

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

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<p>Abstract:</p> <p>The Virginia Department of Transportation (VDOT) contracted with the National Center for Asphalt Technology (NCAT) in 2012 to install, instrument, and monitor three pavement test sections at the NCAT Test Track. These sections were subjected to approximately 20 million 18-kip equivalent single axle loads between 2012 and 2017. Pavement instrumentation was included in each test section to capture the temperature and pavement response from truck loading. The three test sections, having a length of 200 ft each, included two different asphalt overlay thicknesses placed on top of a 5-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.</p> <p>The purpose of this study was to conduct a second round of testing during the 2015-2017 research cycle and to evaluate the performance of the three test sections constructed using cold central plant recycling (CCPR) material. This study follows an earlier effort performed during the 2012-2014 research cycle. The performance of the sections was documented by analyzing the results of deflection testing using a falling weight deflectometer; temperature, pressure, and strain measurements from embedded instruments; and observable surface deterioration of the pavement sections.</p> <p>The study found that the performance of the three recycled sections continues to be excellent after 20 million equivalent single axle loads of traffic loading. This was evidenced by the following examples of functional performance: no observable cracking at the pavement surface, rut depths less than 0.3 in, and steady measurements of ride quality. The claim of excellent performance is also supported by the following examples of structural performance: steady or increasing modulus values for the asphalt/CCPR layer and steady or slightly increasing tensile strain at the bottom of the CCPR layer, vertical base pressure, and vertical subgrade pressure.</p> <p>The study recommends that VDOT continue to sponsor trafficking of two of the recycled sections for the 2018 track cycle and further recommends that VDOT find ways to identify and fund additional projects to implement the pavement recycling concepts on in-service pavements in Virginia.</p>				

FINAL REPORT

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

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ABSTRACT

The Virginia Department of Transportation (VDOT) contracted with the National Center for Asphalt Technology (NCAT) in 2012 to install, instrument, and monitor three pavement test sections at the NCAT Test Track. These sections were subjected to approximately 20 million 18-kip equivalent single axle loads between 2012 and 2017. Pavement instrumentation was included in each test section to capture the temperature and pavement response from truck loading. The three test sections, having a length of 200 ft each, included two different asphalt overlay thicknesses placed on top of a 5-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 conduct a second round of testing during the 2015-2017 research cycle and to evaluate the performance of the three test sections constructed using cold central plant recycling (CCPR) material. This study follows an earlier effort performed during the 2012-2014 research cycle. The performance of the sections was documented by analyzing the results of deflection testing using a falling weight deflectometer; temperature, pressure, and strain measurements from embedded instruments; and observable surface deterioration of the pavement sections.

The study found that the performance of the three recycled sections continues to be excellent after 20 million equivalent single axle loads of traffic loading. This was evidenced by the following examples of functional performance: no observable cracking at the pavement surface, rut depths less than 0.3 in, and steady measurements of ride quality. The claim of excellent performance is also supported by the following examples of structural performance: steady or increasing modulus values for the asphalt/CCPR layer and steady or slightly increasing tensile strain at the bottom of the CCPR layer, vertical base pressure, and vertical subgrade pressure.

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INTRODUCTION

Pavement recycling is a technology that can be used to restore the service life of pavement structures and to stretch available funding for pavement rehabilitation (Asphalt Recycling and Reclaiming Association [ARRA], 2015). In general, pavement recycling remixes the existing pavement material (either in situ or through a mobile plant) and reuses it in the final pavement in the form of a stabilized layer. Some of the most commonly cited benefits of using pavement recycling techniques to rehabilitate and repair asphalt concrete pavements include reduced use of virgin materials, reduced fuel consumption, reduced lane closures, reduced emissions related to construction, and reduced cost (Nataatmadja, 2001; Stroup-Gardiner, 2011; Thenoux et al., 2007). Pavement recycling methods include the following processes: cold planing, hot in-place recycling, cold recycling, and full depth reclamation (FDR). Cold recycling includes the techniques of cold in-place recycling (CIR) and cold central plant recycling (CCPR) (ARRA, 2015).

FDR is often used to correct structural deficiencies and defects that are deep within the pavement structure. The depth of pulverization achieved by the reclaimer (shown in Figure 1) depends on the thickness of the bound layers of the existing pavement but is typically 4 to 12 in (ARRA, 2015). FDR is performed on the bound layers and a portion of the underlying unbound materials. FDR may consist of simply pulverizing and remixing the roadway foundation, termed mechanical stabilization, but it most often incorporates one or several stabilizing agents, termed chemical stabilization. A list of typical FDR stabilizing agents includes chemical stabilizers such as portland cement, lime, fly ash, cement, and lime kiln dust and asphalt-based stabilizers in the forms of asphalt emulsion and foamed asphalt (ARRA, 2015). The most commonly used stabilizing agents are foamed or emulsified asphalt binder in combination with a chemical additive such as cement or lime (Wirtgen GmbH, 2010).



Figure 1. Reclaimer Used for Full Depth Reclamation

CIR rehabilitates the upper portions of the bound layers of an asphalt pavement, typically extending to depths of 4 to 6 in (ARRA, 2015). Typical recycling agents for CIR include asphalt emulsion and foamed asphalt along with a chemical additive such as cement, lime, fly ash, or lime kiln dust (ARRA, 2015). Chemical additives are often used in combination with asphalt recycling agents to improve resistance to moisture damage and early strength.

CCPR is a process through which reclaimed asphalt pavement (RAP), taken either from a milled roadway or from existing stockpiles of millings, are recycled and used to construct a roadway. The RAP is brought to a centrally located plant (see Figure 2) that is used to mix recycling additive(s), similar to those used with CIR, with the RAP to ensure a consistent mix. The plants are portable in that they can be set up temporarily on or near a project or kept at a fixed location. Aside from the use of a central plant, the addition of recycling agents into the CCPR mixture is similar to CIR. Recent studies have also shown that mechanical properties of CCPR and CIR mixtures are similar (Apeageyi and Diefenderfer, 2013; Bowers et al., 2018; Diefenderfer et al., 2016a). However, the CCPR process offers the opportunity to process the RAP through a mobile crusher on-site before adding it to the CCPR plant for improved gradation control compared to CIR.

The benefits of using the CCPR process are 2-fold. First, material can be removed from the roadway and stockpiled to be used as a recycled layer after the underlying foundation is either stabilized or replaced if needed. Second, existing stockpiles of RAP can be treated and used in the construction of new pavements or in the rehabilitation of different existing pavements. Although the CCPR process has not been used as widely as CIR, it has been used recently for successful implementation on sections with high traffic volumes (Diefenderfer et al., 2016b; Ma et al., 2017; Timm et al., 2018).



Figure 2. Cold Central Plant Recycling Plant

To study the performance of pavement sections constructed with different recycling techniques, the Virginia Department of Transportation (VDOT) contracted with the National Center for Asphalt Technology (NCAT) in 2012 to install, instrument, and monitor three pavement test sections at the NCAT Test Track. These sections, designated Sections N3, N4, and S12, were subjected to approximately 20 million 18-kip equivalent single axle loads (ESALs) between 2012 and 2017.

Although all three sections included a 5-in-thick CCPR base, Sections N3 and N4 included an aggregate base and two different asphalt surface thicknesses. The CCPR base placed in Section S12 was placed on top of a cement-stabilized foundation, simulating FDR. The cement-stabilized foundation was placed in accordance with typical FDR procedures and used a reclaimer; however, it does not meet the true definition of an FDR layer since the existing asphalt pavement was removed prior to construction. All three sections were surfaced with two or more layers of an asphalt mixture.

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 cement-stabilized foundation (composed of the existing aggregate base and upper portion of the existing subgrade) was 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 and continued for 2 years. After a 12-month break between test cycles, a second round of testing began in October 2015 and continued until November 2017. Pavement instrumentation was included in each test section to capture the temperature and pavement response from truck loading.

Originally, the researchers considered using CIR to construct the recycled base on the test sections. However, the 200-ft length of each section was considered too short to allow the production of a consistent material, and thus the CCPR process 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 evaluate the structural performance of Virginia's recycled pavement sections as built at the NCAT Test Track. The study also sought to determine the performance of pavement sections having different overlay designs with and without a stabilized foundation.

The scope of the study included the three pavement sections at the NCAT Test Track constructed during the 2012 track cycle: Sections N3, N4, and S12. Two sections included CCPR material plus an aggregate base (having two different asphalt surface thicknesses), and one used CCPR material in conjunction with a cement-stabilized foundation, simulating FDR. The three test sections each had a length of 200 ft. The performance was assessed during the 2012 and 2015 track cycles (covering a period from 2012 to 2017) by analyzing the results of field testing and the response of temperature, pressure, and strain sensors placed during construction.

METHODS

Two tasks were undertaken to achieve the study objectives:

1. Summarize the design and construction of the recycled test sections.
2. Evaluate the performance of the recycled test sections through field tests and embedded instrumentation.

Summarize Design and Construction of Recycled Test Sections

The construction processes were summarized by NCAT and VDOT staff who were present during the construction of the test sections in 2012; Diefenderfer et al. (2016b) provide this information. The "Results and Discussion" section also includes a discussion of the design of the three test sections.

Evaluate Field Performance of Recycled Test Sections

Rut Depth and Ride Quality

Rut depth measurements 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 had a length of only 200 ft, the data were reported as average values over the length of the entire 200-ft section.

Elastic Modulus of Asphalt/CCPR Layers

Deflection testing to assess structural capacity was performed every 2 to 3 weeks 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) during the 2012 research cycle and three load levels (6,000; 9,000; 12,000 lbf) during the 2015 research cycle. The removal of the 16,000 lbf load level in the second cycle was to help reduce testing costs. Following two unrecorded seating drops, three deflection basins were recorded at each load level. The deflection data were analyzed in accordance with AASHTO's *Guide for Design of Pavement Structures* (AASHTO, 1993). The analysis included calculating the combined modulus of the asphalt/CCPR layers. The results are presented as average values for each test date.

Instrumentation

Instruments to measure pavement temperature, pressure, and strain were placed in the three test pavement sections during construction. The temperature was measured at the top of the surface layer, at the mid-depth of the combined thickness of the CCPR material 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 the bottom of the CCPR layer. Pressure cells measured the pressure at the top of the aggregate layer and at the top of the subgrade. Additional details regarding the instrumentation used at the NCAT Test Track are provided by Timm (2009).

RESULTS AND DISCUSSION

Summary of Design and Construction of Recycled Test Sections

A mix design process, described in Wirtgen GmbH (2010), was used to find the optimum properties of the CCPR material. The optimum properties were found at a foamed asphalt content of 2% using PG 67-22 binder and 1% Type II portland cement as a chemical additive. Similarly, the optimum cement content for the cement-stabilized base was 4% Type II portland cement. The cement-stabilized base, composed of a mixture of existing aggregate and subgrade from the NCAT Test Track, had an average compressive strength of 256 psi after 7 days, a

maximum dry density of 130.0 lb/ft³, 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. Additional mix design details, including the mix design process, are provided by Diefenderfer et al. (2016b).

The three sections constructed at the NCAT Test Track, Sections N3, N4, and S12, are shown graphically in Figure 3 including the as-planned thickness of each layer. Construction of the three test sections occurred during July and August 2012. Each of the three test sections featured a stone matrix asphalt (SMA) surface layer, one or more dense-graded asphalt mixture layers, and then 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 dense-graded intermediate mixture had a 19.0 mm NMAS and used a PG 67-22 binder. The CCPR material was made from 100% RAP. All of the aggregate/RAP materials for the SMA, dense-graded asphalt mixture, and CCPR material were shipped from Virginia to NCAT.

The bound pavement layers (SMA, dense-graded asphalt mixture, and CCPR) in Sections N3 and N4 were constructed on top of a crushed granite aggregate base layer; Section S12 was built on a 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 and 4 in of asphalt materials over 5 in of CCPR material, 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 CCPR material was placed using a conventional asphalt paver. The CCPR material was placed at a depth of approximately 6 in and later profile milled to the desired thickness. Diefenderfer et al. (2016b) provides additional construction details.



Figure 3. Schematic of NCAT Test Track Sections Including CCPR. The 8-in FDR layer was produced using an FDR process, but only unbound materials were reclaimed. The layer is called a “cement-stabilized foundation” in this report. NCAT = National Center for Asphalt Technology; CCPR = cold central plant recycling; FDR = full depth reclamation; AC = asphalt concrete; Agg = aggregate.

Figure 4 shows the average as-built thickness of each test section and reflects the natural variation attributable to standard construction practices at the NCAT Test Track. The thickness of each layer was measured at 12 different locations within each section and averaged. From this it can be seen that Sections N3 and N4 can be used to determine the difference in performance of a CCPR base having 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.

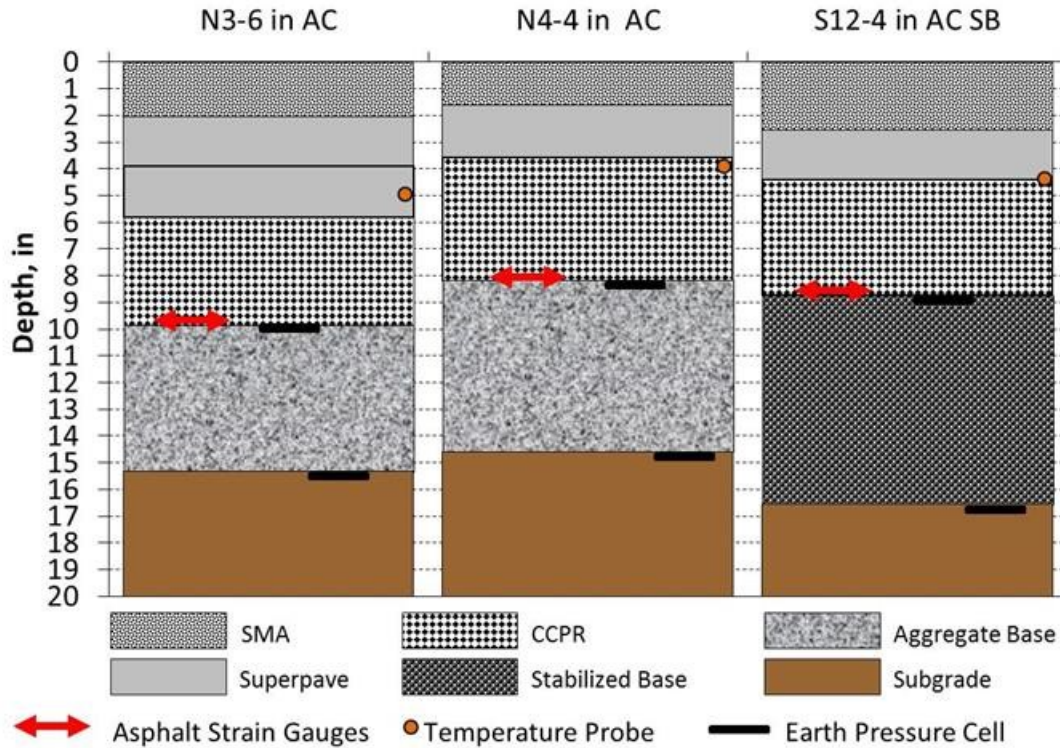


Figure 4. VDOT Sections at NCAT Test Track Showing Average As-Built Thicknesses and Depth of Instrumentation. NCAT = National Center for Asphalt Technology; AC = asphalt concrete; SB = stabilized base; SMA = stone matrix asphalt; CCPR = cold central plant recycling.

Field Performance of Recycled Test Sections

An assessment of the performance of the three test sections was conducted periodically during each of the 2-year research cycles. During this time, approximately 20 million ESALs (10 million ESALs per research cycle) 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. The current status at 20 million ESALs is an equivalent traffic level to approximately 8, 10, and 20 years of traffic on the most heavily traveled portions of I-81, I-95, and I-64, respectively.

Visual Observation

At the end of the second 2-year research cycle, no cracking or other observable surface distresses were noted on any of the three recycled pavement sections. Figure 5 shows the condition of the three test sections at the completion of the second research cycle.



Figure 5. Visual Condition of VDOT Sections at NCAT Test Track: (a) Section N3; (b) Section N4; (c) Section S12.

Rut Depth and Ride Quality

Figure 6 shows the rut depths measured in each of the three test sections. The figure shows that the rut depth increased during the two track cycles from approximately 0.1 in to less than approximately 0.3 in. The results also show the effects of elevated summer temperatures by discrete increases in rut depth at approximately 3.5 and 8 million ESALs. The results also show a decrease in the measured rut depth in Section S12 (4-in overlay on a stabilized base) between the first and second track cycle (beginning at approximately 10 million ESALs). Between the two track cycles, a short portion (approximately 20 ft) of the beginning of Section S12 was milled to address a local rough area resulting from the initial construction.

Figure 6 also shows linear trendlines fit to the rut depth measurements from each section. Since the pavement surface in Section S12 was corrected to address a local deficiency between track cycles (at 10 million ESALs), separate trends were calculated for the period 0-10 million ESALs and 10-20 million ESALs. Figure 6 also shows the regression equation of the line that could be used to describe the data from each section. There are two equations for Section S12: the upper one shown in Figure 6 represents data from 0-10 million ESALs, and the lower one shown in Figure 6 represents data from 10-20 million ESALs. Figure 6 shows that the rutting performance for Section S12 is expected to be better than for Sections N3 (6-in overlay with non-stabilized base) and N4 (6-in overlay with non-stabilized base) since Section S12 has a lesser slope after 10 million ESALs. From Figure 6 it is clear that the magnitude of rutting in Section S12 was influenced by the milling operation, but the researchers do not believe the change in rutting with respect to traffic loading (or rate of rutting) would be similarly influenced. The trendlines indicate that the rutting performance for Sections N3 and N4 is expected to be similar.

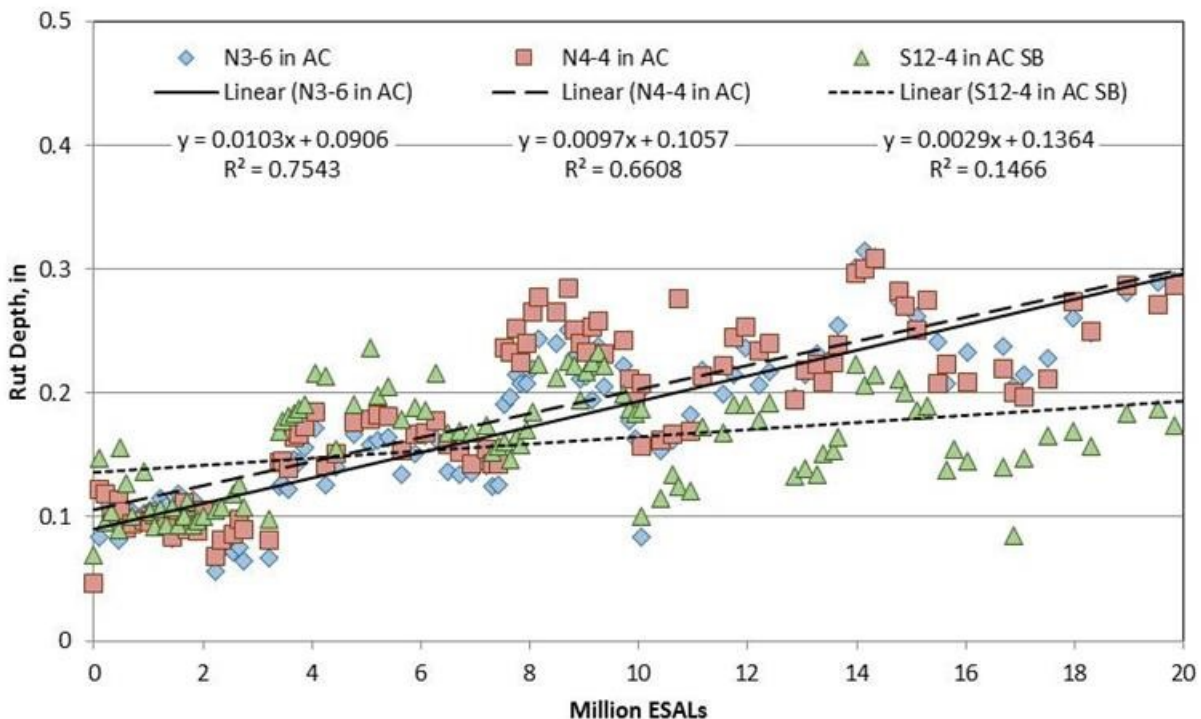


Figure 6. Rut Depth Results. AC = asphalt concrete; SB = stabilized base.

Table 1 gives an indication of the expected rutting service life for the three test sections. The regression equations shown in Figure 6 were rearranged to solve for the number of ESALs given a rut depth of 0.5 in. From Table 1 it can be seen that the expected rutting service life of the three sections as originally constructed is between approximately 35 and 40 million ESALs. The modification to the initial portion of Section S12 improved the projected rutting service life from approximately 35 million to approximately 150 million ESALs. If similar distresses were observed on a comparable in-service pavement, thin milling and replacement of the surface course would be a typical maintenance strategy.

Figure 7 shows the ride quality, expressed in terms of the International Roughness Index (IRI), for the three recycled sections. The IRI increased for Sections N3 and N4 with respect to ESAL loading. The IRI was calculated for Section S12 for the two research cycles separately for reasons mentioned previously. For the 2012 research cycle, the IRI for Section S12 increased at a greater pace (increased slope) as compared to that for Sections N3 and N4. For the 2015 research cycle, the IRI for Section S12 increased slightly, but at a lesser pace (lesser slope) as compared to the trend for Section S12 from the 2012 research cycle.

Figure 7 also shows linear trendlines fit to the ride quality data from each section. Similar to the rutting data shown in Figure 6, two separate trendlines were calculated to model the IRI for the period 0-10 million ESALs and 10-20 million ESALs. Figure 7 also shows the regression equation of the line that could be used to describe the data from each section. There are two equations for Section S12: the upper one shown in Figure 7 represents data from 0-10 million ESALs, and the lower one shown in Figure 7 represents data from 10-20 million ESALs. Figure 7 shows that Section N4 has the best ride quality performance in that it has the least slope and lowest IRI value compared to those of the other two sections. This finding is somewhat surprising given that this section has the thinnest cross section (when compared to Section N3) and weaker foundation support (when compared to Section S12). If one of the sections were to begin to show signs of advanced deterioration, it is expected that Section N4 would be the first section to do so.

Table 2 gives an indication of the expected service life in terms of ride quality for the three test sections. The regression equations shown in Figure 7 were rearranged to solve for the number of ESALs given an IRI value of 140 in per mile. VDOT classifies an interstate or primary roadway segment having an IRI greater than 140 in per mile as having “poor” ride quality (VDOT, 2016).

Table 1. Regressed Rutting Service Life

Section	Regression Equation	Million ESALs to 0.5 Inches Rutting
N3	$y = 0.0103x + 0.0906$	39.7
N4	$y = 0.0097x + 0.1057$	40.6
S12a ^a	$y = 0.0112x + 0.1051$	35.2
S12b ^b	$y = 0.0025x + 0.1258$	150.0

^a Regression equation for 0-10 million ESALs.

^b Regression equation for 10-20 million ESALs.

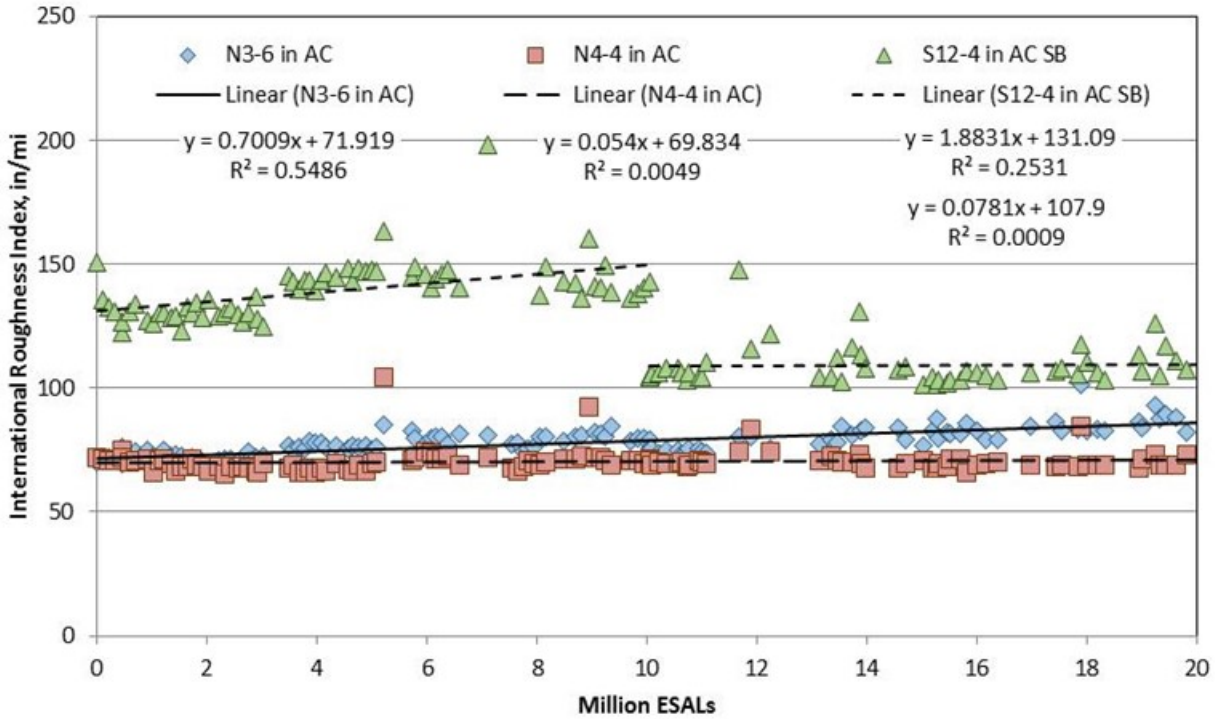


Figure 7. Ride Quality Results. AC = asphalt concrete; SB = stabilized base.

Table 2. Regressed Ride Quality Service Life

Section	Regression Equation	Million ESALs to 140 Inches per Mile
N3	$y = 0.7009x + 71.919$	97.1
N4	$y = 0.054x + 69.834$	1,299
S12a ^a	$y = 1.8831x + 131.09$	4.73
S12b ^b	$y = 0.0781x + 107.9$	411.0

^a Regression equation for 0-10 million ESALs.

^b Regression equation for 10-20 million ESALs.

From Table 2 it can be seen that the expected ride quality service life of the three sections as originally constructed is between approximately 5 million and 1.3 billion ESALs. The modification to the initial portion of Section S12 improved the projected ride quality service life from approximately 5 million to approximately 411 million ESALs. It is likely that the actual future IRI of these sections could be greater than that shown in Table 2 once other deterioration mechanisms begin.

If an in-service pavement were undergoing deterioration resulting in a high IRI value, potential maintenance strategies would vary, depending on other evident distress mechanisms, from thin overlay to mill and overlay to a more substantial structural overlay if needed. However, these options would not be selected based on IRI alone.

Deflection Testing

Periodic testing with a falling weight deflectometer (FWD) was used to assess structural changes in the test sections. Figures 8 and 9 show the averaged deflections measured at the load plate and at a distance of 72 in away from the load plate, respectively. The data points shown in Figures 8 and 9 were produced by first normalizing each deflection measurement to a 9,000 lbf load level and then averaging triplicate measurements at four locations along the outside wheel path of each section. The data shown were not temperature normalized, and so any changes attributable to temperature fluctuations remain.

In addition to the averaged deflection values, Figures 8 and 9 use a linear trendline to show the deflection data trend over the two test cycles. In Figure 8, the slope of the trendline for Sections N3 (6-in overlay with non-stabilized aggregate base) and N4 (4-in overlay with non-stabilized aggregate base) indicates that the deflection at the load plate (D_0) is increasing slightly with respect to time. The slope of the trendline for Section S12 (4-in overlay on a stabilized base) indicates that the deflection at the load plate (D_0) is decreasing slightly with respect to time. In Figure 9, the slope of the trendline for all three sections is so small that the deflection at a distance of 72 in from the load plate (D_9) is essentially constant with respect to time.

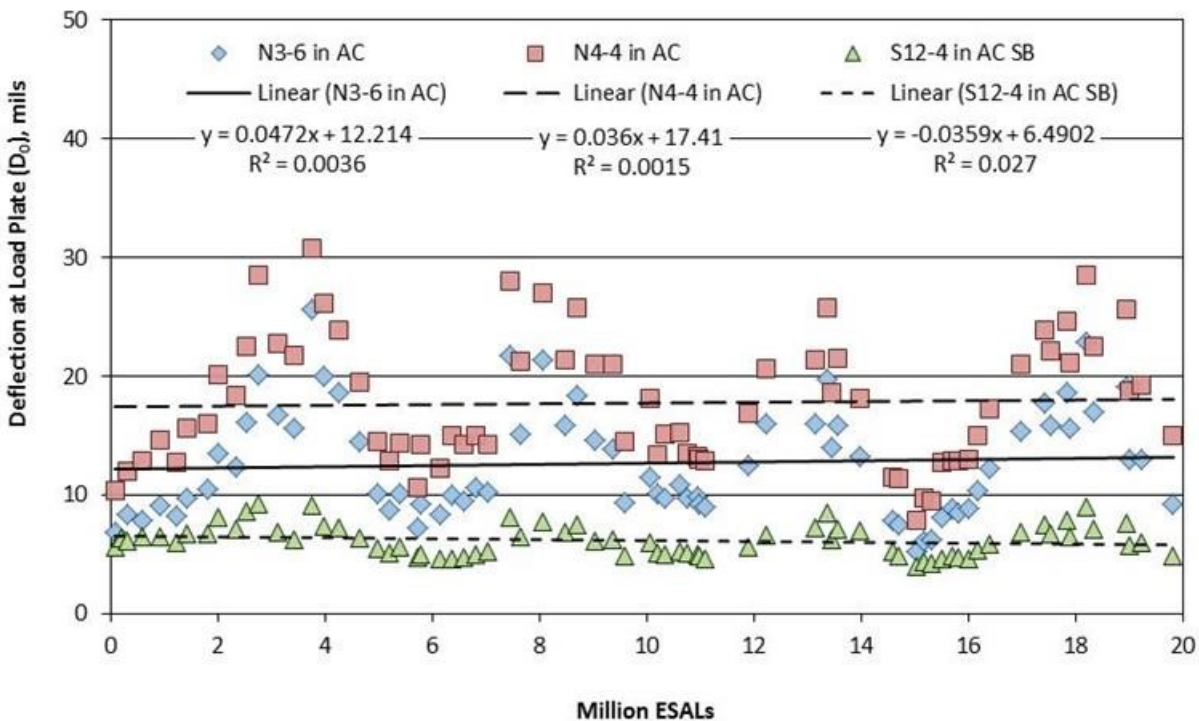


Figure 8. Averaged Deflection at the Load Plate (D_0). AC = asphalt concrete; SB = stabilized base.

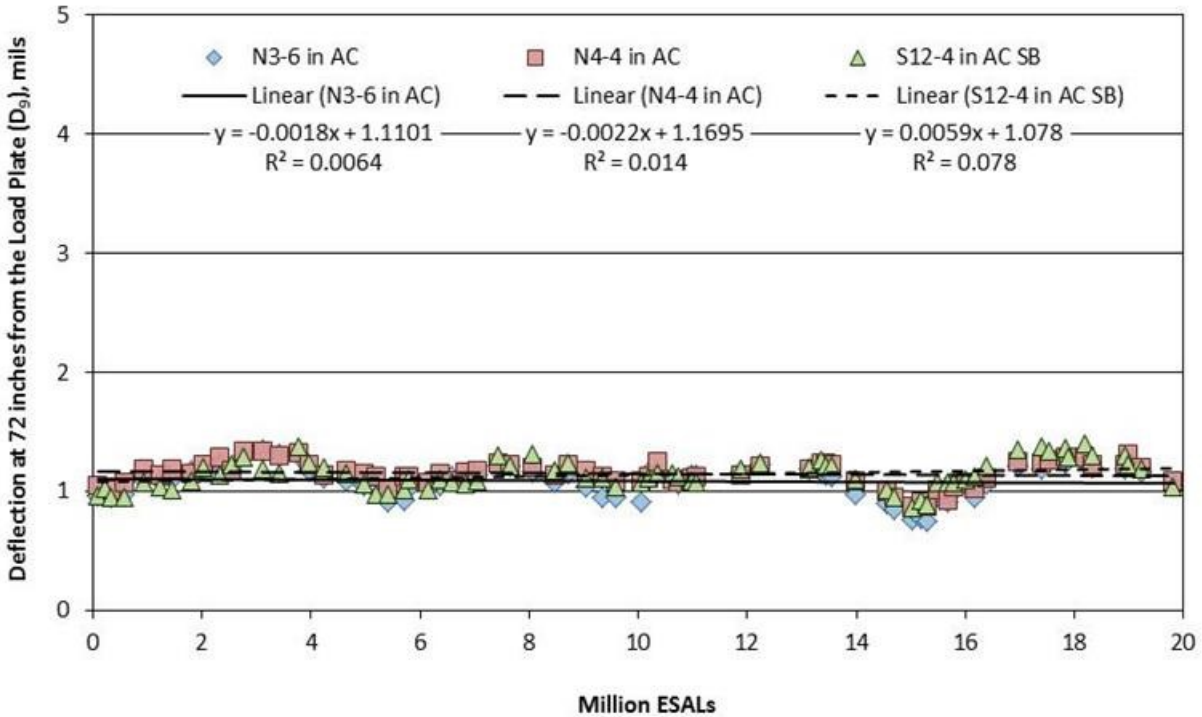


Figure 9. Averaged Deflection at a Distance of 72 Inches From the Load Plate (D_9). AC = asphalt concrete; SB = stabilized base.

Elastic Modulus of Asphalt/CCPR Layers

The elastic modulus of the asphalt/CCPR layers, as calculated from FWD measurements, is shown in Figure 10. For this study, all the asphalt layers (including the CCPR layer) were grouped together and treated as a single layer because of their similar viscoelastic behavior. The data presented in Figure 16 are normalized with respect to a temperature of 68°F. Despite the normalization process, the influence of seasonal temperature fluctuations is seen in Figure 10.

The elastic modulus for Sections N3 and N4 is similar but appears to be significantly less than the elastic modulus for Section S12. Since the asphalt and CCPR materials are the same for the three sections and were produced and placed at the same time for the three sections, the researchers do not expect significant differences in the material properties. It is possible that the FWD analysis is attributing some of the stiffness of the cement-stabilized foundation to the asphalt materials in Section S12. Over the two research cycles, the elastic modulus decreased slightly for Sections N3 and N4 and increased slightly for Section S12 with respect to ESAL loading. The increase in modulus is thought to be attributed to further hydration of the cement in the cement-stabilized foundation.

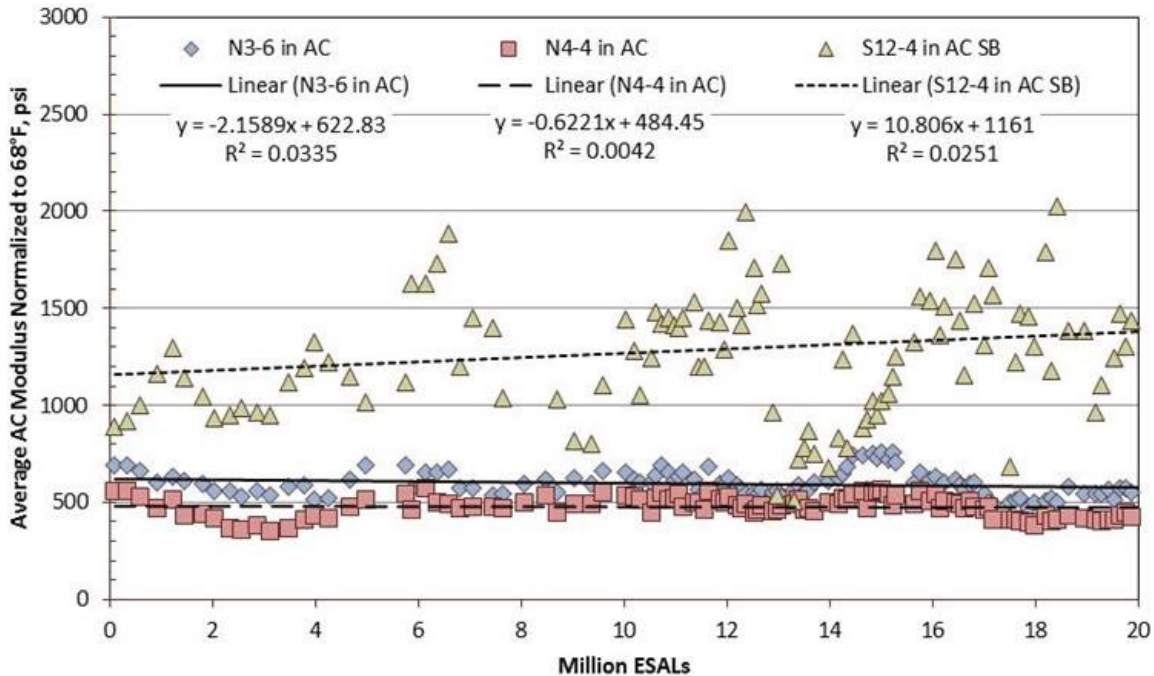


Figure 10. Temperature Normalized Modulus Results for Asphalt/CCPR Layers. AC = asphalt concrete; SB = stabilized base.

Instrumentation

Instruments were installed in the three test sections to monitor the pavement temperature in addition to the pressure and strain response from truck loading. These data are presented with respect to ESAL loading.

Horizontal Strain Response

Figure 11 shows the tensile strain response from trucking operations at the bottom of the CCPR layer with respect to temperature. The data shown in Figure 11 were obtained from strain gauges placed at the bottom of the CCPR layer in all three test sections. The strain response was well correlated to temperature, as shown by the exponential regression equations included in Figure 11. Sections N3 (6-in overlay with non-stabilized aggregate base) and N4 (4-in overlay with non-stabilized aggregate base) continued to have similar temperature sensitivity as demonstrated by the similar exponential coefficients in their respective regression equations.

Figure 11 also shows that Section S12 (4-in overlay on a stabilized base) had relatively less strain than the other sections and 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.

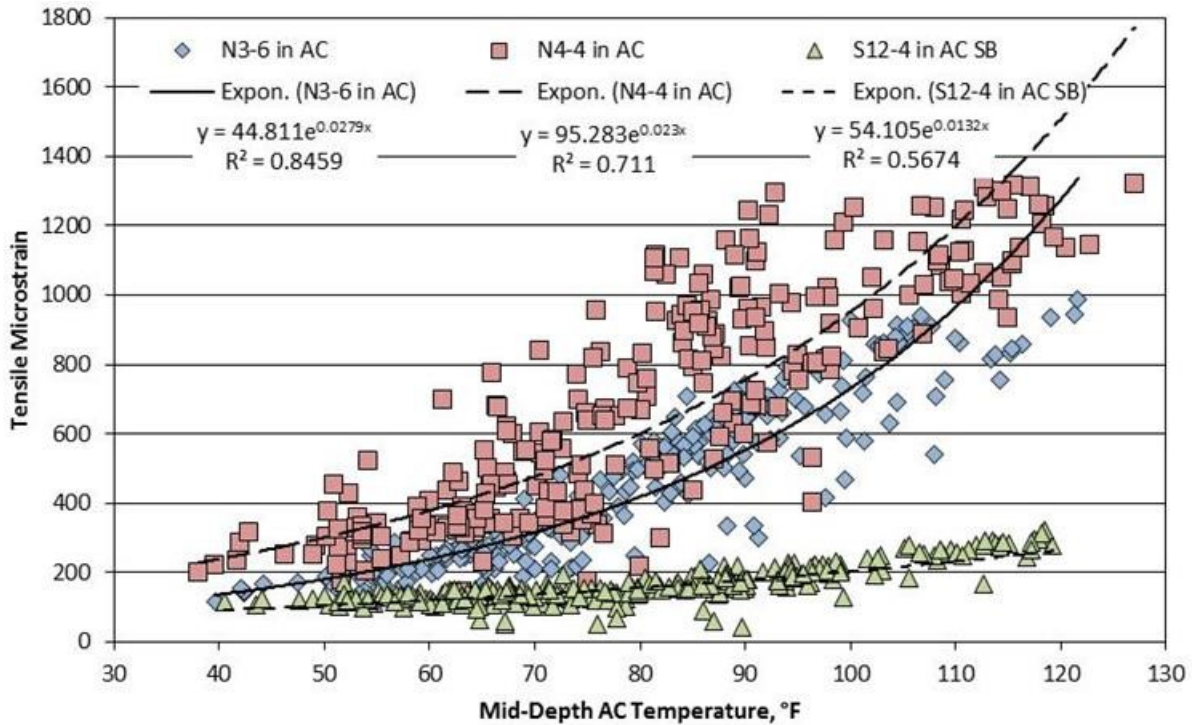


Figure 11. Tensile Strain at Bottom of CCPR Layer vs. Mid-Depth Temperature. CCPR = cold central plant recycling; Expon. = exponential trend line; AC = asphalt concrete; SB = stabilized base.

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 bottom-up 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. Although fatigue cracking had not occurred, it is possible that fatigue cracks will form in Section N4 before the other sections based on the trends in the strain data.

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 12 shows the strain response for the three recycled test sections normalized to 68°F. Viewing the data in this format makes it easier to identify trends such as potential deterioration within the test section. Based on the slope of the regressed strain, the data show that the strain response increased with respect to ESAL loading for Sections N3 and N4 whereas the strain response for Section S12 remained nearly constant over the two test cycles. The data also show that when Sections N3 and N4 were compared, the section having the thinner asphalt surface layers (Section N4) also had the greater strain, as expected. The strain data show that the inclusion of an extra 2 in of asphalt concrete to Section N3 reduced the strain by approximately 35% to 40% when compared to the strain measured in Section N4. The data also show that the strain reduction attributed to the cement-stabilized layer was approximately 65% to 75% when the measured strain from Section N4 was compared to that from Section S12 despite the two sections having the same asphalt surface layers and CCPR thickness. From this it would be expected that Section N3 would have a greater fatigue life than Section N4 and that Section S12 would have a much greater fatigue life than either Section N3 or N4.

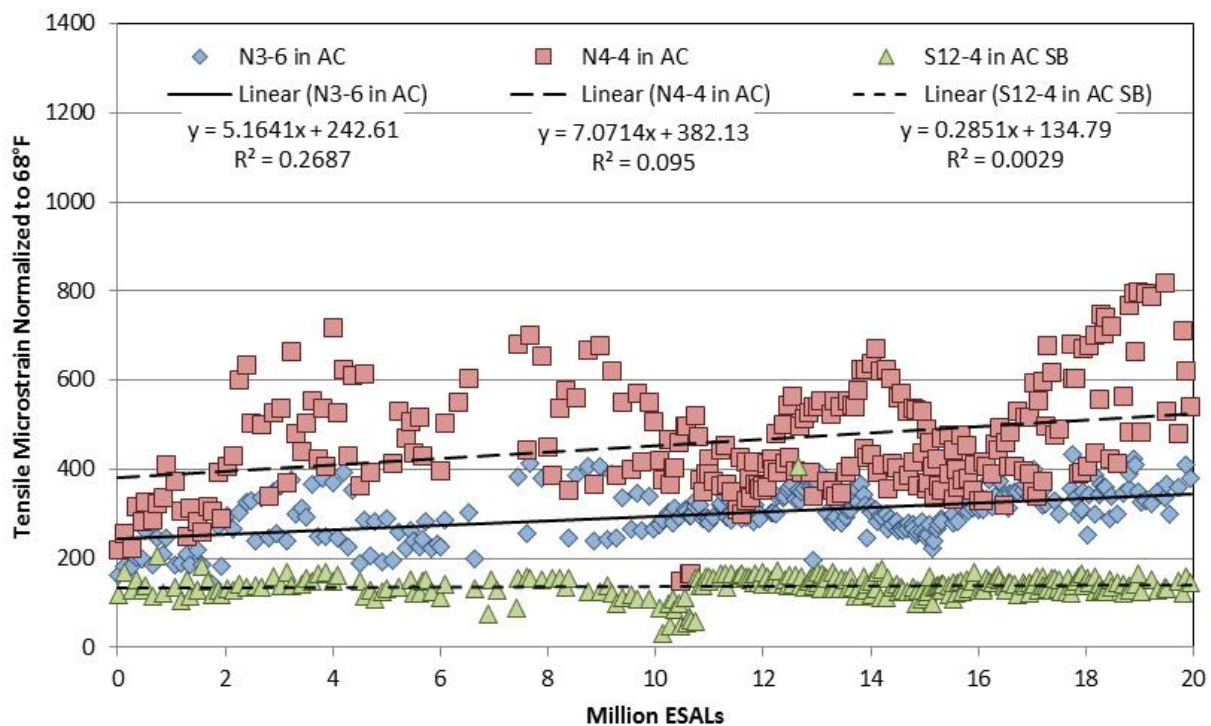


Figure 12. Tensile Strain at Bottom of CCPR Layer (normalized to 68°F) vs. Equivalent Single Axle Loads. CCPR = cold central plant recycling; AC = asphalt concrete; SB = stabilized base.

Table 3 expands the discussion on temperature normalized strain by comparing the strain at different traffic intervals to determine if any changes are occurring. Table 3 shows the results of significance testing to determine if the difference in mean strain is significant when the following traffic intervals are compared: 0-10 million ESALs versus 10-20 million ESALs, 0-5 million ESALs versus 5-10 million ESALs, 10-15 million ESALs versus 15-20 million ESALs, and 0-5 million ESALs versus 15-20 million ESALs. The values shown in Table 3 are the p-values obtained by using a Student t-test assuming unequal variances (after the assumption was verified using an F-test); values in bold type denote that the difference in mean temperature normalized strain values is statistically significant.

Table 3 shows that the difference in mean temperature normalized strain is statistically significant when Track Cycle 1 and Track Cycle 2 (0-10 versus 10-20 million ESALs) are compared for Section N3. The difference in mean temperature normalized strain is statistically significant when the first half of Track Cycle 1 and the second half of Track Cycle 1 (0-5 versus 5-10 million ESALs) are compared for Section N4. The difference in mean temperature normalized strain is statistically significant when the first half of Track Cycle 2 and the second half of Track Cycle 2 (10-15 versus 15-20 million ESALs) are compared for Sections N3 and N4. To compare the widest traffic loading range, the difference in mean temperature normalized strain is statistically significant when the first half of Track Cycle 1 and the second half of Track Cycle 2 (0-5 versus 15-20 million ESALs) are compared for Sections N3 and N4. Table 3 also supports the claim that the temperature normalized strain is not changing in Section S12 since none of the differences in mean strain was statistically significant.

Table 3. Probability That Difference in Mean Temperature Normalized Strain Is Non-Significant

Traffic Interval, Million ESALs	Section		
	N3	N4	S12
0-10 vs. 10-20	8.378x10⁻⁸	0.4609	0.3560
0-5 vs. 5-10	0.0561	0.0005	0.0791
10-15 vs. 15-20	6.770x10⁻⁵	0.0003	0.8980
0-5 vs. 15-20	4.500x10⁻⁹	0.0005	0.8825

Bold text denotes significant differences.

Vertical Base Pressure Response

Figure 13 shows the vertical pressure from trucking operations at the top of the base layer versus temperature; also shown are the regression equations for each section. From Figure 13 it was observed that there is a strong relationship with temperature for each section. As expected, Section S12 (4-in overlay on a stabilized base) had a lower base pressure than the other two sections, presumably from the higher modulus of the materials within the pavement structure. The base pressures measured in Sections N3 (6-in overlay with non-stabilized aggregate base) and N4 (4-in overlay with non-stabilized aggregate base) continue to show similar behavior. The expected reduction in base pressure from the additional asphalt layer thickness used in Section N3 was not observed.

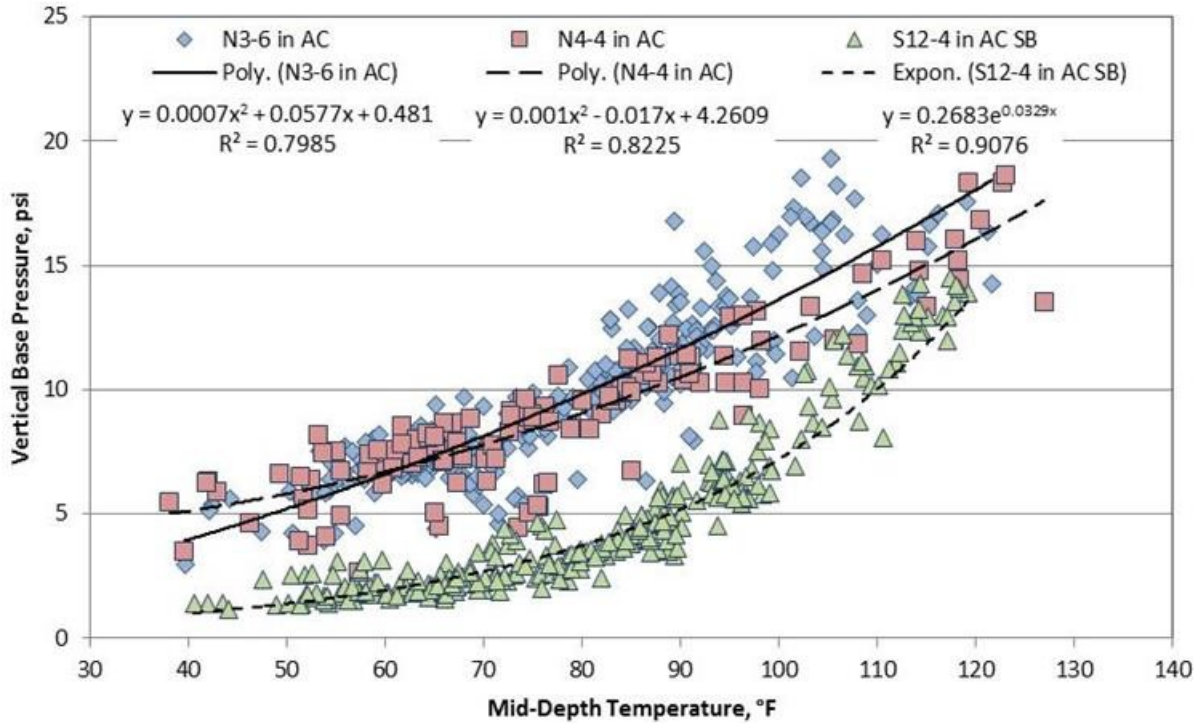


Figure 13. Vertical Base Pressure vs. Mid-Depth Temperature. AC = asphalt concrete; Poly. = polynomial trend line; SB = stabilized base.

Figure 14 shows the temperature normalized (normalized to 68°F) vertical base pressure versus ESALs and regression equations for each section. As observed in Figure 14, the base pressures measured in Sections N3 (6-in overlay with non-stabilized aggregate base) and N4 (4-in overlay with non-stabilized aggregate base) followed a similar trend; both increased with respect to ESAL loading, which may indicate some distress development not yet seen at the pavement surface. However, the base pressure in Section S12 (4-in overlay on a stabilized base) continued to decrease with respect to ESAL loading, presumably from continued curing of the stabilized base. The continued decrease in base pressure in Section S12 was somewhat surprising given that the sections were approximately 6 years old at the end of the second track cycle.

Vertical Subgrade Pressure Response

Figure 15 shows the pressure response for the three recycled sections from pressure sensors located on top of the subgrade. Similar to the trends observed with the vertical base pressure response, the subgrade pressure in Section S12 (4-in overlay on a stabilized base) had a lesser response than in the other two sections, presumably from the higher modulus of the materials within the pavement structure. The subgrade pressures measured in Sections N3 (6-in overlay with non-stabilized aggregate base) and N4 (4-in overlay with non-stabilized aggregate base) continued to show similar behavior. The expected reduction in subgrade pressure from the additional asphalt layer thickness used in Section N3 was not observed.

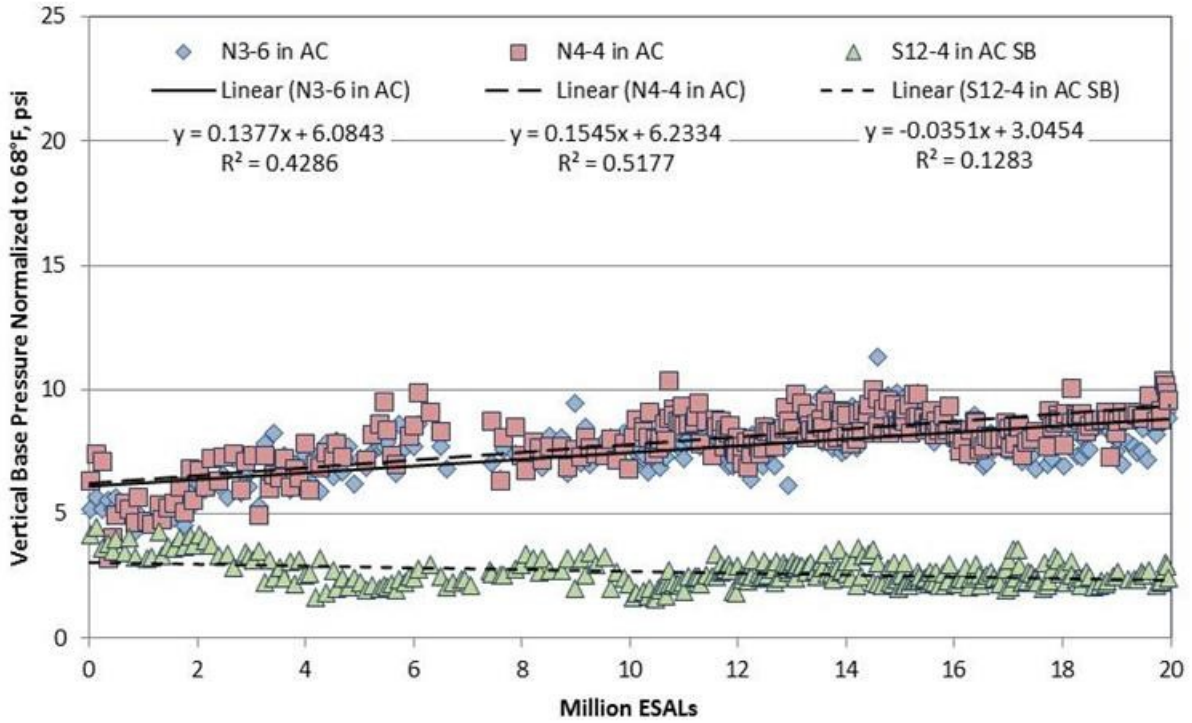


Figure 14. Vertical Base Pressure vs. Equivalent Single Axle Loads. AC = asphalt concrete; SB = stabilized base

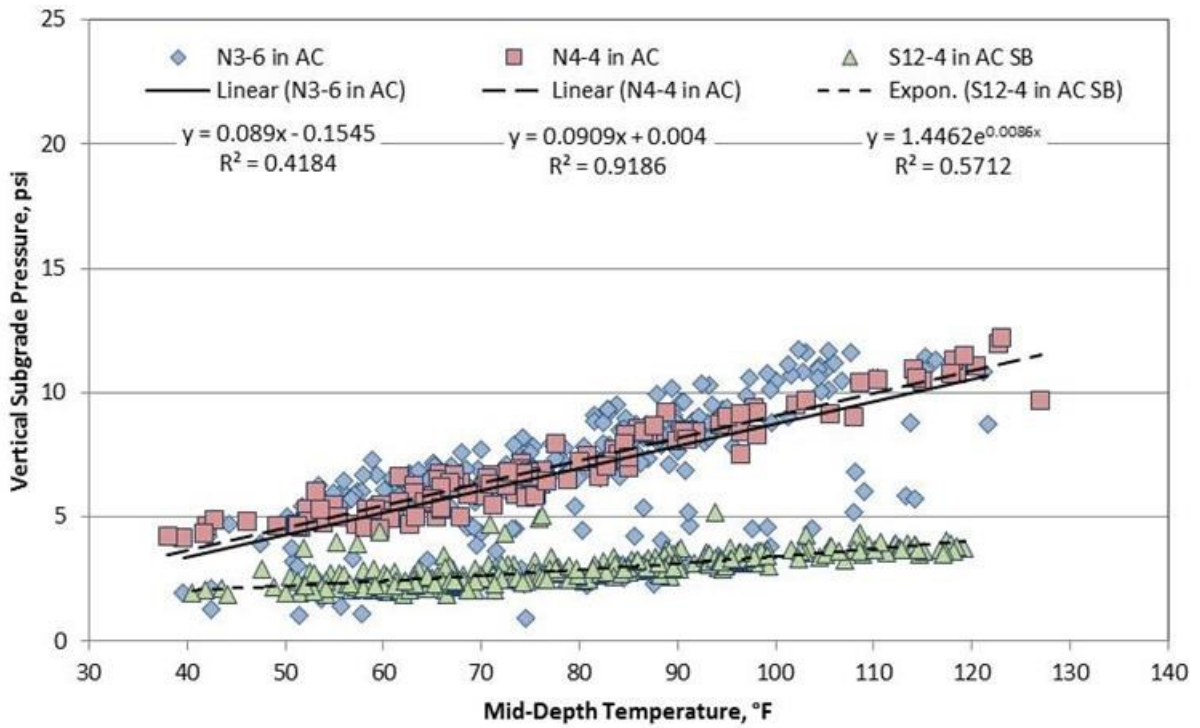


Figure 15. Vertical Subgrade Pressure vs. Mid-Depth Temperature. AC = asphalt concrete; SB = stabilized base.

Figure 16 shows the temperature normalized (normalized to 68°F) vertical subgrade pressure versus ESALs and regression equations for each section. As with the vertical base pressure, the vertical subgrade pressure in Section S12 (4-in overlay on a stabilized base) had a lesser subgrade pressure than in the other two sections over the range of applied ESAL loading. The response for Section N4 (4-in overlay with non-stabilized aggregate base) remained steady across the two track cycles, and the response for Section S12 decreased slightly, as evidenced by the low and negative slope coefficients for their regression equations, respectively. The response for Section N3 (6-in overlay with non-stabilized aggregate base) increased over the two track cycles. In particular, there appeared to be a jump in the response between the first track cycle (0-10 million ESALs) and the second track cycle (10-20 million ESALs). It is unclear if this was an artifact of the data collection process or an actual increase in subgrade pressure.

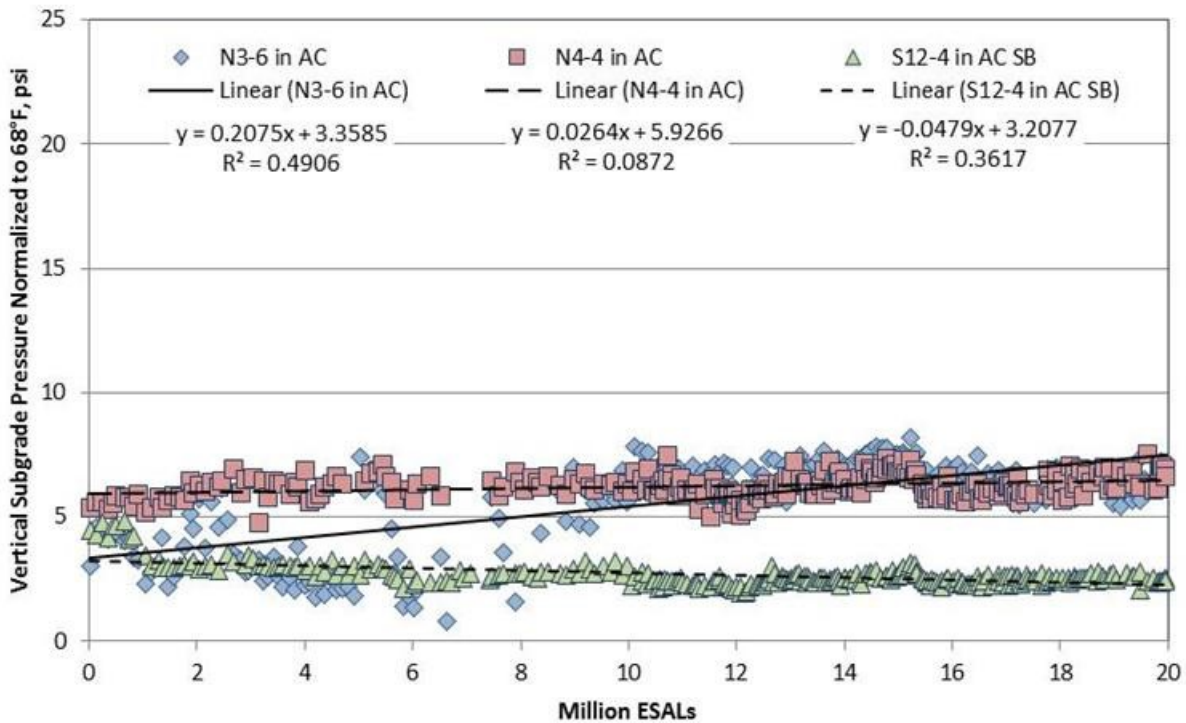


Figure 16. Vertical Subgrade Pressure vs. Equivalent Single Axle Loads. AC = asphalt concrete; SB = stabilized base.

SUMMARY OF FINDINGS

- The performance of the three recycled sections constructed at the NCAT Test Track in 2012 continued to be excellent after 20 million ESALs of traffic loading.
- No cracking was observed at the surface of any of the sections.
- The rutting performance of Sections N3 (6-in overlay with non-stabilized aggregate base) and N4 (4-in overlay with non-stabilized aggregate base) was similar.

- The rutting performance of Section S12 (4-in overlay with a stabilized base) differed between the two track cycles because of maintenance at the start of the test section. Following this, the slope of the linear trendline describing the change in rutting versus ESALs was less for Section S12 than for the other two sections.
- The additional 2 in of asphalt included in Section N3 (6-in overlay with non-stabilized aggregate base) did not yet improve the rutting performance when compared to that of Section N4 (4-in overlay with non-stabilized aggregate base). Based on the tensile strain reduction, the additional asphalt thickness is expected to influence the performance at higher traffic levels.
- The slope of the trendline describing the measured ride quality, expressed in terms of the IRI, was less for Section S12 (4-in overlay with a stabilized base) compared to that of the other two sections from 10 to 20 million ESALs. The ride quality of Section S12 also differed between the two track cycles because of maintenance at the start of the test section.
- The additional 2 in of asphalt included in Section N3 (6-in overlay with non-stabilized aggregate base) did not improve the ride quality when compared to that of Section N4 (4-in overlay with non-stabilized aggregate base).
- The temperature normalized modulus values of asphalt/CCPR layers decreased with respect to ESALs for Sections N3 (6-in overlay with non-stabilized aggregate base) and N4 (4-in overlay with non-stabilized aggregate base) but increased with respect to ESALs for Section S12 (4-in overlay with a stabilized base).
- The tensile strain measured at the bottom of the CCPR layer increased with increasing temperature for all three sections, suggesting that the CCPR material has viscoelastic properties. The variation in strain with respect to temperature in Section S12 (4-in overlay with a stabilized base) was much less than in the other two sections.
- The tensile strain measured at the bottom of the CCPR layer was greater for Section N4 (4-in overlay with non-stabilized aggregate base) when compared to that for Section N3 (6-in overlay with non-stabilized aggregate base). The measured tensile strain in Section S12 (4-in overlay with a stabilized base) was much less than in the other sections.
- The temperature normalized tensile strain measured at the bottom of the CCPR layer increased across the two track cycles for all three sections.
- The slope of the trendline describing the change in strain versus ESALs was greater for Section N4 (4-in overlay with non-stabilized aggregate base) when compared to that for Section N3 (6-in overlay with non-stabilized aggregate base). The slope of the trendline describing the change in strain versus ESALs was the least for Section S12 (4-in overlay with a stabilized base) when compared to those of the other sections.
- The vertical base pressure measured in Sections N3 (6-in overlay with non-stabilized aggregate base) and N4 (4-in overlay with non-stabilized aggregate base) was similar. The

expected reduction in base pressure from the additional asphalt layer thickness used in Section N3 was not observed. The vertical base pressure measured in Section S12 (4-in overlay on a stabilized base) was less than the pressure measured in the other two sections.

- The temperature normalized vertical base pressure increased in Sections N3 (6-in overlay with non-stabilized aggregate base) and N4 (4-in overlay with non-stabilized aggregate base) with respect to ESAL loading at a similar pace. The temperature normalized vertical base pressure in Section S12 (4-in overlay on a stabilized base) decreased with respect to ESAL loading.
- The vertical subgrade pressure measured in Sections N3 (6-in overlay with non-stabilized aggregate base) and N4 (4-in overlay with non-stabilized aggregate base) was similar. The vertical subgrade pressure measured in Section S12 (4-in overlay on a stabilized base) was less than the pressure measured in the other two sections.
- The temperature normalized vertical subgrade pressure remained steady across the two track cycles for Section N4 (4-in overlay with non-stabilized aggregate base). However, there was a discrete jump in the temperature normalized vertical subgrade pressure for Section N3 (6-in overlay with non-stabilized aggregate base) between the two track cycles (at about 10 million ESALs). It is unclear if this was an artifact of the data collection process or an actual increase in subgrade pressure.
- The temperature normalized vertical subgrade pressure in Section S12 (4-in overlay on a stabilized base) decreased with respect to ESAL loading.
- The additional 2 in of asphalt included in Section N3 (6-in overlay with non-stabilized aggregate base) did not result in a reduction in the vertical base pressure or vertical subgrade pressure when compared to that of Section N4 (4-in overlay with non-stabilized aggregate base).
- The inclusion of the cement-stabilized base layer in Section S12 resulted in a reduction in vertical base pressure and vertical subgrade pressure when compared to those of the other two sections.

CONCLUSIONS

- *The three test sections are examples of new or reconstructed pavement structures that can be constructed using a high percentage of recycled materials (ranging from approximately 32% to 80%) and can achieve a long service life. Although more data are needed, the current trends in the strain data and subsequent modeling indicate that Section S12 may be performing as a perpetual pavement. Continued trafficking and monitoring will provide a full evaluation of this assessment.*
- *Part of the increase in modulus values for the asphalt/CCPR layers 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 reduction in tensile strain measured at the bottom of the CCPR layer for Section N3 (6-in overlay with non-stabilized aggregate base) when compared to Section N4 (4-in overlay with non-stabilized aggregate base) is attributed to the additional 2 in of asphalt mixture placed on top of the CCPR. However, this reduction was much less than the reduction in tensile strain measured at the bottom of the CCPR layer attributed to use of the cement-stabilized base layer in Section S12.*
- *Given the lower tensile strain measured at the bottom of the CCPR layer and the higher modulus for the asphalt/CCPR layers for Section S12, this section is expected to have the best performance of the three recycled sections placed at the NCAT Test Track.*
- *The increase in modulus and reduction in vertical base pressure and vertical subgrade pressure over time in Section S12 is attributed to the continued curing of the cement-stabilized base layer.*
- *The distress and instrumentation data analyzed to date do not permit the development of distress prediction equations. The development of these relationships would be beneficial to agencies working to develop pavement recycling projects.*

RECOMMENDATIONS

1. *The Virginia Transportation Research Council (VTRC) should continue to sponsor trafficking of the recycled sections for the 2018 track cycle in an effort to develop distress prediction equations for recycled pavements. If VDOT intends to sponsor only two sections, Sections N4 and S12 should be included as they are the sections that are expected to represent the extremes in performance.*
2. *VDOT's Materials Division, VDOT's Maintenance Division, and VTRC should proactively work with VDOT districts to identify locations for future pavement recycling projects. Previous research identified pavement recycling as a means to reduce pavement rehabilitation costs and environmental impacts. This study showed that pavement sections built with recycled materials can have a long service life. By employing pavement recycling more often, VDOT and Virginia will also realize these savings.*

IMPLEMENTATION AND BENEFITS

Implementation

Recommendation 1 will be implemented by VTRC by sponsoring Sections N4 and S12 during the 2018 track cycle. This cycle will apply another 10 million ESALs to these sections, bringing the total to 30 million ESALs. The current status at 20 million ESALs is equivalent to

approximately 8, 10, and 20 years of traffic on the most heavily traveled portions of I-81, I-95, and I-64, respectively.

Recommendation 2 has already been partially implemented by the Hampton Roads District. Starting in 2016, Segment 2 of the I-64 widening project near Williamsburg was awarded with a recycling design similar to the one for Section S12 at the NCAT Test Track. This work is currently underway. In 2017, Segment 3 of the I-64 widening project was awarded with a similar recycling-based design. In 2018, VDOT's Materials Division developed a plan to assist VDOT districts with implementing at least one recycled project per district by the 2019 construction season.

Benefits

With regard to implementing Recommendation 1, the benefits of continuing to sponsor the recycled test sections at the NCAT Test Track include the ability to learn how long the pavement sections could serve VDOT in a real-world environment and to learn the failure mechanisms of the recycled materials once failure begins to occur. It is not known if the recycled layer will begin to crack or begin to lose stiffness and thus undergo a reduction in load carrying capacity.

With regard to implementing Recommendation 2, previous studies have shown there are significant cost and environmental benefits with using pavement recycling. The work described in this report provides evidence of excellent performance where high volumes of truck traffic are applied. By using recycling techniques, VDOT would experience these savings even when using the techniques on high-volume pavement sections.

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