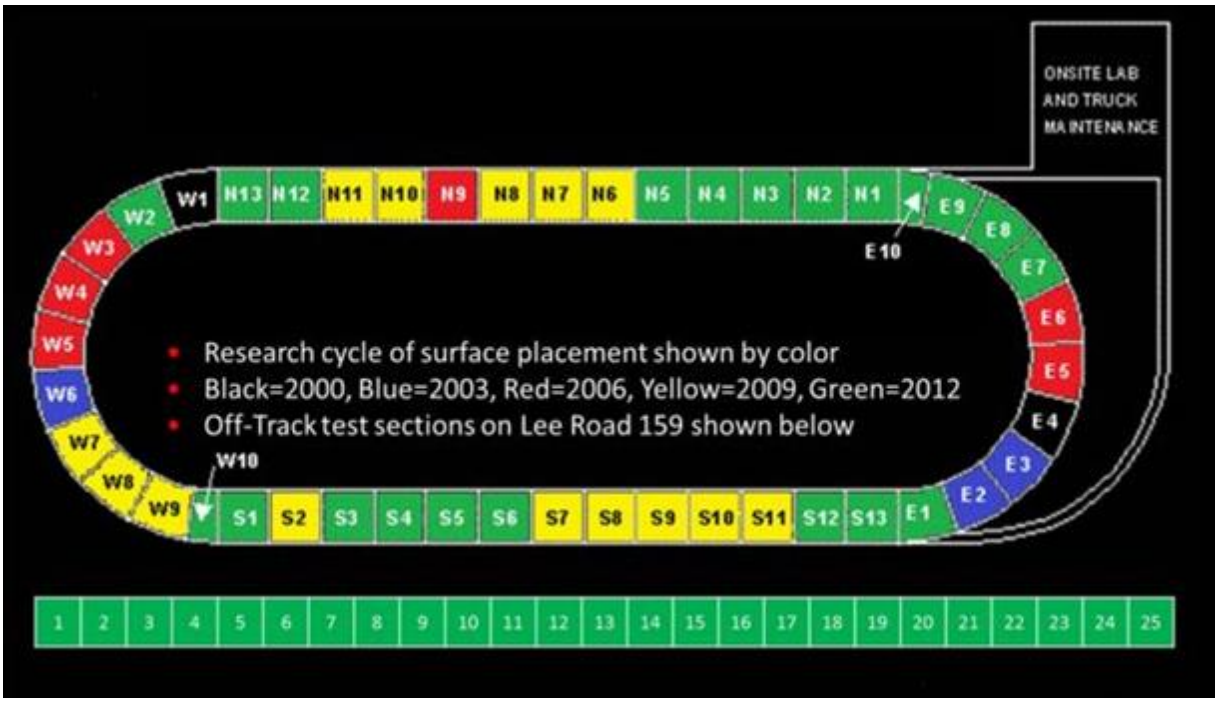

Figure 1. NCAT Test Track Diagram (VDOT Sections Are N3, N4, and S12). From PaveTrack.com.

Figure 2. Cold Central Plant Recycling Plant at NCAT Test Track

Figure 3. Transferring Cold Central Plant Recycled Material to an Asphalt Paver

Section S12 was designed to have 4 in of asphalt materials over 8 in of cement-stabilized base. The base stabilization was performed in place using a reclaimer to simulate FDR. To construct the stabilized base layer, approximately 6 in of crushed granite aggregate base and 2 in of the subgrade were reclaimed using 4% (by weight) Type II portland cement. During construction, a calibrated distributer truck was used to apply the proper amount of cement to the surface of the section. Following this, the reclaimer was used to pulverize the existing aggregate layer and a portion of the subgrade and mix the cement as shown in Figure 4. Compaction of the cement-stabilized material was accomplished using a 6-ton vibratory soil compactor with a padfoot drum. The compacted material was shaped using a motor grader (shown in Figure 5) and then recompacted. Density was monitored during construction using a nuclear density gauge operated in direct transmission mode.

Figure 4. Full-Depth Reclamation Process in Section S12

Figure 5. Shaping Cement-Stabilized Layer Using a Motor Grader

Figure 6 shows the average as-built thickness of each test section and reflects the natural variation due to standard construction practices at the NCAT Test Track. The average thickness represents measurements taken at 12 different locations within each section. From this it can be seen that Sections N3 and N4 can be used to determine the difference in performance of a CCPR base with two different asphalt overlay thicknesses. In addition, Sections N4 and S12 can be used to determine the difference in performance between sections constructed with a 6-in-thick aggregate base versus an 8-in-thick cement-stabilized base layer since both have the same 4-in asphalt overlay.

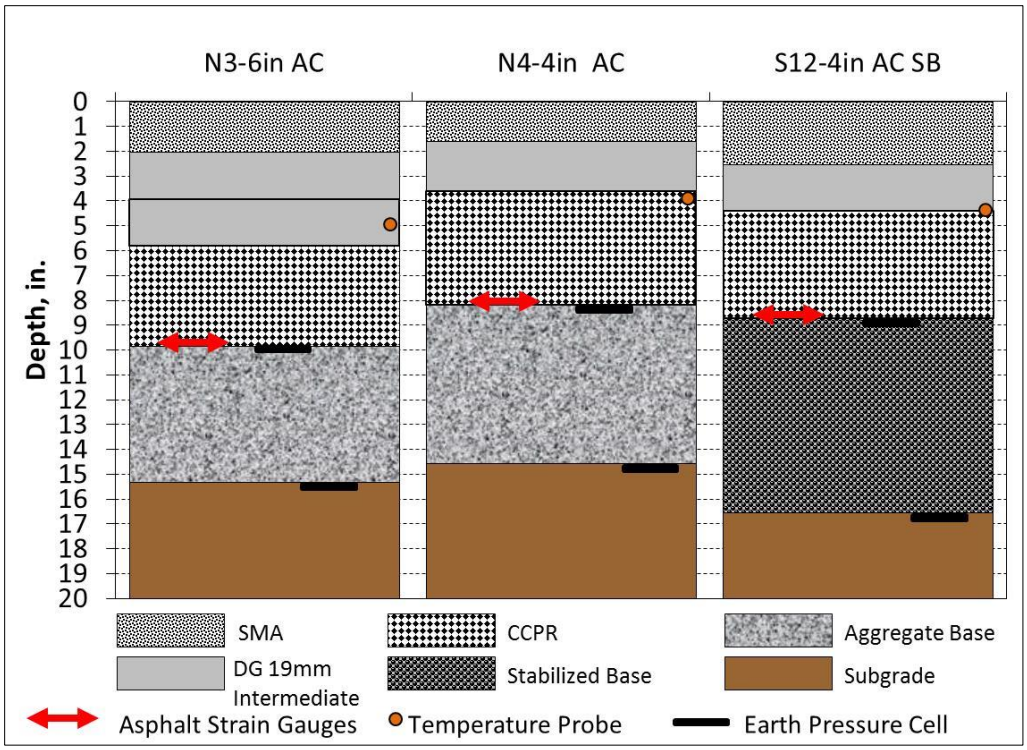

Figure 6. Schematic of VDOT Sections at NCAT Test Track Showing Average As-Built Thicknesses and Depth of Instrumentation. AC = asphalt concrete, SB = stabilized base, SMA = stone matrix asphalt; DG = dense graded; CCPR = cold central plant recycling.

Tables 1 and 2 show the as-built construction properties of the materials placed within Sections N3, N4, and S12. Table 1 shows that the SMA surface mixture had binder content ranging from 6.0 to 6.1%, air void content ranging from 4.2% to 4.7%, and RAP content of 12.5%. The dense-graded 19.0 mm intermediate mixture had binder content ranging from 4.4% to 4.6%, air void content ranging from 6.4% to 7.4%, and RAP content of 30%. When the RAP content of the asphalt mixtures and the recycled content of the CCPR and stabilized base layers are considered, the pavement structure in Section S12 contains approximately 80% recycled material. Table 2 shows that the constructed dry density of the CCPR layer was greater than 100% of the dry density control value determined during the mix design.

Figure 6 also shows the location of instrumentation used in this study. Six horizontal asphalt strain gauges, oriented longitudinally (parallel to traffic), were placed at the bottom of the CCPR layer to capture the bending of the asphalt-bound layers. Earth pressure cells, placed at the top of the aggregate base and top of the subgrade, measured vertical pressures transmitted through the sections. Temperature probes were installed after paving at the middle of the composite asphalt mixture and CCPR layers to measure mid-depth temperature during trafficking. Prior to construction, there was some debate as to where to place the horizontal asphalt strain gauges to capture the location of maximum bending strain. Given previous testing of recycled materials, it was determined that the maximum bending strain would likely be at the bottom of the CCPR layer rather than at the bottom of the asphalt mixture. Field performance testing during this study would prove this to be the correct location.

Table 1. VDOT Experiment As-Built Layer Properties

Layer No.	Parameter	Section N3 (6-in overlay with aggregate base)	Section N4 (4-in overlay with aggregate base)	Section S12 (4-in overlay on stabilized base)
1	Description	2-in 12.5 mm NMAS SMA with 12.5% RAP and PG 76-22 binder		
	Binder Content, %	6.1	6.0	6.1
	Air Voids, %	4.3	4.7	4.2
	Thickness, in	2.0	1.6	2.6
2	Description	2-in 19 mm NMAS Superpave with 30% RAP and PG 67-22 binder		
	Binder Content, %	4.6	4.6	4.7
	Air Voids, %	7.1	7.4	6.7
	Thickness, in	1.9	2.0	1.8
3	Description	2-in 19 mm NMAS dense-graded with 30% RAP and PG 67-22 binder	N/A	N/A
	Binder Content, %	4.4		
	Air Voids, %	6.4		
	Thickness, in	1.9		
4	Description	CCPR: 100% RAP with 2% foamed PG 67-22 binder and 1% Type II portland cement		
	Thickness, in	4.0	4.6	4.3
5	Description	Crushed granite aggregate base (CGAB)		6-in aggregate base + 2-in subgrade stabilized in-place with 4% Type II portland cement
	Thickness, in	5.5	6.4	7.8
6	Description	Subgrade: AASHTO A-4(0) Soil		

NMAS = nominal maximum aggregate size; SMA = stone matrix asphalt; RAP = reclaimed asphalt pavement; CCPR = cold central plant recycling.

Table 2. VDOT Experiment As-Built CCPR Density

	Section N3		Section N4		Section S12	
Parameter	Dry Density, lb/ft^3	Dry Density, % of mix design[a]	Dry Density, lb/ft^3	Dry Density, % of mix design[a]	Dry Density, lb/ft^3	Dry Density, % of mix design[a]
Average	130.4	102.7	128.8	101.5	130.7	102.9
Standard Deviation	1.0	0.8	1.3	1.0	2.1	1.7
Minimum	129.1	101.6	126.5	99.6	127.0	100.0
Maximum	131.6	103.7	130.2	102.5	133.5	105.1

CCPR = cold central plant recycling.
[a] 127 lb/ft^3.

Laboratory Performance Evaluation

Gradation and Binder Content

The binder content and gradation were measured on materials collected during construction. The average measured binder content of the source RAP materials was 5.77%. The average measured binder content of the stabilized CCPR mixture was 7.73%. During the CCPR production, 2% foamed asphalt was added to the source RAP materials, which accounts for the difference between the source RAP and the CCPR mixture binder contents.

Table 3 shows the gradation of the CCPR materials. According to Wirtgen GmbH (2010), aggregate for cold recycling should pass certain gradation requirements, the most critical being the percent passing the 0.075 mm sieve. The materials passing the 0.075 mm sieve are necessary for dispersion of the tiny foam droplets that are critical for binding the foamed material. Wirtgen GmbH (2010) recommended that 2% to 9% of material pass the 0.075 mm sieve. From Table 3 it can be seen that the NMAS was 12.5 mm and nearly 9% passed the 0.075 mm sieve.

Table 3. Gradation of RAP Source Material

Sieve Size, mm	% passing
25	100
19	98.1
12.5	87.2
9.5	77.0
4.75	54.7
2.36	39.6
1.18	29.5
0.6	23.6
0.30	17.4
0.150	12.9
0.075	8.7

RAP = reclaimed asphalt pavement.

Dynamic Modulus (|E*|) Specimen Fabrication and Testing

The dynamic modulus of a laboratory -produced CCPR mixture fabricated from materials produced during construction was measured; the test was conducted on five replicates. Figure 7 shows the results of the dynamic modulus testing at the four test temperatures and at frequencies of 10, 1, and 0.1 Hz (the results at 25, 5, and 0.5 Hz are not shown for brevity but followed the same trend). The error bars indicate ±1 standard deviation. By visual inspection of the data in Figure 7 it can be seen that the dynamic modulus of the CCPR materials decreased with increasing temperature at each test frequency and increased with increasing test frequency at each temperature. These trends are similar for asphalt materials. Similar observations regarding the influence of temperature for recycled materials have been reported by others (Cross and Jakatimath, 2007; Diefenderfer and Link, 2014; Lee and Kim, 2007; Schwartz and Khosravifar, 2013). These studies also documented stiffness values for recycled materials similar to those shown herein.

Figure 7 also shows the between-specimen repeatability as error bars denoting ±1 standard deviation. The dynamic modulus coefficient of variation (COV) calculated from the five replicates for all tests ranged from approximately 5% to 25% with generally larger COVs at the higher test temperatures and lower test frequencies. AASHTO TP 79 (AASHTO, 2013) allows a COV of 5.8% for five replicates (the maximum is 9.2% for two replicates); however, this specification is for asphalt materials and not recycled materials. The higher-than-allowable COV was expected, based on the researchers' experiences and the literature (Cross and Jakatimath, 2007; Schwartz and Khosravifar, 2013), and suggests that revisions to tolerances specified in AASHTO TP 79 (AASHTO, 2013) are needed when recycled materials are evaluated.

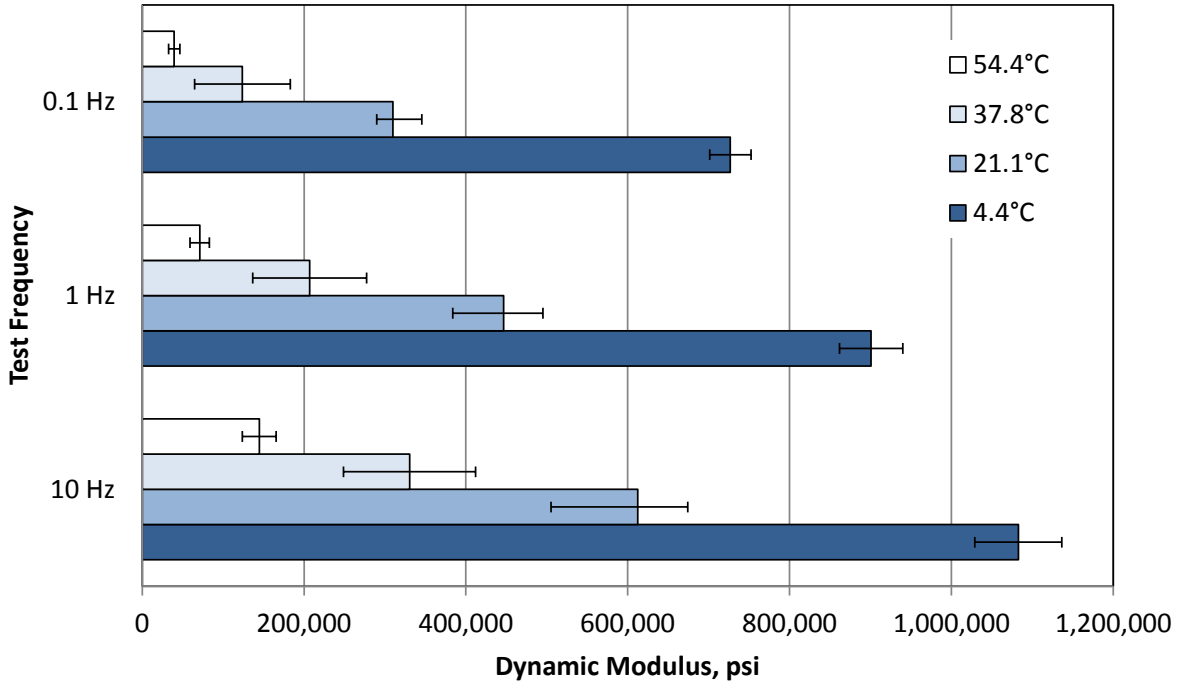

Figure 7. Results of Dynamic Modulus Testing at Three Frequencies with Respect to Test Temperature. Error bars represent ±1 standard deviation.

Field Performance Evaluation

An assessment of the performance of the three test sections was conducted periodically during the 2-year track cycle. During this time, approximately 10 million ESALs were applied to the pavements by a fleet of trucks pulling loaded trailers. Each truck pulled multiple trailers to increase the number of load repetitions; however, each trailer axle was loaded only to the maximum permissible by federal interstate standards.

Rut Depth and Ride Quality

Rut depth and ride quality measurements were simultaneously collected using an inertial profiler during the 2-year test cycle. Figure 8 shows the results of the rut depth measurements. Each data point represents the average value within the section. From Figure 8 it can be seen that the maximum rut depth measured was less than approximately 0.3 in. The data also show three instances where the rut depth for each section increased in the late spring and summer months, which correspond to times when the pavement temperatures were highest and thus the pavement stiffness was lowest. In general, the rut depth measured in Section N4 (4-in overlay with non-stabilized aggregate base) was slightly greater than the rut depth measured in Sections N3 (6-in overlay with non-stabilized aggregate base) and S12 (4-in overlay on a stabilized base), but the differences between the sections would not be considered practically significant. The distribution of the rutting within the various layers of the pavement structure was not identified.

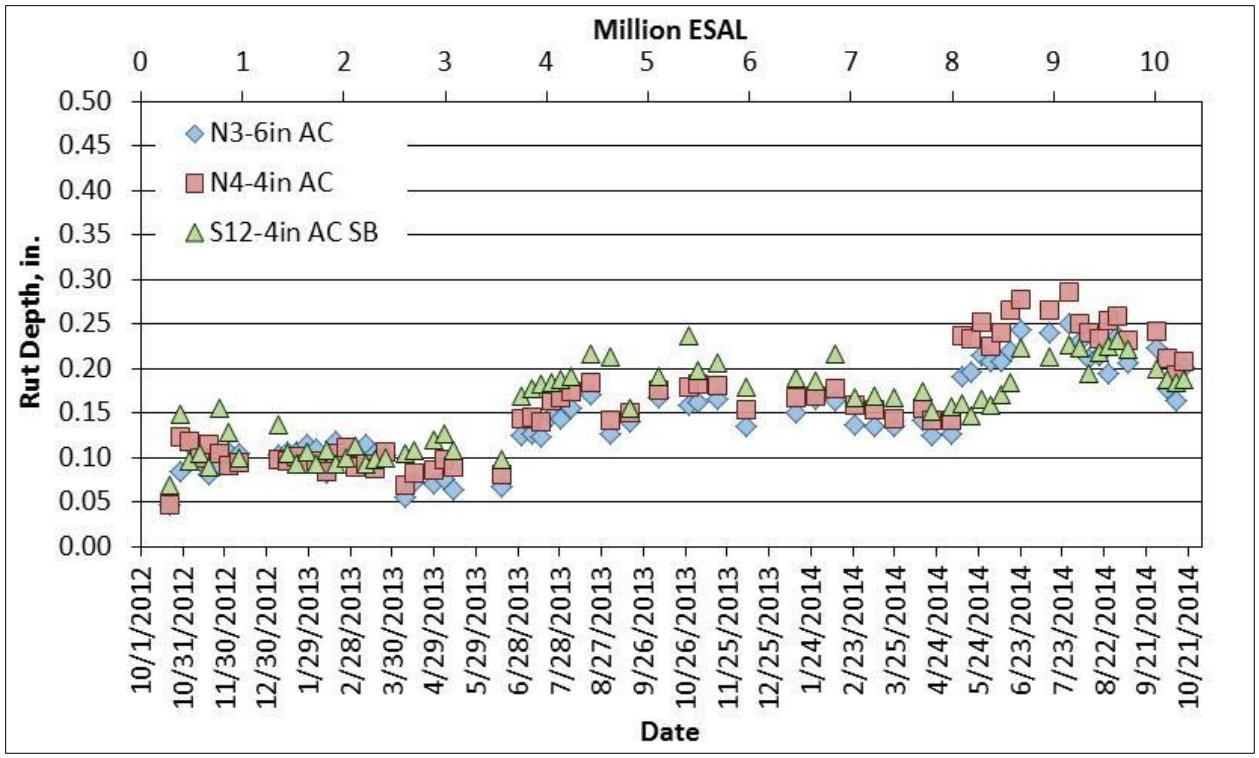

Figure 8. Rut Depth Measurements. ESAL = equivalent single axle loads.

Figure 9 shows the results of ride quality measurements, expressed as the IRI for the three test sections. Each data point represents the average value within the section. From Figure 9 it can be seen that the ride quality remained relatively steady throughout the entire 2-year test. This shows that the three pavement sections did not undergo significant deterioration that would affect the ride quality as measured at the surface of the pavement. In addition, it can be seen that Section S12 (4-in overlay on a stabilized base) had a much higher average IRI value than the other two sections (having a non-stabilized aggregate base). The higher IRI value for Section S12 was influenced by a localized area located approximately 50 ft into the section (if this section is excluded, the average IRI value for Section S12 would be approximately 120 in/mi). If these test sections were actually located on the interstate or primary network, VDOT would classify the ride quality as "good" for Sections N3 and N4 and "fair" for Section S12 (VDOT, 2014).

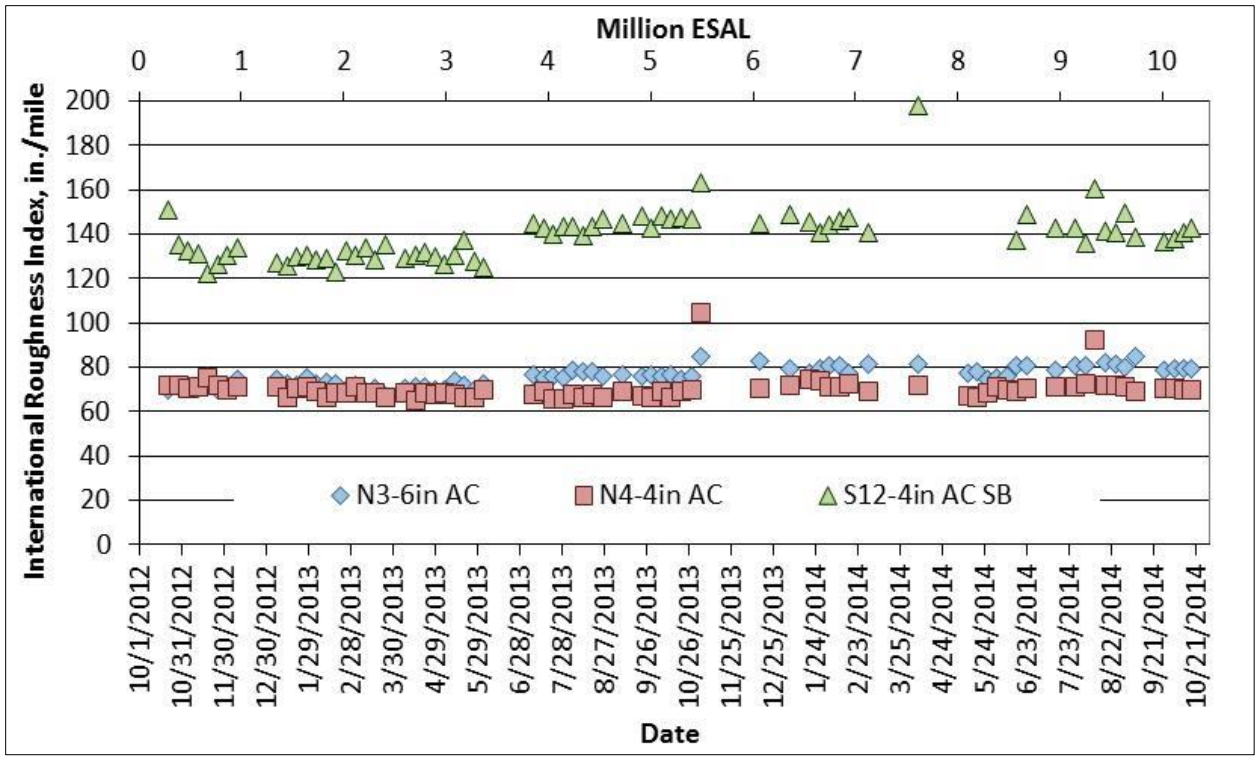

Figure 9. Ride Quality Measurements. ESAL = equivalent single axle loads.

Visually Observed Cracking

Throughout the 2-year test cycle, no cracks were observed at the pavement surface in any of the three test sections.

Instrumentation Response

Instruments were installed in the three test sections to monitor the pavement temperature and response (specifically, the pressure and strain) from truck loading. These data are presented with respect to temperature and time (at a normalized temperature of 68°F).

Tensile Strain Measurements

Figure 10 shows the tensile strain response from trucking operations at the bottom of the CCPR layer with respect to temperature. The strain response was strongly correlated to temperature as shown by the exponential regression equations in the figure. Sections N3 (6-in overlay with non-stabilized aggregate base) and N4 (4-in overlay with non-stabilized aggregate base) exhibited similar temperature sensitivity as demonstrated by the similar exponential coefficients in their respective regression equations.

Figure 10 also shows that Section S12 (4-in overlay on a stabilized base) underwent relatively less strain than the other sections and demonstrated less sensitivity to temperature. The exponential regression coefficient for Section S12 was approximately one-half that of the other sections. The strain magnitude was also much lower in Section S12 than in Section N3 (6-

in overlay with non-stabilized aggregate base) or N4 (4-in overlay with non-stabilized aggregate base) but increased with increasing temperatures. This is the combined effect of the lower temperature sensitivity and greater stiffness of the cement-treated base since the measured tensile strain is also a function of the underlying, supporting layer. The cement-stabilized base layer used in Section S12 was seen to limit the tensile strain in the CCPR layer as compared to the non-stabilized aggregate base layer used in Sections N3 and N4.

The benefit of the additional 2 in of asphalt mixture in Section N3 as compared to Section N4 was seen across the temperature spectrum in that the strain in Section N3 was approximately 40% less than the strain in Section N4 at 68°F. If the concept of fatigue cracking holds true for CCPR materials in the same way it does for dense-graded asphalt mixtures, the reduced strain seen in Section N3 should result in a longer fatigue life for this design.

The benefit of including the stabilized base in Section S12 (4-in overlay) as compared to the aggregate base in Section N4 (4-in overlay) was also seen across the temperature spectrum. At 68°F, the strain in Section S12 was approximately 70% less than the strain in Section N4; the difference increased with increasing temperature. In addition, the stabilized base in Section S12 was beneficial in that the strain in Section S12 was approximately 50% less than the strain in Section N3 (6-in overlay) at 68°F. This may be surprising given that Section N3 has a 6-in asphalt overlay and Section S12 has a 4-in asphalt overlay. The strain values for Section S12 were within the ranges suggested in the literature that denote a long-life pavement structure (i.e., a pavement that performs over a long service life with no structural rehabilitation) as defined by Tran et al. (2015). However, it is unclear if the concept of reduced strains equaling long-term performance is transferable to a recycled pavement in the same way that it is for a structure built using traditional dense-graded asphalt mixtures.

Figure 11 shows the temperature-normalized horizontal strain at the bottom of the CCPR layer, corrected to a reference temperature of 68°F. Figure 11 also shows linear trend lines developed for each data set. The strain in Sections N3 (6-in overlay with non-stabilized aggregate base) and N4 (4-in overlay with non-stabilized aggregate base) increased over time. The slope of the linear regression developed for Section N3 indicated relatively little change in strain over time. The greater slope and corresponding higher R^2 for the linear regression developed for Section N4 may indicate that some damage occurred within this section that was not yet observable at the pavement surface. The much lesser slope for the linear regression developed for Section S12 (4-in overlay on a stabilized base) indicated that there was little change in the strain over time and that there was likely no internal damage. In addition, the negative slope indicated that the section is continuing to undergo a gain in stiffness (a similar finding was identified by Diefenderfer and Apeagyei, 2011a, and Diefenderfer et al., 2012).

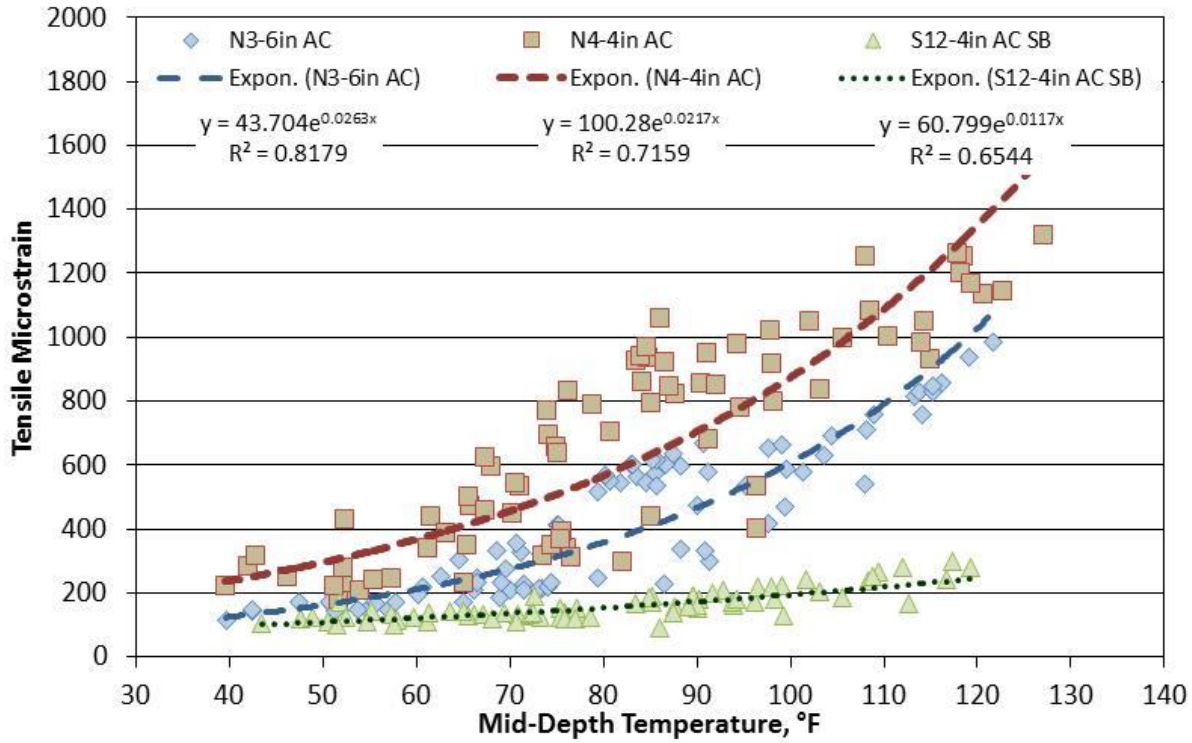

Figure 10. Tensile Strain at Bottom of CCPR Layer vs. Temperature. CCPR = cold central plant recycling.

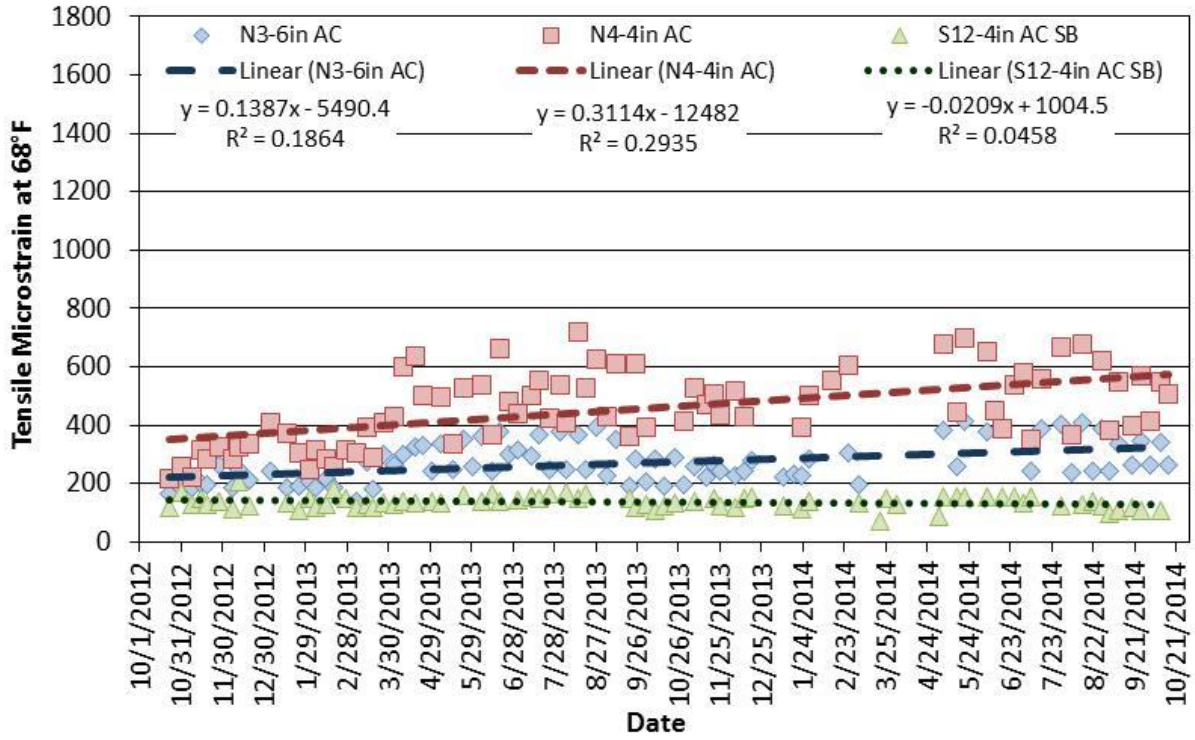

Figure 11. Tensile Strain at Bottom of CCPR Layer (Normalized to 68°F) vs. Time. CCPR = cold central plant recycling.

The tensile strain values shown in Figures 10 and 11 can be compared with fatigue endurance limit (FEL) strain values reported in the literature. If a pavement is designed such that the horizontal tensile strain at the bottom of the asphalt layers is kept below the FEL, no bottom-up fatigue cracking should be expected. Tran et al. (2015) stated that a summary of the literature showed that FEL values ranged from early conservative estimates of 70 microstrains to more recent estimates of 200 microstrains. Based on the results of field testing, Tran et al. (2015) suggested that a distribution of limiting strain be used as a design criterion rather than a single threshold value; the suggested distribution showed that 50% of the tensile strains at the bottom of the asphalt layer should be less than 100 microstrains and 90% should be less than 221 microstrains. From Figure 10 it appears that the strain values from Sections N3 and N4 are greater than the suggested FEL distribution but the strain values from Section S12 are close to the suggested distribution. It is not known if the FEL concept holds true for recycled materials such as CCPR.

Vertical Base Pressure

Figure 12 presents the vertical pressure created during trucking operations at the top of the base layer with respect to temperature along with the developed exponential regression equations for each section. Sections N3 (6-in overlay with non-stabilized aggregate base) and N4 (4-in overlay with non-stabilized aggregate base) show similar behavior and a strong correlation with temperature. However, the expected reduction in base pressure from the additional asphalt layer thickness used in Section N3 was not observed. Section S12 (4-in overlay on a stabilized base), however, shows a lesser base pressure, presumably from the higher modulus of the materials within the pavement structure.

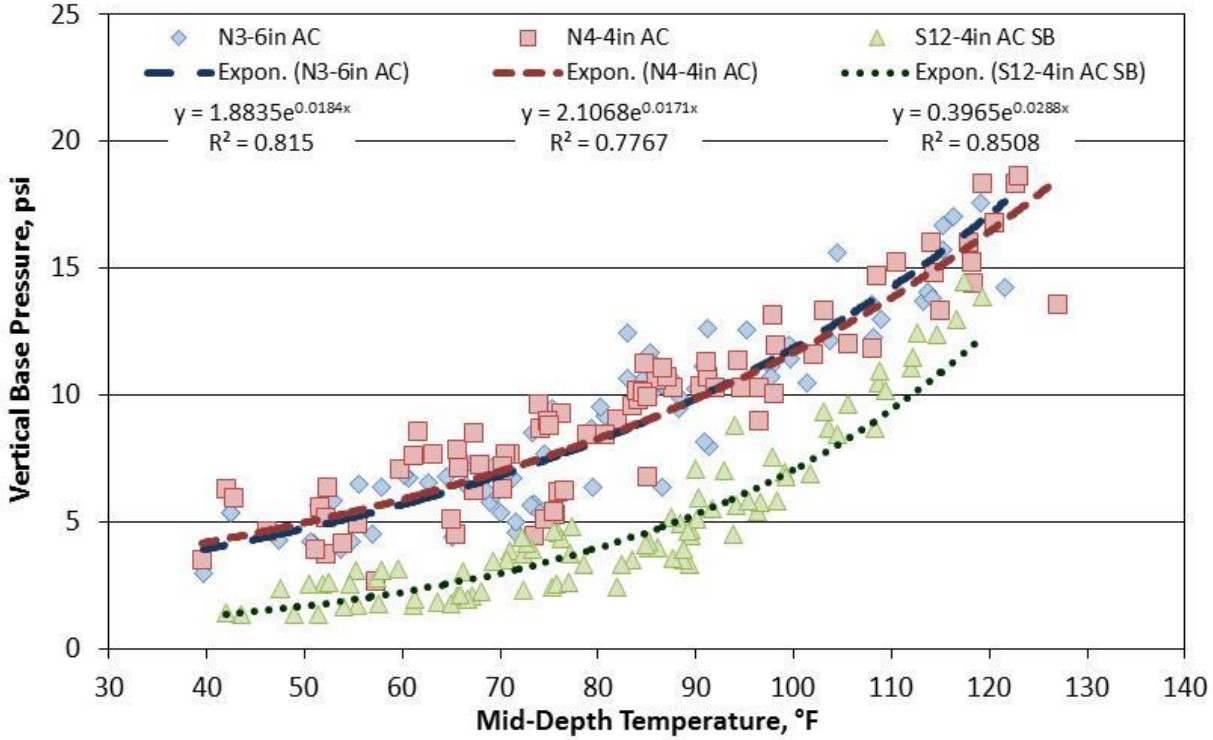

Figure 12. Vertical Base Pressure vs. Temperature

Figure 13 shows the vertical base pressure normalized to 68°F with respect to time. As noted in Figure 12, the base pressure measured in Sections N3 (6-in overlay with non-stabilized aggregate base) and N4 (4-in overlay with non-stabilized aggregate base) follow a similar trend; both increase over time, which may indicate some distress development not yet seen at the pavement surface. Conversely, as Section S12 (4-in overlay on a stabilized base) stiffens over time, there is a corresponding reduction in base pressure.

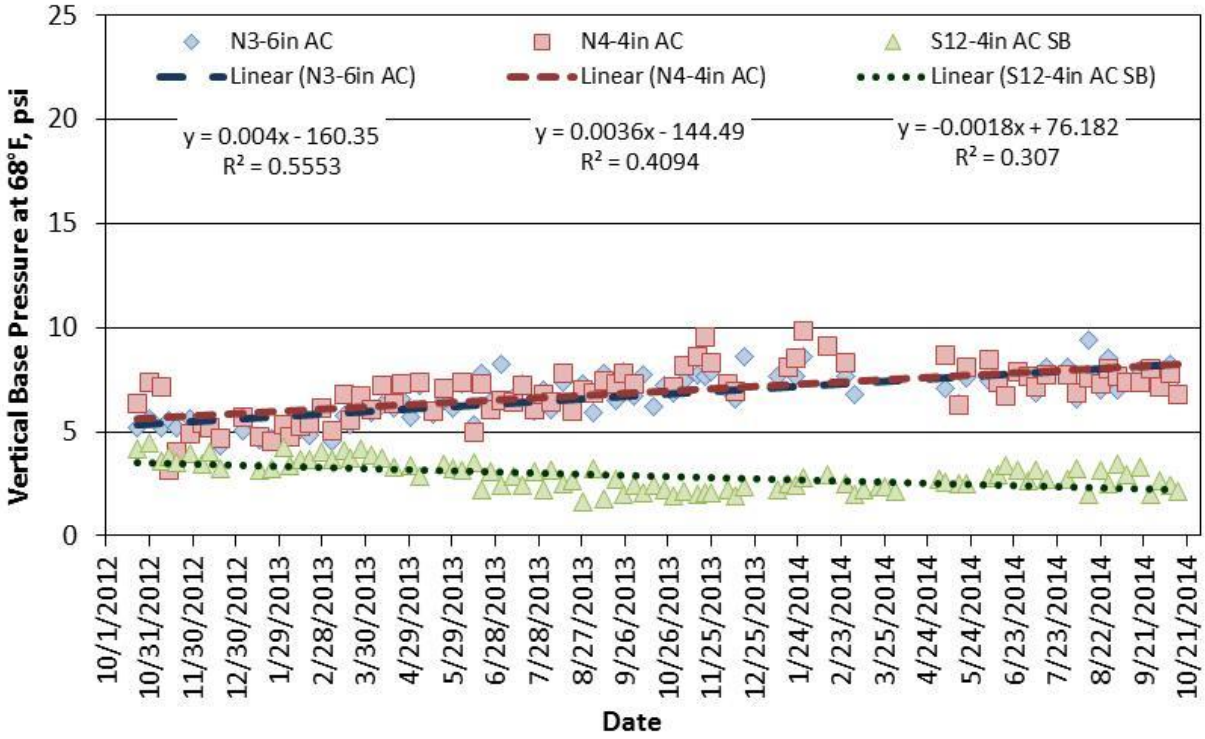

Figure 13. Vertical Base Pressure (Normalized to 68°F) vs. Time

Vertical Subgrade Pressure

Figure 14 shows the vertical pressure created during trucking operations at the top of the subgrade layer with respect to temperature. Also shown are the developed exponential regression equations for each section. As with the vertical base pressure, Sections N3 (6-in overlay with non-stabilized aggregate base) and N4 (4-in overlay with non-stabilized aggregate base) show similar behavior and a strong correlation with temperature. It is unclear why there is much more variability in the data from Section N3 as compared to the data from the other two sections. Section S12 (4-in overlay on a stabilized base) also shows a lesser subgrade pressure and a lesser dependence on temperature, presumably from the higher modulus and temperature insensitivity of the overlying materials.

The temperature-normalized (68°F) vertical subgrade pressure is shown in Figure 15. Sections N3 (6-in overlay with non-stabilized aggregate base) and N4 (4-in overlay with non-stabilized aggregate base) show very little change in pressure over time and a low R^2. Section S12 (4-in overlay on a stabilized base) shows a decreasing trend, consistent with a curing process and, thus, stiffening of the cement-stabilized layer over time.

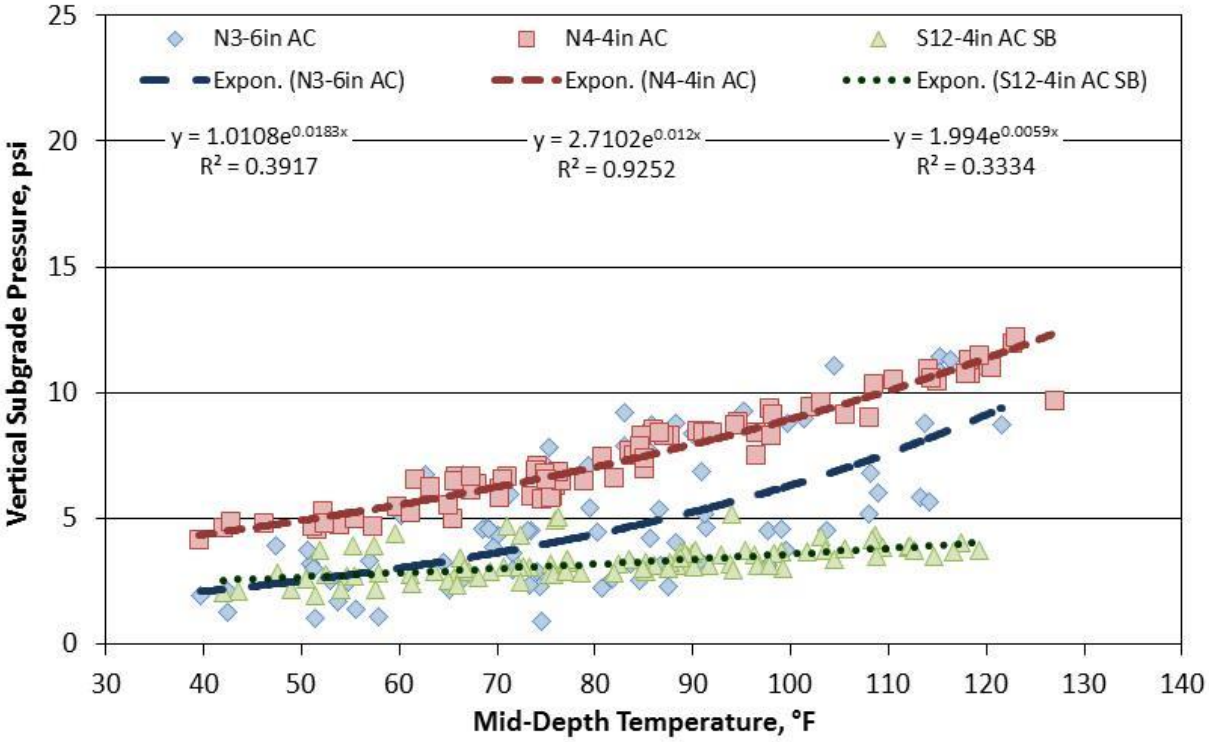

Figure 14. Vertical Subgrade Pressure vs. Temperature

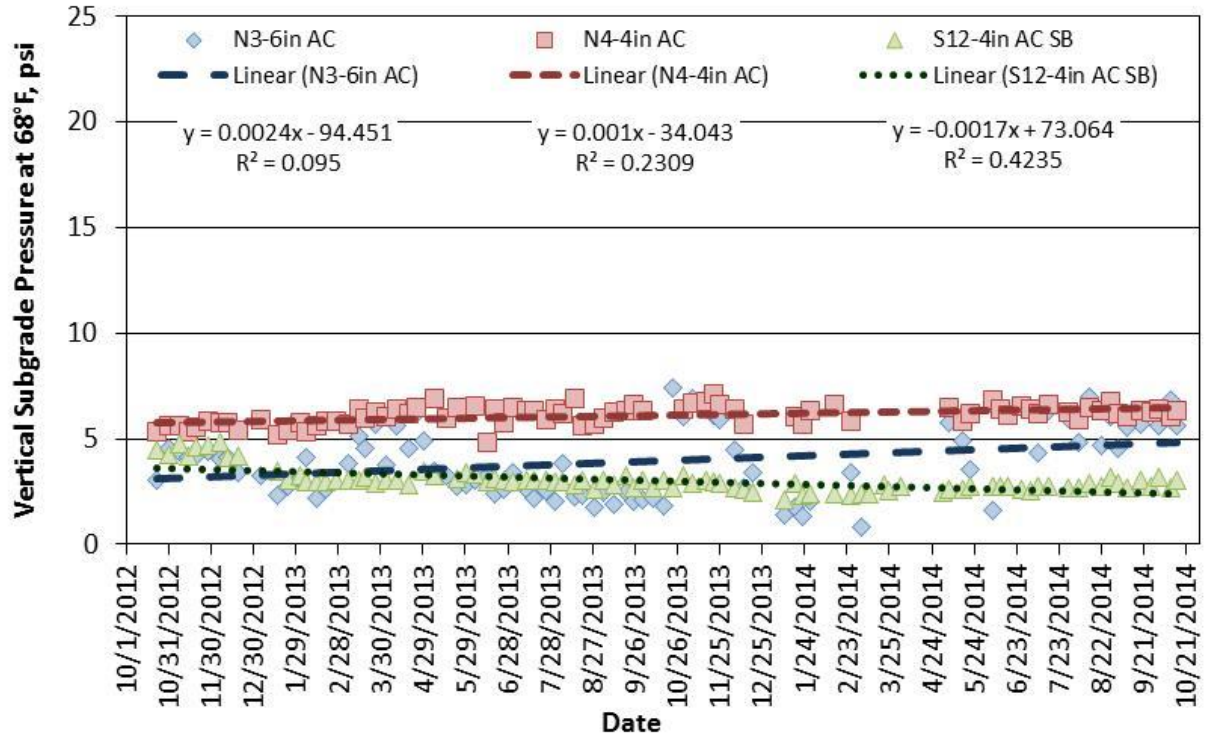

Figure 15. Vertical Subgrade Pressure (Normalized to 68°F) vs. Time

Deflection Testing

Deflection testing using an FWD was periodically performed during the 2-year test cycle. Testing was conducted at four locations within each section; two were located within the wheel path (and one of these two was above the instrumentation). During the analysis, each pavement section was treated as a three-layer structure consisting of the asphalt mixture and CCPR lifts as Layer 1, the aggregate base (Sections N3 and N4) or stabilized base (Section S12) as Layer 2, and the subgrade as Layer 3. Previous research demonstrated that the CCPR exhibited behavior similar to that of asphalt mixtures (Kim et al., 2009), so the researchers of this study decided to combine the asphalt mixture and CCPR layers for the back-calculation process. Subsequent dynamic modulus testing of CCPR material in the laboratory confirmed that the CCPR exhibited behavior consistent with that of asphalt materials and supported the combination of asphalt mixture and CCPR for analysis (Diefenderfer and Link, 2014).

Figure 16 shows the influence of mid-depth temperature on the back-calculated combined asphalt mixture and CCPR moduli. The sections having a non-stabilized aggregate base layer (Sections N3 and N4) show a strong influence of mid-depth temperature on the modulus, as demonstrated by a reduction in modulus with increasing temperature. Similar behavior was reported for asphalt mixtures at the NCAT Test Track (West et al., 2012), which further supports the decision to combine the CCPR and asphalt mixture for analysis purposes. Of interest, Section N3 (6-in overlay with non-stabilized aggregate base) appears to be slightly more temperature sensitive (i.e., a steeper slope was found for the regression) than Section N4 (4-in overlay with non-stabilized aggregate base). This may be caused by Section N4 having a higher percentage of recycled material in the combined asphalt mixture and CCPR layer than Section N3. Previous studies at the NCAT Test Track that compared a virgin asphalt mixture section to a 50% RAP section found the RAP section to be less temperature sensitive, presumably as a result of the presence of more aged binder (Vargas-Nordcbeck and Timm, 2013). Section S12 (4-in overlay on a stabilized base) showed a greater stiffness and much less temperature sensitivity than the other two sections, as shown in Figure 16. The lesser temperature sensitivity is demonstrated by an exponential regression coefficient that is less than one-half that of the other two sections and a corresponding lower R^2. Although it initially seemed reasonable to expect the stiffness of Section S12 to be similar to that of Section N4 (4-in overlay with non-stabilized aggregate base), because of the similar thickness of the asphalt mixture and CCPR layer, the analysis showed an increased stiffness. This is likely an artifact of the back-calculation process whereby the asphalt mixture and CCPR in Section S12 was given a greater apparent stiffness to adjust for lower measured deflections (this is also known as the compensating layer effect).

Figure 17 shows the normalized asphalt mixture and CCPR modulus with respect to time; linear trends were found for each data set. Sections N3 (6-in overlay with non-stabilized aggregate base) and N4 (4-in overlay with non-stabilized aggregate base) showed very little change in modulus over time. However, the modulus for Section S12 (4-in overlay on a stabilized base) clearly increased over time. It appears that the cement-stabilized layer was curing during the course of the study as expected. This mirrored the findings of Diefenderfer and Apeagyei (2011a) and Diefenderfer et al. (2012). The increased stiffness also supports the measured pavement responses shown previously in this study.

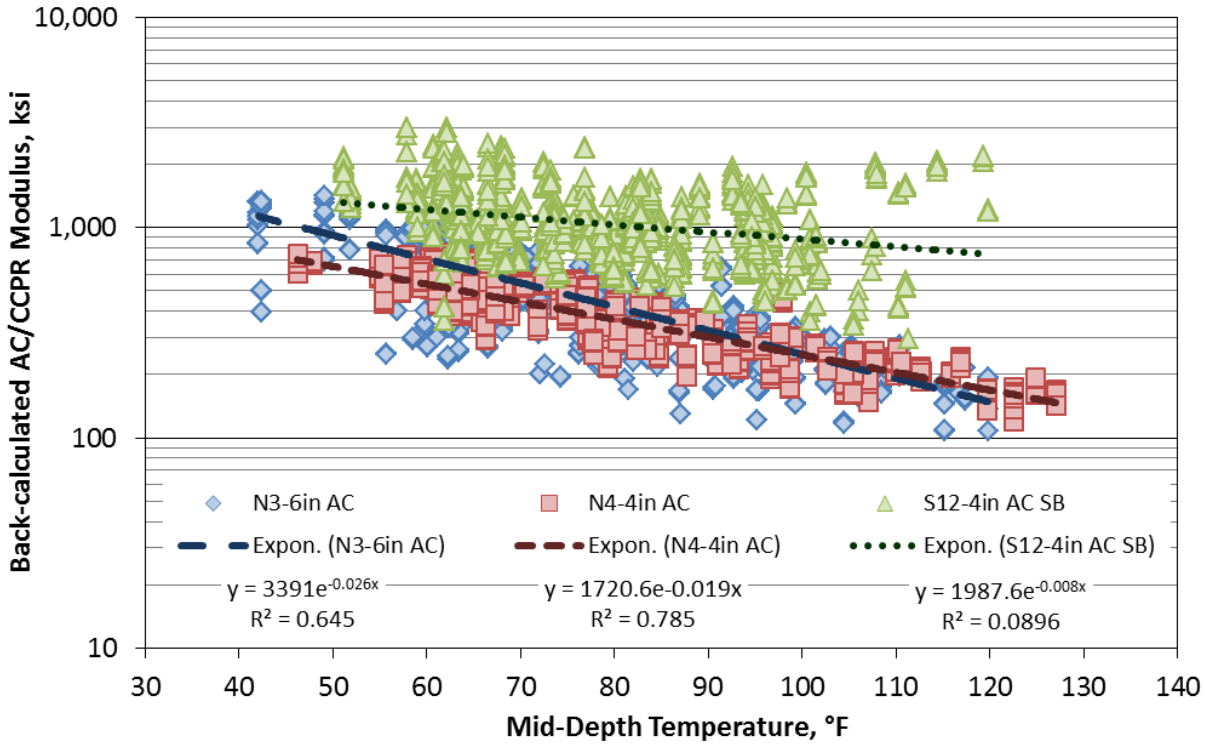

Figure 16. Back-calculated Modulus of Asphalt Mixture and CCPR vs. Temperature. AC = asphalt concrete; CCPR = cold central plant recycling.

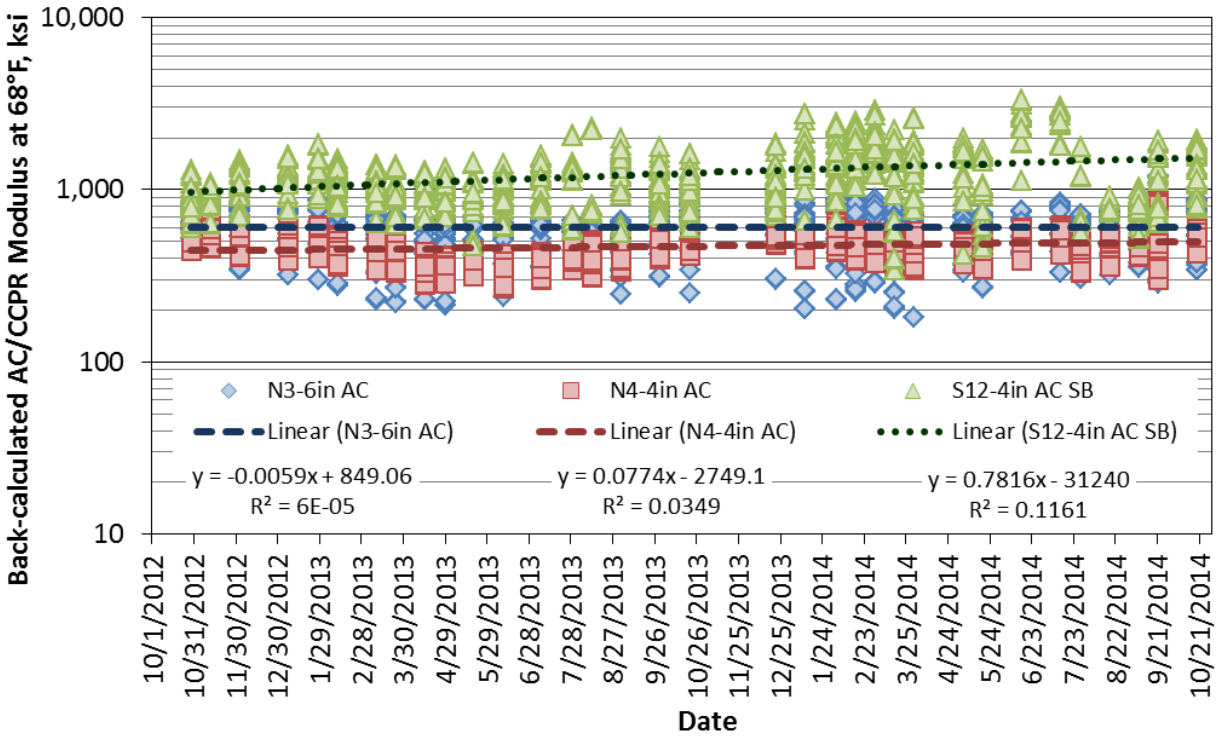

Figure 17. Back-calculated Modulus of Asphalt Mixture and CCPR at 68°F vs. Date. AC = asphalt concrete; CCPR = cold central plant recycling.

CCPR Structural Coefficient

The structural performance of Sections N3 (6-in overlay with non-stabilized base) and N4 (4-in overlay with non-stabilized base) over the 2-year research cycle was used to determine the structural contribution of the CCPR in terms of a structural layer coefficient. According to AASHTO's *Guide for Design of Pavement Structures* (AASHTO, 1993), a direct correlation may be established between the structural layer coefficient and the measured elastic modulus of each layer. Deflection testing is often used to calculate the elastic modulus of pavement layers and using this correlation, the structural contribution of a specific pavement layer can be determined. A structural layer coefficient for Section S12 (4-in overlay on a stabilize base) was not calculated since the presence of the stiff FDR-like stabilized base layer complicates the analysis. The back-calculation procedure is likely to attribute some of the stiffness from the stabilized layer to the overlying CCPR, artificially increasing its structural contribution. Since the CCPR layer for all three sections was produced and placed at the same time, the researchers assumed the CCPR layer in Section S12 was similar to the CCPR layer in Sections N3 and N4.

In the literature, structural layer coefficients for CIR and CCPR layers recycled with foamed asphalt have been reported as ranging between the layer coefficients of a granular base (0.06 to 0.14) and those of asphalt mixtures (0.35 to 0.54). Tia and Wood (1983) suggested using structural layer coefficients ranging from 0.25 to 0.40 for an artificially aged paving mixture recycled with foamed asphalt. Similarly, based on pavement deflection measurements in two road projects in Indiana, Van Wijk and Wood (1983) and Van Wyk et al. (1983) estimated the average layer coefficient of CIR with foamed asphalt to be between 0.26 and 0.37, with values as low as 0.10 and as high as 0.43. Marquis et al. (2003) determined that the layer coefficient varied from 0.22 to 0.35 for three foamed asphalt recycling projects in Maine. Sebaaly et al. (2004) recommended a layer coefficient of 0.26 for CIR layers in Nevada. Loizos and Papavasiliou (2006) and later Loizos et al. (2007) followed an analytical approach based on multilayer elastic analysis to estimate a structural layer coefficient of approximately 0.25 for a CIR constructed on a major highway in Greece. More recently, Diefenderfer and Apeagyei (2014) used deflections testing and laboratory measurements of the resilient modulus and indirect tensile strength of CCPR field cores to estimate a layer coefficient ranging from 0.36 to 0.48.

The ranges reported in the literature are wide and suggest that project- or material-specific coefficients may be needed for accurate performance prediction. In addition, under-estimating or over-estimating the true structural capacity of a CCPR layer can result in non-optimized (either excessive or deficient) thickness designs. Further, only two of the studies found in the literature specifically considered CCPR materials with foamed asphalt (Diefenderfer and Apeagyei, 2014; Diefenderfer and Link, 2014); most of the studies addressed other in-place recycling techniques. Although it has been suggested that CCPR performs similarly to CIR in the field (Apeagyei and Diefenderfer, 2013), the specific structural properties of CCPR for this project were measured.

Originally reported by Schwartz and Khosravifar (2013) and based on graphical correlations between the layer coefficient and the elastic modulus of asphalt mixtures developed at the AASHO Road Test (AASHTO, 1993), the layer coefficient was determined using the correlation presented in Equation 1 as follows:

$$a = 0.1665 \times ln(E) - 1.7309 \quad \text{(Eq. 1)}$$

where

a = structural layer coefficient

E = modulus of elasticity calculated from FWD testing (ksi).

Equation 1 was used to calculate a combined asphalt mixture and CCPR structural layer coefficient ($a_{AC/CCPR}$) using the temperature-normalized modulus values ($E_{AC/CCPR}$) obtained from FWD testing and shown in Figure 16. To determine a separate structural layer coefficient for just the CCPR material (a_{CCPR}), the combined $a_{AC/CCPR}$ value from Equation 1 needs to be separated by incorporating the thickness of the CCPR and asphalt mixture layers along with an assumed asphalt mixture structural layer coefficient, as shown in Equation 2:

$$a_{CCPR} = \frac{a_{AC/CCPR} \times D_{AC/CCPR} - [a_{AC} \times D_{AC}]}{D_{CCPR}} \quad \text{(Eq. 2)}$$

where

a_{CCPR} = CCPR structural layer coefficient

$a_{AC/CCPR}$ = combined asphalt mixture and CCPR structural layer coefficient (a from Equation 1)

$D_{AC/CCPR}$ = combined asphalt mixture and CCPR thickness (9.84 in in Section N3 and 8.17 in in Section N4)

a_{AC} = assumed asphalt mixture structural layer coefficient

D_{AC} = asphalt mixture thickness (5.81 in in Section N3 and 3.59 in in Section N4)

D_{CCPR} = CCPR thickness (4.03 in in Section N3 and 4.58 in in Section N4).

A structural layer coefficient of 0.54 for the asphalt mixture layers (a_{AC}) was used based on a previous layer coefficient calibration study performed at the NCAT Test Track (Peters-Davis and Timm, 2009). The value results in a conservative analysis relative to Virginia's standard value for a_{AC} of 0.44 (VDOT, 2000) in that a greater proportion of the structural capacity is attributed to the asphalt mixture layers and a smaller proportion to the CCPR. Reducing the structural layer coefficient of the asphalt mixture layer to 0.44 results in a calculated CCPR structural coefficient that is about 20% greater than if 0.54 is used.

Figure 18 shows the CCPR structural layer coefficient value calculated at each testing date and location for Sections N3 and N4 (using 0.54 as the assumed asphalt mixture layer structural coefficient). As seen in Figure 17, the CCPR structural layer coefficient varied according to the measured layer moduli. As expected, the layer coefficients showed certain variations with time, similar to those observed with the back-calculated moduli. The very small slope of the linear trendlines showed that the layer coefficients were not increasing or decreasing greatly over time and, thus, average values are reported herein. The average layer coefficients were calculated as 0.39 for Section N3 and 0.36 for Section N4, with standard deviations of 0.13 and 0.06, respectively. These values were within the range described in the literature. A statistical analysis (*t* test) revealed that these two values were different at a 95% confidence level ($\alpha = 0.05$, $p = 0.000$), and it was determined that the difference was not practically significant for pavement design purposes.

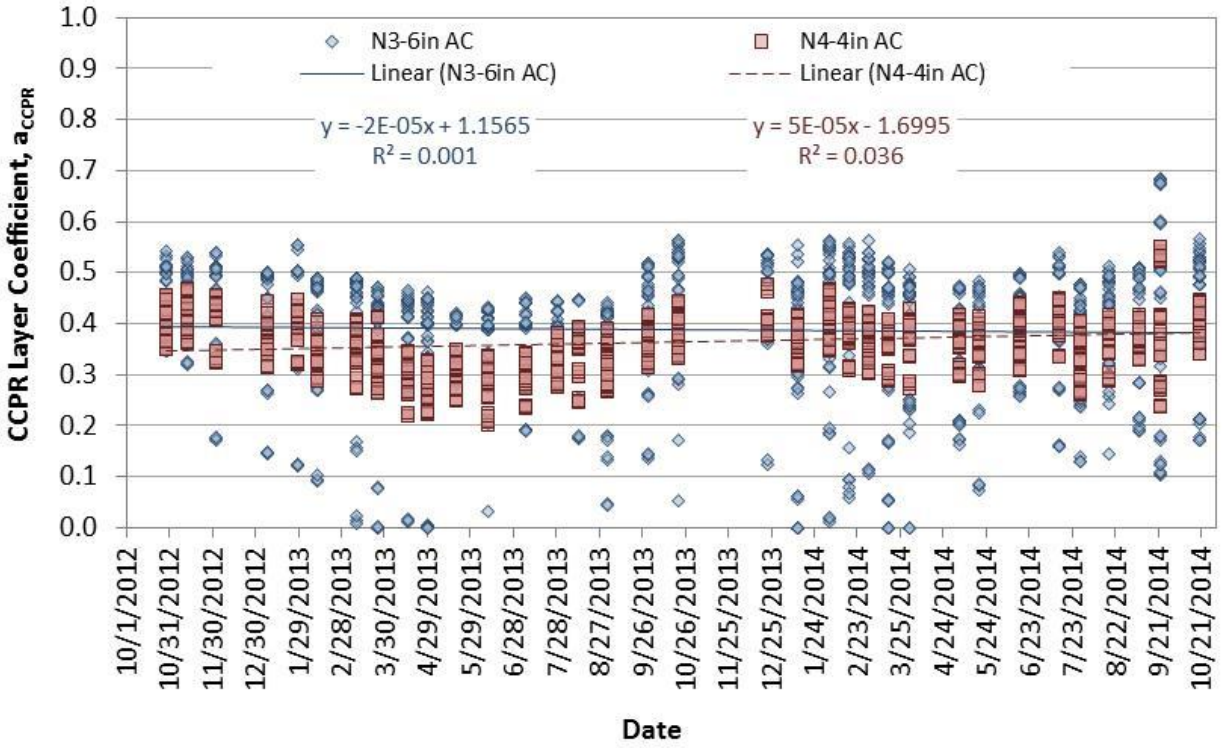

Figure 18. Structural Coefficient vs. Date/Traffic. CCPR = cold central plant recycling.

Although the average layer coefficient was lower for Section N4, the lowest individual values corresponded to one of the test locations in Section N3 (explaining the greater standard deviation obtained for this section). Figure 19 shows that most of the lower layer coefficient results in Section N3 could be attributed to Random Location 4. Random Location 4 was in the middle of the instrumented area of the section, which could contribute to lower moduli and lower structural coefficients because of disturbances caused by gauge installation. If Random Location 4 is removed from the analysis, the average layer coefficient for Section N3 increases to 0.43, with a corresponding standard deviation of 0.09. This relatively high layer coefficient approaches the value of 0.44 used by VDOT for conventional asphalt mixtures (VDOT, 2000). Although the discrepancies associated with the results from Random Location 4 had an effect on the calculated layer coefficient for Section N3, it is not possible to pinpoint the cause. Therefore, the researchers decided to include the variability attributed to Random Location 4 in the analysis as doing so would yield a more conservative structural layer coefficient from a design perspective. Once the trafficking is concluded, a forensic investigation is planned to determine the actual cause.

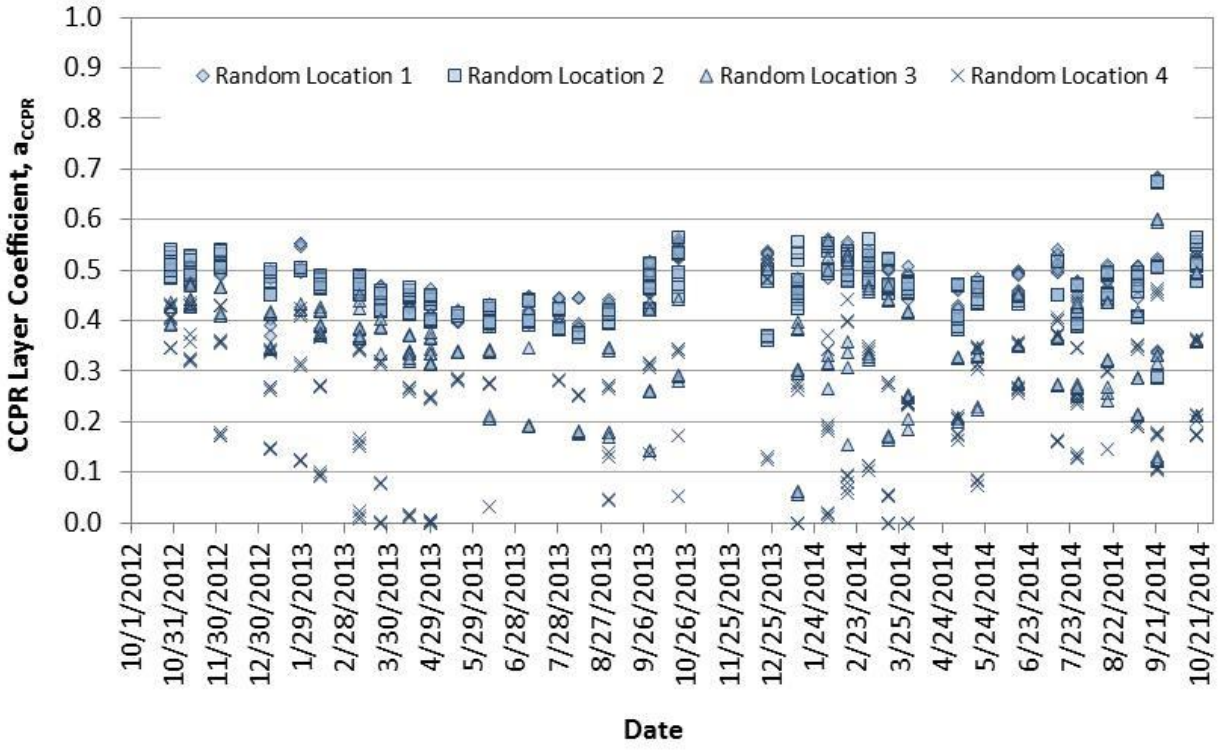

Figure 19. Structural Coefficient vs. Date for Section N3. CCPR = cold central plant recycling.

Summary of Findings

- No observable surface distresses were noted for any of the three sections during the 2-year test cycle in which approximately 10 million ESALs were applied.

- The ride quality (in terms of IRI) changed very little during the test cycle.

- The rutting was less than 0.3 in in all three sections after approximately 10 million ESALs were applied. The distribution of rutting within the various layers of the pavement structure was not identified.

- When the RAP content of the asphalt mixtures and the recycled content of the CCPR and stabilized base layers were considered, the pavement structure in Section S12 (4-in overlay on a stabilized base) contained approximately 80% recycled material. Including the aggregate base, the recycled content in Sections N3 (6-in overlay with non-stabilized aggregate base) and N4 (4-in overlay with non-stabilized aggregate base) was approximately 32% and 37%, respectively.

- The back-calculated asphalt mixture and CCPR moduli in Sections N3 (6-in overlay with non-stabilized aggregate base) and N4 (4-in overlay with non-stabilized aggregate base) responded to changes in temperature in a manner similar to conventional asphalt mixtures; a similar observation was noted in a laboratory study of CCPR (Diefenderfer and Link, 2014).

- The back-calculated asphalt mixture and CCPR moduli in Section S12 (4-in overlay on a stabilized base) had much less temperature sensitivity and higher moduli than the other sections.

- The stiffness of Section S12 (4-in overlay on a stabilized base) increased with time, based on the measured strain response and the temperature-normalized asphalt mixture and CCPR modulus from FWD testing. This was likely due to continued curing of the cement-stabilized base layer.

- The temperature-normalized modulus increased slightly or remained unchanged with respect to time for Sections N3 (6-in overlay with non-stabilized aggregate base) and N4 (4-in overlay with non-stabilized aggregate base), respectively. Conversely, Section S12 (4-in overlay on a stabilized base) showed an increase in temperature-normalized modulus over time.

- The measured strain from Section N3 (6-in overlay with non-stabilized aggregate base) was approximately 40% lower at 68°F than that from Section N4 (4-in overlay with non-stabilized aggregate base). The strain in Section S12 (4-in overlay on a stabilized base) was approximately 69% and 49% less than the strain levels for Sections N3 and N4 at 68°F.

- The relative ranking of each section based on measured strain (S12 < N3 < N4) was consistent across all temperatures.

- The temperature-normalized strain in Sections N4 (4-in overlay with non-stabilized aggregate base) and N3 (6-in overlay with non-stabilized aggregate base) increased slightly with respect to time. The amount of increase was less in Section N3. The temperature-normalized strain in Section S12 (4-in overlay on a stabilized base) decreased with respect to time.

- The aggregate base pressure measurements did not capture any differences between Sections N3 (4-in overlay with non-stabilized aggregate base) and N4 (6-in overlay with non-stabilized aggregate base). A higher dispersion in the data was noted from Sections N3 and N4, and thus differences were not statistically significant. The base pressure measurements in Section S12 (4-in overlay on a stabilized base) were lower than in the other sections. The temperature-normalized aggregate base pressure in Sections N3 and N4 increased with time whereas the temperature-normalized base pressure in Section S12 decreased as the section stiffened.

- The subgrade pressure measurements did capture differences between the sections. The highest reported was in Section N4 (4-in overlay with non-stabilized aggregate base) and the lowest was in Section S12 (4-in overlay on a stabilized base). Very little change in the temperature-normalized pressure with respect to time was noted in Sections N3 (6-in overlay with non-stabilized aggregate base) and N4, although Section S12 again tended toward lower pressure over time.

- A mathematical procedure was used to estimate the layer coefficient for the CCPR from back-calculated modulus data. The resulting layer coefficients ranged from 0.36 to 0.39 per inch (at a reference temperature of 68°F) by assuming a structural layer coefficient of 0.54 for the overlying asphalt mixture. These values were within the ranges reported in the literature for cold-recycled materials with foamed asphalt. Assuming a structural layer coefficient of 0.44 for the overlying asphalt mixture, the layer coefficient for the CCPR would be 20% higher.

- The dynamic modulus values of the CCPR materials decreased with increasing test temperature and increased with increasing test frequency. These results were similar to those for asphalt mixtures and showed that CCPR exhibits viscoelastic behavior.

- The dynamic modulus COV of the CCPR materials was greater, in general, at higher test temperatures and lower test frequencies.

- The dynamic modulus COV calculated for five replicates was outside the acceptable tolerance as specified in AASHTO TP 79 (AASHTO, 2013). However, these limits were developed based on traditional asphalt mixtures. This study (and other studies in the literature) suggests that different limits of variability are needed for recycled materials.

- The low horizontal tensile strain values at the bottom of the bound layers for Section S12 (4-in overlay on a stabilized base) were similar to those that described a long-life pavement structure (Tran et al., 2015).

CONCLUSIONS

- *Given that no observable surface distresses were evident during the testing, the three pavement designs used in this study are adequate for a design traffic level of at least 10 million ESALs for the subgrade support conditions described.*

- *The three test sections are examples of new or reconstructed pavement structures that could be built using a high percentage of recycled materials (ranging from approximately 32% to 80%) to achieve a long service life.*

- *The temperature-dependent response and the location of the maximum tensile strain support the assumption of treating CCPR as a viscoelastic material for mechanistic-modeling and pavement design purposes. However, this does not rule out the possibility that other material type behaviors may be needed to describe CCPR behavior completely.*

- *Part of the increased asphalt mixture and CCPR moduli measured from Section S12 (4-in overlay on a stabilized base) may be caused by the back-calculation process in which some of the stiffness from the stabilized base layer may be artificially attributed to the asphalt mixture and CCPR layer (also known as the compensating layer effect).*

- *The reduced temperature sensitivity of the asphalt mixture and CCPR modulus of Section S12 (4-in overlay on a stabilized base) was likely caused by the use of the cement-stabilized layer. The stiffness properties of the cement-stabilized layer were not expected to vary with temperature.*

- *The measured strain response in Section N4 (4-in overlay with non-stabilized aggregate base) increased with time, likely because of some internal damage that is not yet detectable by FWD testing or observed at the surface. The measured strain response in Section N3 (6-in overlay with non-stabilized aggregate base) was stable with time and thus was likely not undergoing internal damage.*

- *The difference in measured strain (40% less at 68°F) between Sections N3 (6-in overlay with non-stabilized aggregate base) and N4 (4-in overlay with non-stabilized aggregate base) was attributed to the increased asphalt mixture thickness as expected.*

- *The difference in measured strain (69% less at 68°F) between Sections N4 (4-in overlay with non-stabilized aggregate base) and S12 (4-in overlay on a stabilized base) was attributed to the presence of the cement-stabilized base layer.*

- *The temperature-normalized base pressure was not found to be different between Section N3 (6-in overlay with non-stabilized aggregate base) and Section N4 (4-in overlay with non-stabilized aggregate base). This may have been the case because of the higher variability in the measured data in Section N3 and was an unexpected result.*

- *The temperature-normalized base pressure for Section S12 (4-in overlay on a stabilized base) was much less than in the other sections; this decrease was thought to be caused by the presence of higher modulus materials within the pavement structure.*

- *The additional 2 in of asphalt mixture placed in Section N3 (6-in overlay with non-stabilized aggregate base) reduced the temperature-normalized subgrade pressure when compared to that of Section N4 (4-in overlay with non-stabilized base). In addition, the temperature-normalized subgrade pressure for Section S12 (4-in overlay on a stabilized base) was less than for the other sections; again, this decrease was thought to be caused by the presence of the underlying cement-stabilized base.*

RECOMMENDATIONS

1. *VDOT's Materials Division, VDOT's Maintenance Division, and VTRC should continue to emphasize the use of pavement recycling (such as CCPR and FDR) as an asphalt pavement rehabilitation or new construction technique for use by VDOT districts on those projects for which it is most suitable.* Other agencies and the literature reported considerable per-project cost and environmental savings when these techniques were used.

2. *VDOT's Materials Division, VDOT's Maintenance Division, and VTRC should proactively identify locations for future pavement recycling projects and work with VDOT districts to perform further detailed project level pavement investigations at these potential locations.* VDOT's Pavement Management System can be used to locate sites for potential recycling projects based on observed surface distresses, traffic volume, structural capacity, and frequency of maintenance.

3. *VDOT's Materials Division and VTRC should review the VDOT Materials Division Memorandum MD 386-15 (VDOT, 2015) with district pavement management and design staff.* The memorandum states that cold pavement recycling should be considered for pavement rehabilitation projects where it is a viable option.

BENEFITS AND IMPLEMENTATION

Benefits

The drivers for using pavement recycling methods are the potential cost savings and reductions in environmental impacts. These savings and reductions are achievable since pavement recycling techniques reuse nonrenewable resources, reduce the amount of virgin materials consumed, reduce the energy consumed in materials production, and reduce the transportation of paving materials. A study by Alkins et al. (2008) showed that using pavement recycling as a rehabilitation strategy can reduce greenhouse gas emissions by 50% and have cost savings ranging from 40% to 50% per project. The Nevada Department of Transportation similarly showed cost savings of nearly $600 million over a 20-year period (Bemanian et al., 2006).

An example of cost savings achievable by VDOT is presented here using typical awarded materials costs for VDOT as of December 2015. The example is a 1-mile segment of pavement to be constructed (as either a new roadway or a reconstruction of an existing roadway) using recycling versus conventional techniques. Typical unit prices from the VDOT Statewide Bid Tab Query are shown in Table 3. As there was no information for CCPR and FDR projects, these prices were estimated at $45 per ton and $12 per yd^2, respectively, based on the experiences of the researchers.

Table 3. Virginia Statewide Average Unit Prices per Ton for Awarded Contracts (January 2014-April 2015)

Material	Nominal Maximum Aggregate Size, mm (Asphalt Binder Performance Grade)	Unit Price, $
Asphalt Surface (SMA)	12.5 (76-22)	106
Asphalt Surface (Superpave)	9.5 (64-22)	99
Asphalt Intermediate (SMA)	19.0 (76-22)	93
Asphalt Intermediate (Superpave)	19.0 (64-22)	80
Asphalt Base (Superpave)	25.0 (64-22)	82
Cement-Treated Aggregate Base	N/A	44

Source: VDOT Statewide Bid Tab Query.
SMA = stone matrix asphalt.

Tables 4 and 5 show example pavement designs for high-volume and low-volume roadways, respectively, using both recycled and conventional techniques. The pavement designs were developed in accordance with VDOT guidelines using the AASHTO 1993 pavement design methodology (VDOT, 2000; VDOT, 2015). Using the two examples shown and assuming similar service lives, the use of a strategy that employs pavement recycling techniques has the potential to save approximately $188,000 and $60,000 per lane-mile, respectively, for high-volume and low-volume roadways. These equate to savings of approximately 35% and 26%, respectively, for high-volume and low-volume roadways.

Implementation

The three recommendations developed for this study will be implemented as follows:

1. VDOT's Materials Division will continue emphasizing recycling processes as tools for pavement construction and rehabilitation and is working to increase awareness of these techniques. A recent example includes the delivery of a National Highway Institute pavement recycling course that was presented to VDOT staff with the goal of increasing awareness and knowledge of pavement recycling processes.

2. VDOT's Pavement Management System can be used to identify potential recycling projects. An initial list of potential sites can be developed by noting where the service life of past maintenance activities is less than the average. This initial list can be refined by following the project selection guidelines provided in the VDOT Materials Division Memorandum MD 386-15 (VDOT, 2015) and by conducting a project-level investigation of the distress mechanisms.

3. The VDOT Materials Division Memorandum MD 386-15 (VDOT, 2015) will be reviewed with VDOT district pavement management and design staff at future pavement-related meetings (e.g., the VTRC Pavement Research Advisory Committee, the VDOT Pavement Forum, etc.) by staff from the Materials Division and VTRC.

Table 4. Example Pavement Designs for Higher Volume Roadway

Material		Assumed thickness, in	Structural Layer Coefficient	Structural Layer Coefficient × Thickness	Density, lb/ft^3	Quantity for 1 Lane-mile	Cost
Recycled	Asphalt Surface (SMA)	2	0.44	0.88	145	766 tons	$81,154
	Asphalt Intermediate (SMA)	2.5	0.44	1.1	145	957 tons	$89,001
	CCPR	6	0.35[a]	2.1	132	2,091 tons	$94,090
	FDR	12	0.25[a]	3	N/A	7,040 yd^2	$84,480
	Total	22.5		7.1			$348,724
Conventional	Asphalt Surface (SMA)	2	0.44	0.88	145	766 tons	$81,154
	Asphalt Intermediate (SMA)	2.5	0.44	1.1	145	957 tons	$89,001
	Asphalt base	8	0.44	3.52	142	2,999 tons	$245,921
	Cement-Treated Aggregate Base	8	0.2	1.6	130	2,746 tons	$120,806
	Total	20.5		7.1			$536,882

SMA = stone matrix asphalt; CCPR = cold central-plant recycling; FDR = full-depth reclamation.
[a] VDOT, 2015.

Table 5. Example Pavement Designs for Lower Volume Roadway

Material		Assumed Thickness, in	Structural Layer Coefficient	Structural Layer Coefficient × Thickness	Density, lb/ft^3	Quantity for 1 Lane-mile	Cost
Recycled	Asphalt Surface (Superpave)	2	0.44	0.88	145	766 tons	$75,794
	CCPR	6	0.35[a]	2.1	132	2091 tons	$94,090
	Total	8		3.0			$169,884
Conventional	Asphalt Surface (Superpave)	2	0.44	0.88	145	766 tons	$75,794
	Asphalt Base	5	0.44	2.2	142	1874 tons	$153,701
	Total	7		3.1			$229,495

CCPR = cold central-plant recycling.
[a] VDOT, 2015.

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