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Assessment of Composite Pavement in Virginia: A Trial Section on US 60

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The Strategic Highway Research Program 2 (SHRP2) identified composite pavement as a “renewal solution” to support for implementation, and the Virginia Department of Transportation (VDOT) received funding to demonstrate its potential. In 2017, this funding was applied to support major rehabilitation of two westbound lanes of US 60 in Henrico County, Virginia, a project that essentially replaced 1.1 miles of deteriorated concrete pavement with a new composite system consisting of continuously reinforced concrete pavement (CRCP) overlaid with stone matrix asphalt (SMA).

This new composite pavement was designed in accordance with the 1993 AASHTO *Guide for Design of Pavement Structures* and was constructed in accordance with VDOT specifications and standards existing at the time. During construction, material properties were characterized to enable mechanistic-empirical (ME) analysis, and AASHTOWare Pavement ME Design software was then used to analyze the pavement again using the “asphalt concrete overlay over CRCP” option as suggested in the SHRP2 research. Because of the low truck traffic count on US 60, the predicted distresses for a 30-year design life were found to be very low compared to an analysis that uses the Pavement ME Design software default criteria. Through-the-thickness temperature changes were also monitored and it was found that the asphalt overlay provides an insulating effect for the underlying concrete, hence reducing the curling and thermal stresses in the concrete pavement.

SHRP2 researchers suggested that the thickness of the concrete portion of a composite pavement could usually be 1 to 3 in less than that of a plain (bare) concrete for comparable performance. Similar trends were observed for a higher truck traffic scenario in this study when a composite pavement (CRCP overlaid with SMA) was analyzed using the Pavement ME Design software.

VDOT maintains more than 500 lane-miles of CRCP that has been overlaid with asphalt at an average age of 26 years. These pavements, now considered “composite” pavements, are still in service, often after multiple asphalt mill and replace operations, with some as old as 52 years. The average life of these overlays is 10 to 15 years, with the combination of CRCP and SMA often providing 16 to 23 years per cycle.

The main distress mechanisms in a composite pavement are reflective cracking and rutting. The natural cracking and rut resistance of SMA therefore make it an ideal option for the asphalt component of a composite system. Such a design will protect the concrete base before any distresses have developed while also moderating thermal stresses (the insulating effect). The prospects for superior long-term service with low maintenance costs are excellent.

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FINAL REPORT

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INTRODUCTION

The Strategic Highway Research Program 2 (SHRP2) included four focus areas, one of which was infrastructure renewal. The SHRP2 R21 project, Composite Pavement Systems, identified new composite pavement systems as one of the "renewal solutions." Composite pavements have been used in Europe and to a limited extent in the United States (Rao et al., 2013) where a relatively thin functional top layer of high-quality asphalt or concrete is bonded to a lower layer of concrete materials of lesser quality. Two types of composite pavement systems were recommended as part of the SHRP2 R21 project. These included (1) surfacing a new portland cement concrete (PCC) layer with a high-quality asphalt layer(s), and (2) placing a relatively thin, high-quality PCC surface wet-on-wet atop a thicker, less expensive (lower quality) PCC layer. Composite pavements can be designed and constructed as strong, durable, safe, smooth, and quiet pavement with a minimal need for structural maintenance over the design life (Rao et al., 2013).

The Virginia Department of Transportation (VDOT) has traditionally created composite pavements through rehabilitation strategies that cap old concrete pavements with an asphalt overlay. McGhee and Clark (2007) looked at the performance of different surface mixtures on different pavement types using VDOT's windshield-based condition survey data from 2006 and found that SMA overlays on existing continuously reinforced concrete pavements (CRCPs) typically outperformed any other asphalt plant mixture application. Recently, network condition state data from automated distress surveys were used to support a similar comparison and were presented in a peer exchange on composite pavement organized by the Federal Highway Administration (FHWA) and VDOT (Habib, 2017). This comparison likewise suggested longer service-life prospects for asphalt materials when the base was CRCP.

In 2009, the Virginia Transportation Research Council (VTRC) published *Composite Pavement Systems: Synthesis of Design and Construction Practices* (Flintsch et al., 2009). The authors reported that composite pavement with a CRCP base may be cost-effective for very high volumes of traffic. The authors also reported that composite systems are potentially prone to distresses, especially those reflective of the underlying base state (e.g., reflective transverse cracking over jointed concrete pavement) and rutting within the asphalt layer. Premium asphalt surfaces such as SMA and/or reflective cracking mitigation techniques such as saw and seal, interlayers, asphalt binder modification, etc., may be required to mitigate these potential problems.

It is hypothesized that if new composite pavements consisting of new concrete pavement overlaid with a new asphalt concrete (AC) layer are constructed, the underlying concrete will never deteriorate to the extent that a complete pavement removal and replacement are needed; only scheduled maintenance activities to remove and replace the upper asphalt layers will be needed. VDOT received funding as part of the SHRP2 R21 project to demonstrate the potential of a new composite pavement. In 2017, this funding was applied to support major rehabilitation of two westbound lanes of US 60 in Henrico County, a project that essentially replaced 1.1 miles of deteriorated concrete pavement with a new composite system.

PURPOSE AND SCOPE

The purpose of this study was to document the design and construction of a new composite pavement section. This report includes challenges and lessons learned and a discussion of costs and benefits unique to a new composite pavement. Early life performance was also monitored (visually) and summarized for the first 2 years after opening to traffic. The use of a mechanistic-empirical (ME) pavement design approach for a new composite pavement was evaluated. The maintenance need for a composite pavement was also assessed by synthesizing the performance record of old CRCP, which had been overlaid with asphalt and was acting as a composite system throughout the VDOT network.

METHODS

Overview

To achieve the study objectives, the following tasks were conducted:

1. Select a site to implement composite pavement technology (under the SHRP2 R21 project), and characterize the site for pavement design.

2. Design a composite pavement section using the American Association of State Highway and Transportation Officials (AASHTO) 1993 *Guide for Design of Pavement Structures* (AASHTO, 1993) (hereinafter "the 1993 AASHTO guide").

3. Document the construction of and lessons learned for a new composite pavement including material properties based on quality assurance (QA) data.

4. Characterize the materials for analysis in AASHTOWare Pavement ME Design software (hereinafter "Pavement ME Design"), which was developed based on the *Mechanistic-Empirical Pavement Design Guide* (MEPDG) (ARA, Inc., 2004).

5. Evaluate Pavement ME Design for composite pavement design.

6. Measure the initial performance of the composite pavement through visual distress observation, temperature monitoring, and performance assessment using Pavement ME Design.

7. Assess the maintenance activities needed for composite pavement based on the literature and VDOT's Pavement Management System (PMS) database.

8. Assess the costs and benefits of a composite pavement.

Site Selection and Characterization

The project is located near Elko, Virginia, in Henrico County under the jurisdiction of VDOT's Richmond District. The location is shown in Figure 1. A 1.1-mile section of US 60 Westbound at the intersection of I-295, just west of the bridge, was selected for the demonstration of composite pavement. According to VDOT's PMS, this two-lane highway was built in 1979 with 8 in of CRCP on top of 6 in of cement-treated aggregate (CTA). In 2015-16, it was in very poor condition and needed rehabilitation/reconstruction.

Site characterization was necessary to design the composite section. This investigation included visual observation of surface condition, a distress survey by digital video logging (VDOT's PMS), coring to verify the structure, dynamic cone penetrometer (DCP) measurement of the existing subgrade condition, and core strength of the CTA base.

Figure 1. Location of Composite Section on US 60W in VDOT's Richmond District

The DCP was used through the core holes to characterize the in-situ subgrade condition. DCP tests were performed in accordance with ASTM D6951: Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications (ASTM International [ASTM], 2017). Traffic count information was also collected from VDOT's published data.

Composite Pavement Design

Different methods are available for composite pavement design. This pavement was designed using the 1993 AASHTO guide, which was VDOT's general practice at the time of this project design in 2016. A CRCP was designed as new pavement over CTA, and a layer of SMA was chosen as an overlay.

Documentation of Construction and Lessons Learned

VTRC researchers were present on-site during the construction of the composite pavement section. They collected information regarding old pavement removal, base preparation, and CRCP construction: plant production, reinforcement installation, paver operation, curing, and installation of the SMA overlay. The researchers also collected concrete and SMA mixture/samples for material characterization. Construction sequencing and quantities, cold joint formation, and many other construction details were gathered from the field inspectors. Numerous site visits and discussions with field personnel led to the lessons learned.

Quality Assurance Results and Material Characterization

Construction QA was an integral part of the construction. It involved measurements of concrete compressive strength, permeability, pavement thickness, SMA mat density, and thickness. QA activities were executed and monitored by VDOT Richmond District personnel; all data were shared for the purpose of documentation in this study. Additional samples were collected and tested for material characterization that was used for analysis using Pavement ME Design. Hardened CRCP thicknesses were verified by taking 2-in cores after at least 5 days of field curing.

Fresh and Hardened Concrete Characterization

A-3 paving concrete with the mix design shown in Table 1 was used for CRCP construction. The strength requirement was 650 psi flexural strength at the opening to traffic.

Table 1. Concrete Mix Design

Material and Mixture Characteristic	Quantity (lb/yd^3)
Type II hydraulic cement	526
Fly ash	132 (20%)
Coarse aggregate (No. 57)	1,650
Fine aggregate (natural sand)	1,298
Water	241
Water–cementitious material ratio	0.37
Admixtures	Water reducer and air-entrainment
Air (%)	6 ± 2
Slump (in)	0-2

Samples of the fresh concrete mixture were collected from the paver, and several cylinders and beams were prepared per batch for laboratory testing. Cylinders and beams were moist cured in the laboratory and tested at different ages for compressive strength, splitting tensile strength, modulus of elasticity, and flexural strength of beams in accordance with the specified ASTM standards (ASTM, 2017); for permeability in accordance with Virginia Test Method (VTM) 112 (VDOT, 2020a); and for the coefficient of thermal expansion in accordance with the specified AASHTO standards (AASHTO, 2019):

- ASTM C39: Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens

- ASTM C496 / 496M: Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens

- ASTM C78: Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)

- ASTM C469: Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression

- VTM 112: Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration – (Physical Lab)

- AASHTO T 336: Standard Method of Test for Coefficient of Thermal Expansion of Hydraulic Cement Concrete.

SMA Mixture Characterization

SMA mixtures were collected from the plant and characterized at the VTRC laboratory for use in Pavement ME Design. SMA mixtures were produced in accordance with VDOT specifications for SMA mix design (VDOT, 2016). Table 2 presents the mix design for the SMA mixture, including aggregate size and type, fiber content, recycled asphalt pavement (RAP) content, and asphalt binder content. Volumetric analyses were performed for the mixture.

Table 2. SMA Mix Design Used on US 60

Material	SMA-12.5 (PG 64E-22)
Top size ¾ in	32%
Top size 1/2 in (No. 28)	46%
Filler	12%
Additives (fiber)	0.3%
Recycled asphalt pavement, -½ in	10%
Asphalt binder	7%
VCA_{DRC}	42.1%

SMA = stone matrix asphalt; VCA_{DRC} = voids in coarse aggregate in dry rodded condition.

Gyratory pills 150 mm in diameter were compacted to 75 gyrations for volumetric properties in accordance with VDOT specifications. Data collected included asphalt content for the mixture and gradations of aggregates; bulk and Rice specific gravities (G_{mb} and G_{mm}); voids in total mixture (VTM); voids in mineral aggregate (VMA); voids filled with asphalt (VFA); aggregate bulk and effective specific gravities (G_{sb} and G_{se}); dust/asphalt ratio; percent binder absorbed (P_{ba}); and effective binder content (P_{be}).

Laboratory Performance Tests for SMA Mixtures

Laboratory performance tests such as the dynamic modulus test, asphalt pavement analyzer (APA) rut test, Indirect Tensile Asphalt Cracking Test (IDEAL-CT) for cracking resistance, and Texas overlay test were conducted to gather inputs for pavement ME analysis and assess the quality of the SMA mixture for potential cracking and rutting resistance.

Dynamic Modulus Test

Dynamic modulus results are required input in Pavement ME Design. Dynamic modulus tests were performed using the Asphalt Mixture Performance Tester with a 25 to 100 kN loading capacity in accordance with AASHTO T 378 (AASHTO, 2019): Standard Method of Test for Determining the Dynamic Modulus and Flow Number for Asphalt Mixtures Using the Asphalt Mixture Performance Tester (AMPT). Tests were performed on specimens prepared from gyratory-compacted asphalt samples (100 mm in diameter by 150 mm deep). An air-void content of 5% ± 0.5% (based on field-obtained air voids) was obtained for each test specimen. Four testing temperatures ranging from 4.4 °C to 54 °C and six testing frequencies from 0.1 to 25 Hz were used. Tests were performed from the coldest to the warmest temperature. At each test temperature, the tests were performed from the highest to the lowest frequency. Load levels were selected in such a way that at each temperature-frequency combination, the applied strain was in the range of 75 to 100 microstrain. Tests were conducted in the uniaxial mode without confinement. Stress versus strain values were captured continuously and used to calculate dynamic modulus. The results at each temperature-frequency combination for each mixture type are reported for three replicate specimens.

APA Rut Test

The APA rut test was conducted in accordance with VTM 110 (VDOT, 2020b). APA rut tests were also conducted on gyratory-compacted specimens at a test temperature of 64 °C on specimens that used an applied load of 120 lb and a hose pressure of 120 psi. The loading wheel speed was 2 ft/sec, and about 135 min was required to complete 8,000 cycles of load application. The recorded rut depth results after 8,000 cycles of load applications included the left, right, and average rut depth of the three replicate gyratory specimens.

IDEAL-CT

The IDEAL-CT for cracking resistance was proposed by researchers at the Texas Transportation Institute (Zhou et al., 2017). According to Zhou et al. (2017), this test shows promise in relating to field performance, reasonable repeatability, and simplicity by requiring no cutting, drilling, gluing, or notching of the specimen. The IDEAL-CT is typically run at room temperature with cylindrical specimens 150 mm in diameter and 62 mm high with a loading rate of 50 mm/min. The test uses a gyratory pill compacted to 7% air voids that is placed in a Marshall load frame (or similar load frame) and loaded to failure in indirect tensile mode. The load-displacement curve is used to determine the CT index, a crack susceptibility indicator.

Texas Overlay Test

The Texas overlay test was performed to assess the susceptibility of each mixture to reflective cracking. The test was performed in accordance with TX-248-F-09 (Texas Department of Transportation [DOT], 2019) using a universal testing machine with a loading capacity of 25 to 100 kN. Testing was performed at a temperature of 25 ± 0.5 °C. Loading was applied for a total of 1,200 cycles or until a reduction of 93% or more of the maximum load was reached.

Binder Recovery and Grading

Asphalt binder properties are required inputs in Pavement ME Design. Extraction of binder from loose mixture was performed in accordance with AASHTO T 164, Quantitative Extraction of Asphalt Binder From Hot Mix Asphalt (HMA), Method A (AASHTO, 2019), using n-propyl bromide as the solvent. Binder was recovered from the solvent using the Rotavap recovery procedure specified in AASHTO T 319, Quantitative Extraction and Recovery of Asphalt Binder From Asphalt Mixtures (AASHTO, 2019). Virgin and extracted binder grading was performed in accordance with AASHTO M 320, Performance-Graded Asphalt Binder (AASHTO, 2019).

Evaluation of AASHTOWare Pavement ME Design

Pavement ME Design was used to analyze and predict performance for the composite pavement using the HMA overlay of CRCP option. Level 1 material inputs (project-specific material properties where actual construction materials were tested) were used for the analysis. Truck traffic distribution data were collected from the site after opening to traffic. Pavement ME analysis was completed using Pavement ME Design, Version 2.2. VDOT completed local calibration of the MEPDG distress models for asphalt and concrete pavements (Smith and Nair, 2016), and those coefficients were used for the analysis. Since regular maintenance activities could not be incorporated in the ME analysis, the predicted distresses such as rutting or the

International Roughness Index (IRI) at the end of design period may not match the actual performance; they will most likely be overpredicted.

In addition to site-specific truck traffic, ME analyses were conducted using higher truck traffic levels (5,000 and 8,000 annual average daily truck traffic [AADTT]), a higher design life (40 and 50 years), and statewide traffic distribution data based on VDOT's *AASHTOWare Pavement ME User Manual* (VDOT, 2017). For Pavement ME Design, VDOT currently uses a CRCP punchout criterion of 6 counts/mile (for a 30-year design life) and for asphalt rutting a criterion of 0.26 in (for a 15-year design life). Such analysis with high truck traffic was conducted to show the benefit of using an asphalt overlay compared to just (bare) CRCP.

Measurement of Initial Performance

The initial performance of the composite pavement system was measured for a period of up to 2 years with the following measures:

1. visual distress observations at the surface of concrete and asphalt (including data from VDOT's PMS)

2. monitoring of the temperature variation along the depth of the composite pavement system.

Temperature measurement sensors (Command Center Sensor 301006-X15) were installed in the concrete pavement at six locations at four different depths: 1, 3, 5, and 7 in into the concrete pavement (or from the top of the base layer); it is important to note that the thickness of the concrete was 8 in. Figure 2 shows the installation of the temperature sensors. All sensors were installed in the left lane about 2 ft inside the left edge of the pavement at distances of 300; 1,000; 1,300; 2,300; 4,150; and 5,100 ft from the I-295 bridge. Sensors were tied to the reinforcement as shown in Figure 2 before the concrete was placed. A fifth sensor was installed in the SMA layer at a depth of 1 in from the top after the pavement was opened to traffic. These sensors were capable of storing the temperature readings collected at 30-minute intervals for up to 6 weeks. Therefore, temperature data were downloaded every month. Only 6 months of data were collected as most sensors had run out of battery power within 6 months.

Air temperature was gathered from Weather Underground (n.d.) for the Richmond, Virginia, area. These temperatures were compared to the collected temperature data to investigate the insulating effect of the asphalt layer on the CRCP temperature change.

Actual material properties from the laboratory test and the traffic count were used to determine the performance of the pavement using Pavement ME Design.

Figure 2. Temperature Measurement Sensors Installation in CRCP. CRCP = continuously reinforced concrete pavement.

Maintenance Needs Assessment

The maintenance needs of a composite pavement were not readily available since the concept of such pavements is relatively new. Therefore, a maintenance needs assessment was performed based on a literature review and the historic data available from VDOT's PMS for presumed composite pavement sections. These sections of pavement were originally built as CRCP and later overlaid with asphalt at the end of the design life.

Cost and Benefit Assessment

The benefit of a new composite pavement was assessed based on the literature review and the performance of the existing composite pavements in the VDOT system. As mentioned previously, these composite sections comprised the old CRCP overlaid with asphalt after many years of service. Cost saving features of a composite pavement were also discussed. Some of the cost information for key (components) items was gathered from the US 60 project and statewide average unit costs as available from VDOT's Construction Division (VDOT, 2020c).

The factors affecting the relative cost and benefit of a composite pavement such as maintenance needs, life expectancy, and end-of-life (salvage) value are also discussed briefly.

RESULTS AND DISCUSSION

Site Selection and Characterization

In 2016, the existing 37-year-old CRCP pavement was in very poor condition before this rehabilitation. There were many punchouts and transverse and longitudinal cracks, as shown in

Figure 3. VDOT's PMS annually rates each section of highway through an automated digital video logging. Pavement condition ratings from zero (0) to 100 are calculated using a deduct value system for each type of distress based on these video images. A higher rating number represents better pavement condition. In 2015, this section had a Concrete Distress Rating (CDR) of 31 and a Concrete Punch Rating (CPR) of 42, both indicating a very poor condition requiring a significant rehabilitation and/or reconstruction.

In September 2015, VDOT's Materials Division conducted falling weight deflectometer (FWD) testing on both lanes and processed the data using MODTAG 4.3.0. The backcalculated subgrade modulus was an average of 196 psi/in.

Six cores were also collected through CTA base layers. Based on the results of core testing, average CRCP and CTA thicknesses were 8.5 in and 4.5 in, respectively. All six concrete cores were in good condition, but a small portion of the CTA cores were disintegrated; four CTA cores had at least 4.5 in intact, one was disintegrated, and the other had about 3 in intact. As noted previously, the PMS records had indicated that the CTA was 6 in thick at construction.

Twelve samples were collected for soil index properties and natural moisture content determination. Six samples were taken directly from underneath the cores, and six were taken from the shoulder area. Index properties are summarized in Table 3.

Figure 3. Surface Distresses on US 60W in September 2015

Table 3. Summary of Soil Index Properties

Sample ID	USCS (ASTM) Soil Classification	AASHTO Soil Classification	Liquid Limit	Plastic Limit	Plasticity Index	Natural Water Content (%)	% Passing No. 200 Sieve
9-181-15	Lean CLAY, with sand (CL)	A-7-6 (17)	45	19	26	29.6	71.8
9-207-15	Lean CLAY, with sand (CL)	A-7-6 (26)	49	18	31	23.7	83.9
9-294-15	Fat CLAY (CH)	A-7-6 (43)	65	22	43	24.1	90.8
9-295-15	Fat CLAY (CH)	A-7-6 (36)	60	23	37	24.4	89.2
9-296-15	Fat CLAY, with sand (CH)	A-7-6 (26)	52	21	31	20.6	82.4
9-297-15	Fat CLAY (CH)	A-7-6 (30)	51	19	32	20.6	87.9
9-298-15	Fat CLAY (CH)	A-7-6 (33)	58	24	34	23.9	88.4
9-299-15	Lean CLAY, with sand (CL)	A-7-6 (20)	42	18	24	17.8	84
9-300-15	Sandy fat CLAY (CH)	A-7-6 (21)	55	20	35	22.6	66
9-301-15	Lean CLAY, with sand (CL)	A-6 (16)	39	17	22	18.7	75.2
9-302-15	Fat CLAY (CH)	A-7-6 (32)	55	23	32	26.7	91
9-303-15	Lean CLAY, with sand (CL)	A-7-6 (17)	42	19	23	21.1	76.6

USCS = Unified Soil Classification System; AASHTO = American Association of State Highway and Transportation Officials.

There were two types of soils present: fat clay (CH) and lean clay (CL). Since large samples could not be collected through the cores, similar soils were combined and tested for resilient modulus in the laboratory; the results are presented in Table 4. For all soils except one, in-situ moisture content was higher than the optimum. Therefore, in-situ resilient modulus would be significantly lower than the laboratory test results.

The DCP index was calculated as penetrations per blow. The corresponding California bearing ratio (along with correction for values less than 10) and resilient modulus (Mr) were calculated in accordance with the recommendation of the MEPDG. The modulus of subgrade reaction (K) value was calculated using the formula K = Mr/19.4 as presented in the 1993 AASHTO guide. The average K value from the DCP testing was 201 psi/in for the first few inches of subgrade. Considering the presence of CH soil and some disintegration of CTA, additional contribution to the K value from CTA was ignored. A K value of 196 psi/in was used for the pavement design.

Table 4. Subgrade Resilient Modulus Test Results

Soil Type	Proctor Result	Resilient Modulus (psi) With Stresses at Confining 2 psi and Deviator 6 psi
CH (all samples combined)	MDD = 108.2 pcf OMC = 19.2%	17,320 psi
CL (all samples combined)	MDD = 107.1 pcf OMC = 19.2%	13,420 psi

MDD = maximum dry density; OMC = optimum moisture content from the standard Proctor test.

Composite Pavement Design

At the time of this project, VDOT was still using the 1993 AASHTO guide; hence, the composite pavement was designed using that guide and a two-step process was followed as explained here.

Pavement Design Using the 1993 AASHTO Guide

The 1993 AASHTO guide was used to design a composite pavement to replace the existing CRCP. The pavement was designed by VDOT's Materials Division. For this project, traffic data were retrieved from the 2015 VDOT daily traffic volume estimates including vehicle classification estimates, and historic data were obtained from VDOT's annual traffic data publications (VDOT, 2015). The initial traffic growth rate was calculated from historical traffic data for the years 2007 and 2014. The annual average daily traffic (AADT) and truck percentages did not change between the 2007 and 2014 data, resulting in a calculated growth rate of 0%. However, considering the recent and continued industrial development in this area, a growth rate of 1.0 was used in the calculation of design equivalent single axle loads. The design parameters are summarized in Table 5.

Table 5. Pavement Design Parameters for US 60W

Design Parameter	Selected Value
Design Life, years	30
AADT (Design Year 2015)	14,140
Single Axle Trucks	3
Tractor Trailers	3
Number of Lanes	2
Lane Distribution Factor	90
Directional Distribution	50
Growth Rate (assumed)	1
Design 18k ESALs	5,306,300
Initial Serviceability	4.5
Terminal Serviceability	2.9
PCC Modulus of Rupture, psi	650
Elastic Modulus of Slab, psi	4,000,000
Mean Effective K-Value, psi/in	196
Reliability Level, %	90
Overall Standard Deviation	0.39
Load Transfer Coefficient, J	2.6
Drainage Coefficient	1

AADT = annual average daily traffic; ESAL = equivalent single axle load; PCC = portland cement concrete.

Since there was no specific design procedure available for new composite pavement, a two-step approach was followed:

1. Design the composite pavement as CRCP.

2. Design an AC overlay while keeping the CRCP thickness fixed at a level lower than required in Step 1.

In this case, the required CRCP thickness was 8.42 in for a 30-year design. In Step 2, the AC overlay thickness was determined assuming an effective CRCP thickness as 8 in. The final recommended design was 8 in of CRCP overlaid with 2.0 in of SMA (VDOT SMA 12.5 (64E-22). An edge drain, VDOT UD-7, was also recommended for incorporation in the rehabilitation.

Documentation of Construction and Lessons Learned

Removal of Existing Pavement and Base Preparation

The existing concrete pavement was removed by crushing the concrete in place using a guillotine breaker and loading the broken pieces on a haul truck using a backhoe as shown in Figure 4. The expectation was to use saw cutting and lift big pieces for removal so the underlying layer would not be damaged.

Although very little disintegration of CTA was seen from preconstruction coring as discussed previously, substantial damage to the existing base was observed after concrete removal, as shown in Figure 5.

Figure 4. Crushed Concrete Being Removed


Figure 5. Rainwater Ponded on the Exposed CTA for Days and Damaged the Base. CTA = cement-treated aggregate.

The in-place crushing operation might have contributed to the damage of the existing CTA base. The haul truck carrying the broken pieces of concrete might also have contributed to the CTA damage as it was driven over the exposed CTA. A significant rain event during the demolition of the pavement and the movement of the loaded haul truck over the CTA base exacerbated the situation. The base was pumping and needed to be removed and replaced and/or stabilized to support construction activities.

In most areas, 15 in of materials (broken CTA base and soft subgrade) was removed except for a few hundred feet in the middle. The base was removed, replaced, and/or stabilized in the following different ways; some of the steps are shown in Figure 6:

1. Remove and replace 15 in of the existing base (broken CTA) and subgrade with 21A aggregate over geogrid. Treat (with cement) the top 12 in with a full-depth reclaimer.

2. Remove 15 in of the existing base (broken CTA) and subgrade, lime stabilize the underlying soft subgrade, and fill with compacted 21A aggregate.

3. Conduct full-depth reclamation of the existing base (broken CTA) and subgrade in place for 12 in using CalCement.

Figure 6. Base Removal, Replacement, and Stabilization

CRCP Paving Operation

The paving operation was conducted mostly at night. The fresh concrete properties are summarized in Table 6.

Reinforcements were installed in accordance with the VDOT Road and Bridge Standard Sheet PR-3 (hereinafter "the VDOT standard") for 8-in-thick CRCP. Longitudinal steel bars were No. 5 at 6-in centers on top of No. 4 transverse bars (at 36-in centers) on longitudinal chairs, as shown in Figure 7. The longitudinal bars were placed at the mid-depth of the slab with an area of 0.64% in accordance with the VDOT standard.

Table 6. Fresh Concrete Properties

Date and Batch	Slump (in)	Air Content (%)	Concrete Temperature (°F)	Air Temperature (°F)	Relative Humidity (%)	Wind Speed (mph)
Truck 1 (9-11-17)	2	6.2	71	61.3	70	2
Truck 2 (9-11-17)	2	6.0	75	63.6	65	1.2
Truck 2 (9-21-17)	1.5	7.1	79	71	81	0

Figure 7. Reinforcement for CRCP. CRCP = continuously reinforced concrete pavement.

Concrete was hauled for approximately 30 minutes in a dump truck and delivered on grade with a belt. A Wirtgen SP25i slipform paver, as shown in Figure 8, was used to pave one lane (12 ft wide) at a time. The placement rate was 1,000 to 1,500 lane-feet per night except for the first night of paving, September 7, when the rate was 200 lane-feet. The construction took place for several nights in 2017; the right lane was constructed on September 7, 10, 11, 13, 14, and 15; the left lane was constructed on September 21, 22, 24, and 25. There were three to five construction cold joints in each lane; a typical joint is shown in Figure 9. Although the VDOT standard calls for doubling the reinforcement at such joints, it was not done for this project because there are some indications in the literature that increased reinforcement may cause congestion and hinder consolidation. Table 7 shows the approximate location of these cold joints.

Figure 8. Paving Right Lane With a Slipform Paver

Figure 9. Construction Cold Joint With Regular Reinforcement

Table 7. Construction Joint Locations

Joint No.	Distance From I-295 Bridge Deck, ft	
	Right Lane	Left Lane
1	200	1,230
2	1,230	2,788
3	2,350	4,130
4	3,630	Not available
5	4,764	Not available

Surface texturing was provided by dragging burlap behind the paver, as shown in Figure 10. A waxed-based curing compound (Dayton Superior White Cure) was sprayed at a rate of 100 to 150 ft^2 per gallon to prevent any moisture loss.

Since one lane at a time was paved to accommodate construction access, tie bars were installed after the right lane was completed. Holes were drilled and the ties bars were secured with epoxy, as shown in Figure 11. The tie bars were No. 4 at 36-in centers as required in the VDOT standard.

Figure 10. Burlap Drag for Surface Texturing

Figure 11. Drilled and Epoxied Tie Bars to Right Lane

The east end of the CRCP was tied to the existing anchor lug as shown in Figure 12a, but the west end was terminated (Figure 12b) at the interface of the existing pavement without any special arrangement (e.g., no special tie-in or transition), as the instruction was not clear in the VDOT standard and old pavement did not have any treatment either. A revision of this standard might be needed.

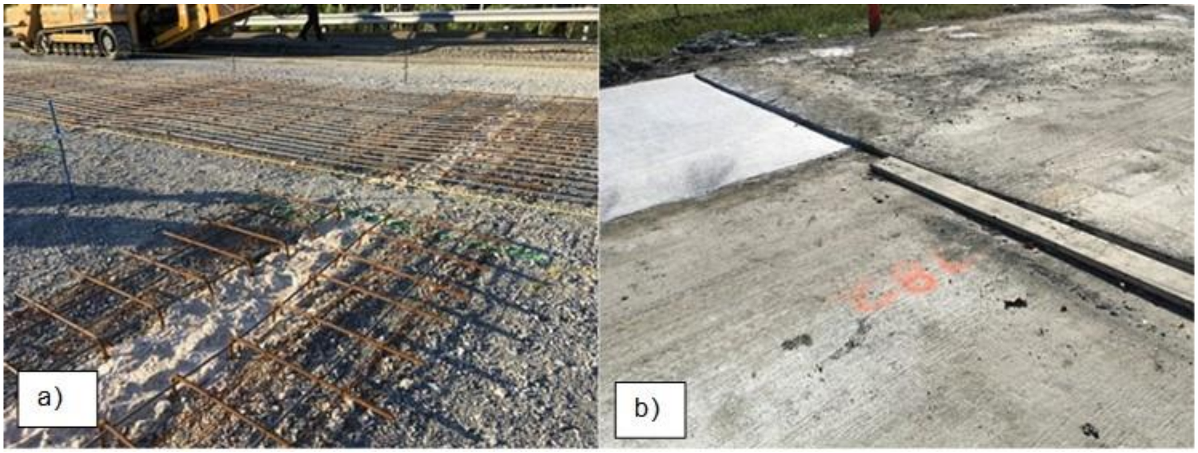

Figure 12. End Points of CRCP: (a) east end tied to anchor lug; (b) free end at west. CRCP = continuously reinforced concrete pavement.

Asphalt Overlay

All sections of CRCP were overlaid, as shown in Figure 13, 2 weeks after the last concrete placement with 2 in of SMA. Unlike with the concrete placement, the left lane was overlaid first, on October 13, 2017, and the right lane was overlaid on October 18, 2017. The VDOT standard construction practice for the SMA mixture was followed.

Figure 13. SMA Overlay Operation. SMA= stone matrix asphalt.

In order to ensure a good bond between the concrete surface and the SMA, a tack coat was applied, but no cleaning of the curing compound from the concrete surface was done. It is important to note that the concrete surface was textured using a burlap drag. No localized shoving or cracking of SMA was observed, indicating no apparent bonding problem after 2 years of traffic. The lane joint in SMA was offset about 3 in from the concrete lane joint. Although most SMA was 2 in, a small portion on the right edge toward the east end was 4 in to correct the superelevation.

As part of VDOT's regular QA operation for SMA, the laydown temperature was verified and a plug was extracted for density testing, as shown in Figure 14.

Figure 14. Checking SMA Laydown Temperature and Plug Extraction for Density Testing. SMA = stone matrix asphalt.

Quality Assurance Results and Material Characterization

As mentioned earlier, construction of both the CRCP and SMA was monitored by VDOT's Richmond District and VTRC. Regular QA samples were collected by the district, and extra samples were collected by VTRC for MEPDG material characterization. Tables 8 and 9 summarize the QA test results for concrete and SMA, respectively.

As mentioned, additional fresh concrete samples were collected from the delivery truck on two occasions (9/11/17 and 9/21/17) for laboratory testing to determine the properties needed for pavement ME analysis. Table 10 summarizes the properties.

SMA Mixture Properties for Analysis Using Pavement ME Design

Volumetric properties and the gradation analysis of field-sampled mixtures are shown in Tables 11 and 12. A quantitative method for ensuring stone-on-stone contact suggests limiting the voids in coarse aggregate (VCA) of the SMA mixture (VCA_{MIX}) to be less than the VCA in the dry-rodded condition (VCA_{DRC}). The SMA mixture used in this project met this criterion. The SMA mixture also met the VDOT volumetric and gradation requirements for SMA 12.5 mixtures.

Table 8. QC/QA Test Results for Concrete

Date Sampled	Air, %	Slump, in	Compressive Strength[a]		Permeability[b]	
			Age, days	psi	Age, days	C
9/07/2017	7.0	1.50	7	3,770		
			28	4,460	28	1,409
9/10/2017	4.0	1.00	8	4,200		
			30	5,580	30	1,431
9/11/2017	6.0	2.00	7	3,770		
			29	4,940	29	1,243
9/13/2017	5.5	2.00	7	3,730		
			28	5,110	28	1,206
9/14/2017	7.9	1.50	7	3,560		
			28	4,470	28	1,614
9/15/2017	6.2	2.00	28	4,480	28	1,272
9/21/2017	5.6	1.75	27	4,730	28	1,155
9/22/2017	7.1	2.00	28	4,180	28	1,269
9/24/2017	5	1.75	29	4,800	29	1,251
9/25/2017	7	1.75	28	4,390	28	1,266
9/27/2017	6.0	2.00	28	4,390	28	1,332
9/28/2017	7.6	2.00	28	4,650	28	1,345

QC/QA = quality control/quality assurance.
[a] Three-sample average.
[b] Average of two samples.

Table 9. SMA QA Test Results

Date Sampled	Lot No.	Core No.	Distance From I-295 Bridge, ft	Thickness, in	Air Voids. %
10/13/2017 (Left lane)	2	1	250	2.00	4.1
		2	1648	2.25	3.7
		3	2456	2.25	4.9
		4	3480	2.00	4.5
		5	4240	2.25	4.9
	3	1	301[a]	2.00	3.7
10/18/2017 (Right lane)	4	1	440	2.00	2.9
		2	1664	2.5	4.6
		3	2267	2.25	3.8
		4	3216	2.5	4.2
		5	4050	2.25	4.6
	5	1	147[a]	2.25	5.0
Average				2.21	4.24

SMA = stone matrix asphalt; QA = quality assurance.
[a] Distance from 0.05 miles west of Whiteside Road.

Table 10. Average Compressive, Splitting Tensile, and Beam Flexure Strengths

Measured Property	Age	No. of Samples	Average	Standard Deviation	Coefficient of Variation (%)
Cylinder compressive strength, psi	1 day	1	1,700	-	
	3 days	3	3,250	131	4.0
	7 days	3	3,630	90	2.5
	14 days	3	4,000	79	2.0
	28 days	3	4,700	47	1.0
	90 days	3	5,900	27	0.4
Cylinder modulus of elasticity, 10^6 psi	7 days	3	3.7	0.15	4.1
	14 days	3	4.1	0.17	4.2
	28 days	3	4.2	0.21	4.9
	90 days	1	4.5	-	-
Cylinder splitting tensile strength, psi	28 days	2	558	-	-
Beam flexure strength, psi	7 days	2	640	-	-
	14 days	2	670	-	-
	28 days	2	710	-	-
	56 days	2	750	-	-
	90 days	2	900	-	-
Permeability, C	28 days	2	720	-	-
Coefficient of thermal expansion, in/˚F	28 days	2	5.2	-	-
28-day drying shrinkage (length change, %)	28 days	3	0.04-0.05	-	-

- indicates not available.

Table 11. Volumetric Properties for SMA 12.5

Property		Criteria Min.	Criteria Max.
Asphalt content (%)	7.46	6.3%	
Air voids (V_a) (%)	3.5	2.0	4.0
VMA (%)	18.9	18.0 (design) 17.0 (production)	
VFA (%)	81.5		
VCA_{MIX}	40.2	<VCA_{DRC} (42.1%)	
Dust/asphalt content ratio	1.19		
Effective binder (P_{be}) (%)	6.87		
Effective film thickness (F_{be}) (microns)	12.1		

VMA = voids in mineral aggregate; VFA= voids filled with asphalt; VCA_{MIX} = voids in coarse aggregate of the SMA mixture; VCA_{DRC} = voids in coarse aggregate in dry-rodded condition.

Table 12. Gradation Results for SMA 12.5

Sieve	Criteria Min.	Criteria Max.	% Passing Average
$^3/_4$ in (19.0 mm)	100	-	100.0
$^1/_2$ in (12.5 mm)	83	93	85.6
$^3/_8$ in (9.5 mm)	-	80	64.3
No. 4 (4.75 mm)	22	28	23.8
No. 8 (2.36 mm)	16	24	18.5
No. 16 (1.18 mm)	-	-	16.0
No. 30 (600 µm)	15	20	13.9
No. 50 (300 µm)	-	-	12.0
No. 100 (150 µm)	-	-	10.2
No. 200 (75 µm)	9	11	8.17

- indicates no requirement.

Dynamic Modulus Test Results

The dynamic modulus of the SMA mixture is the major input for Pavement ME Design. Dynamic modulus test results for 4.5% air voids (selected based on field air voids, unconfined testing) are shown in Figure 15.

Pavement ME Design uses dynamic modulus (E*) to compute critical responses for HMA materials. SMA mixture properties for pavement ME analysis are shown in Table 13. Since dynamic modulus is not tested at -10 ºC (which is a required input in pavement ME analysis), modulus values were estimated from master curves using the time-temperature superposition principle.

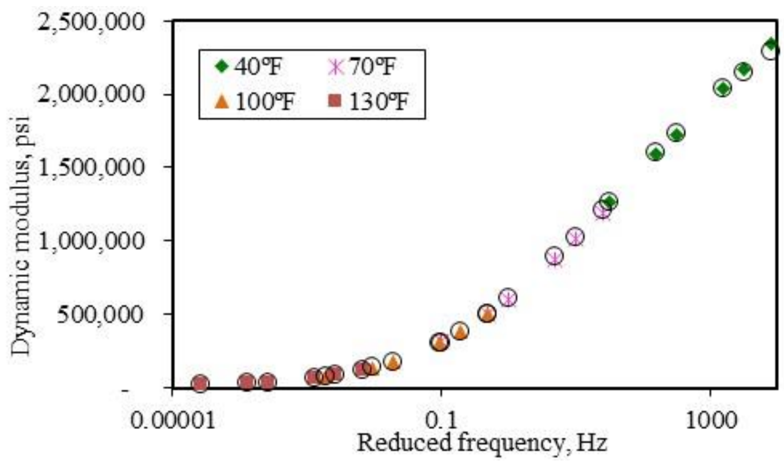

Figure 15. Dynamic Modulus of SMA Mixture (Semi-Log Scale). SMA = stone matrix asphalt.

Table 13. Dynamic Modulus Test Results for SMA

	Dynamic Modulus (E*)						
	Temp.	**25 Hz**	**10 Hz**	**5 Hz**	**1 Hz**	**0.5 Hz**	**0.1 Hz**
SMA 12.5E	-10 ºC	2,781,266	2,691,178	2,614,150	2,403,397	2,298,334	2,021,787
	4 ºC	2,323,554	2,174,114	2,051,449	1,739,002	1,595,417	1,253,876
	20 ºC	1,217,587	1,027,641	890,441	606,067	502,109	308,877
	38 ºC	496,549	379,206	304,971	177,002	138,341	77,130
	54.4 ºC	120,310	86,216	67,017	37,901	30,002	18,149

SMA = stone matrix asphalt.

APA Test Results

The average rut depth from the APA test was 4.2 mm at a test temperature of 64 ºC. This mixture is considered to be rut resistant based on previous research by Prowell et al. (2002) where a criterion of 4.0 mm was proposed for Virginia's SMA when tested at a temperature of 49 ºC. Since the current test was conducted at 64 ºC, the SMA is expected to rut much less than 4.0 mm at a lower temperature of 49 ºC as asphalt mixture becomes stiffer at a lower temperature.

Texas Overlay Test Results

Overlay test results for the SMA mixture are presented in Figure 16. According to the Texas DOT standard (Texas DOT, 2014) for thin overlay (½ to ¾ in), a value greater than 300 cycles indicates good crack resistance. Therefore, the number of cycles to failure for SMA mixtures of more than 600 with an average overlay cycle of 1,012 indicates good reflective crack resistance. In general, a higher overlay failure cycle indicates a higher reflective cracking resistance. Moreover, the SMA overlay is thicker than the Texas DOT thin overlay (2 vs. ¾ in) mentioned previously.

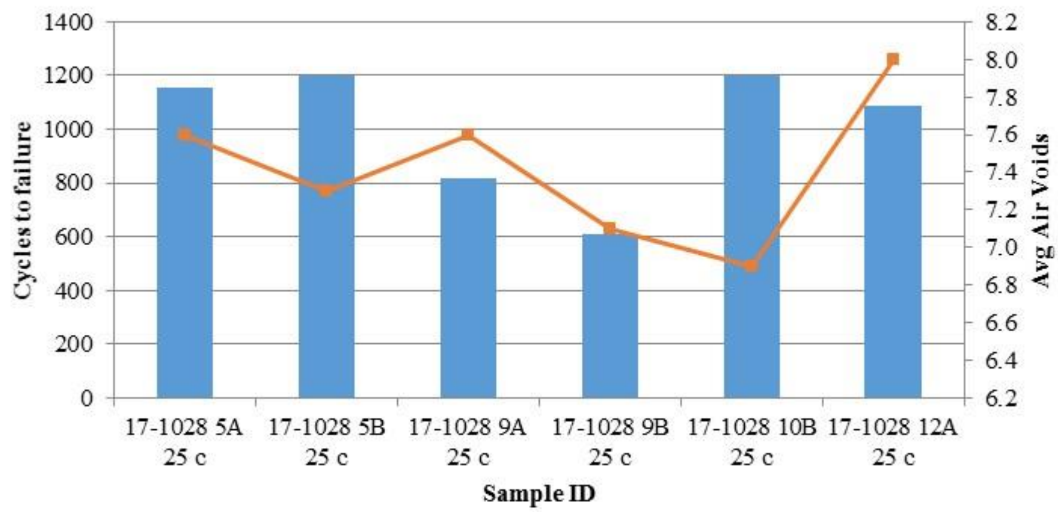

Figure 16. Texas Overlay Test Results. Bar and line indicate cycles to failure and average air voids, respectively.

IDEAL-CT: Cracking Test Index (CT index)

Higher CT index numbers indicate a higher ability of mixtures to resist cracking. VDOT regular dense-graded Superpave mixtures (SM 9.5 and 12.5) had an average CT index value of 80 in a previous study (Diefenderfer and Bowers, 2019). An average CT index value of 712 for the SMA mixture used in this project indicates higher cracking resistance. Results are shown in Figure 17.

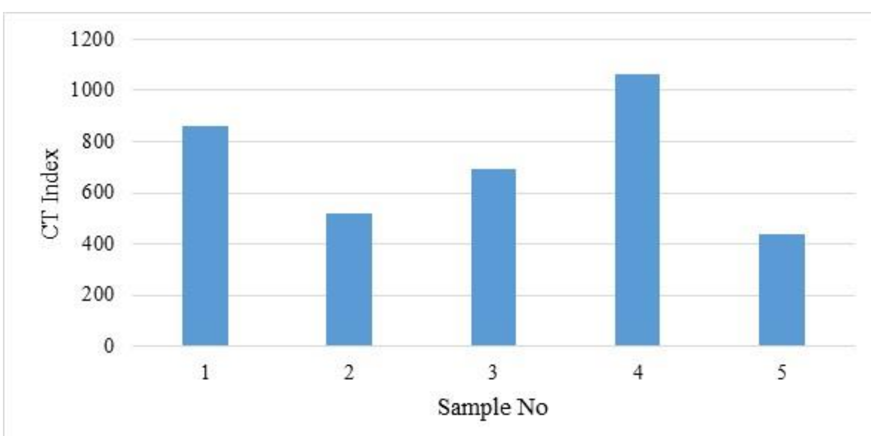

Figure 17. IDEAL-CT Results

Binder Test Results

Binder test results are needed as one of the inputs for analysis using Pavement ME Design. Binder test results are shown in Table 14. Binder testing confirmed a final PG of PG 64E-22 (PG 76-22 binder). Table 15 shows multiple stress and creep recovery (MSCR) test results. A higher percentage recovery (>35%) confirms the presence of polymer in the binder. A lower non-recoverable creep compliance (Jnr) value (<0.5) confirms the superior rut resistance of the asphalt binder.

Table 14. Binder Test Results

Dynamic Shear Modulus (G*), 10 rad/sec (AASHTO T 315), and Phase Angle				
RTFO G*	70 °C	4.781	Phase Angle	70.76
	76 °C	2.616		72.79
	82 °C	1.455		75.14

RTFO = rolling thin film oven.

Table 15. Multiple Stress Creep and Recovery (MSCR) Test Results

MSCR Test Results (AASHTO T 350)	
Test Temperature	64 °C
Avg. % Recovery R100Pa	51.29
Avg. % Recovery R3200Pa	36.89
% Difference	28.08
Non-Recoverable Jnr100Pa	0.3675
Non-Recoverable Jnr3200Pa	0.4915
% Jnr	33.77
Performance Grade	76-22

Jnr = non-recoverable creep compliance.

Analysis Using Pavement ME Design

US 60 Project

The actual materials properties and traffic count for the project (gathered after the construction) were used for analysis using Pavement ME Design. Actual traffic count data were gathered April 27, 2018, through May 4, 2018, after 6 months of opening to traffic subsequent to the construction of the new composite section. Pneumatic tubes at 8-ft spacings, as shown in Figure 18, were installed to collect the class and volume of traffic at two locations. Location 1 was between Dry Bridge Road and the I-295 ramp. Location 2 was 300 ft west of Whiteside Road. Table 16 summarizes the traffic data.

Analysis was completed with Pavement ME Design using the overlay (AC over CRCP) design option. In a SHRP2 report, Rao et al. (2013) also found that the design procedures in Pavement ME Design for HMA over CRCP were most comprehensive and applicable for the design of new composite pavements.

Figure 18. Traffic Data Collection Tubes on US 60W

Table 16. Average Daily Traffic Count (From 4/27/18 to 5/4/18) for US 60W

Class	Location 1 (Between Dry Bridge Rd and I-295 Ramp)				Location 2 (300 ft West of Whiteside Rd)			
	Travel Lane		Passing Lane		Travel Lane		Passing Lane	
	No. of Vehicles	%	No. of Vehicles	%	No. of Vehicles	%	No. of Vehicles	%
1	9	0.23	9	0.38	6	0.15	2	0.10
2	2,738	69.44	1,820	77.89	3,022	72.93	2,009	79.53
3	888	22.38	453	19.15	866	20.85	461	18.06
4	85	2.08	12	0.48	59	1.38	14	0.51
5	62	1.51	14	0.58	64	1.49	14	0.53
6	56	1.36	11	0.46	39	0.91	10	0.39
7	3	0.07	1	0.05	2	0.05	1	0.02
8	47	1.14	5	0.21	30	0.71	5	0.18
9	58	1.41	6	0.23	46	1.06	4	0.17
10	2	0.04	0	0.00	1	0.03	0	0.00
11	0	0.00	0	0.00	0	0.00	0	0.00
12	0	0.00	0	0.00	0	0.00	0	0.00
13	0	0.00	0	0.00	0	0.00	0	0.00
14	0	0.00	0	0.00	0	0.00	0	0.00
15	13	0.34	13	0.57	19	0.44	13	0.51
Total	3,961	100	2,344	100	4,155	100	2,533	100

The design inputs used for analysis are given in the following sections. Based on the data in Table 5, an AADTT of 848 (two direction) was used in the ME analysis. US 60 is a primary highway, and the AADTT was very low compared to the traffic observed on a typical VDOT interstate section. Site-specific traffic data were used to determine the truck class distributions, as shown in Table 17; only Classes 4 through 13 are considered in pavement ME analysis.

Table 17. Vehicle Class Distribution for Analysis Using Pavement ME Design

Truck Class	US 60W, Elko, Virginia (%)	VDOT Statewide Average (VDOT, 2017) (%)
Class 4	27	4
Class 5	19	5
Class 6	17	5
Class 7	1	1
Class 8	15	3
Class 9	20	77
Class 10	1	1
Class 11	0	4
Class 12	0	1
Class 13	0	0

From Table 17 it can be seen that the truck traffic distribution for the US 60 project is different than the statewide traffic distribution. VDOT statewide truck traffic distribution data show that Class 9 trucks are predominant (77%). However, for the US 60 project, Class 9 distribution was only 20%. Both the lower AADTT and Class 9 truck volumes will result in a lower distress prediction in Pavement ME Design. Inputs for axle load spectra and axles per truck were used in accordance with VDOT's *AASHTOWare Pavement ME User Manual* (VDOT, 2017).

A single weather station from near the project location, Richmond, was selected as the reference for climatic data. For concrete material properties, the 28-day modulus of rupture of 710 psi and the elastic modulus of $4.2*10^6$ psi were used. The average elastic modulus of the CTA layer for this project was determined to be $1.42*10^6$ psi from the laboratory test.

The structural layer thicknesses and design parameter for CRCP used in MEPDG analysis for the composite pavement are summarized in Tables 18 and 19, respectively.

Table 18. Layer Thicknesses of Composite Pavement on US 60

Layer Type	Material	Thickness (in)
Flexible overlay	Asphalt: SMA	2.0
Rigid CRCP	PCC	8.0
Stabilized base	Cement-treated aggregate	15.0
Subgrade	A-7-6	10.0
Subgrade	A-7-6	Semi-infinite

CRCP = continuously reinforced concrete pavement; SMA = stone matrix asphalt; PCC = portland cement concrete.

Table 19. CRCP Properties for Pavement ME Design

Property	Value
PCC surface shortwave absorptivity	0.85
Shoulder type	Asphalt
Permanent curl/warp effective temperature difference (ºF)	-10.00
Steel (%)	0.64
Bar diameter (in)	0.63
Steel depth (in)	3.50
Base/slab friction coefficient	8.90

PCC = portland cement concrete; CRCP = continuously reinforced concrete pavement.

The Pavement ME Design distress prediction summary is presented in Table 20, which shows that the composite section on US 60 met all VDOT distress criteria for punchouts for a new CRCP and rutting criteria for AC pavements and Pavement ME Design default criteria for some of the other distresses for the design life of 30 years. The individual distress trends with the number of service years are presented in Figure 19. Distress predictions for punchouts and rutting are very low for the 30-year design period. This might have been the result of the very low truck traffic count and distribution, as estimated by VDOT, in this section. Both laboratory rut tests (APA) on the mixture and MSCR tests on asphalt binders confirmed the superior rut resistance of the SMA mixture in the US 60 project.

Table 20. Summary Results of Analysis Using Pavement ME Design for 30-Year Design of US 60

Distress Type	Distress at Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	**Predicted**	**Target**	**Achieved**	
Terminal IRI (in/mi)	140.00	138.40	95.59	95.69	Pass
Permanent deformation, AC only (in)	0.25	0.09	95.00	100.00	Pass
AC bottom-up fatigue cracking (% lane area)	6.00	1.86	95.00	100.00	Pass
AC total transverse cracking: thermal + reflective (ft/mi)	2500.00	1061.87	95.00	100.00	Pass
AC thermal cracking (ft/mi)	1000.00	1.00	50.00	100.00	Pass
AC top-down fatigue cracking (ft/mi)	2000.00	329.20	95.00	100.00	Pass
CRCP punchouts (No./mi)	6.00	0.07	95.00	100.00	Pass
Chemically stabilized layer, fatigue fracture (% lane area)	25.00	0.26	-	-	-

IRI = International Roughness Index; AC = asphalt concrete; CRCP = continuously reinforced concrete pavement; - = not available.

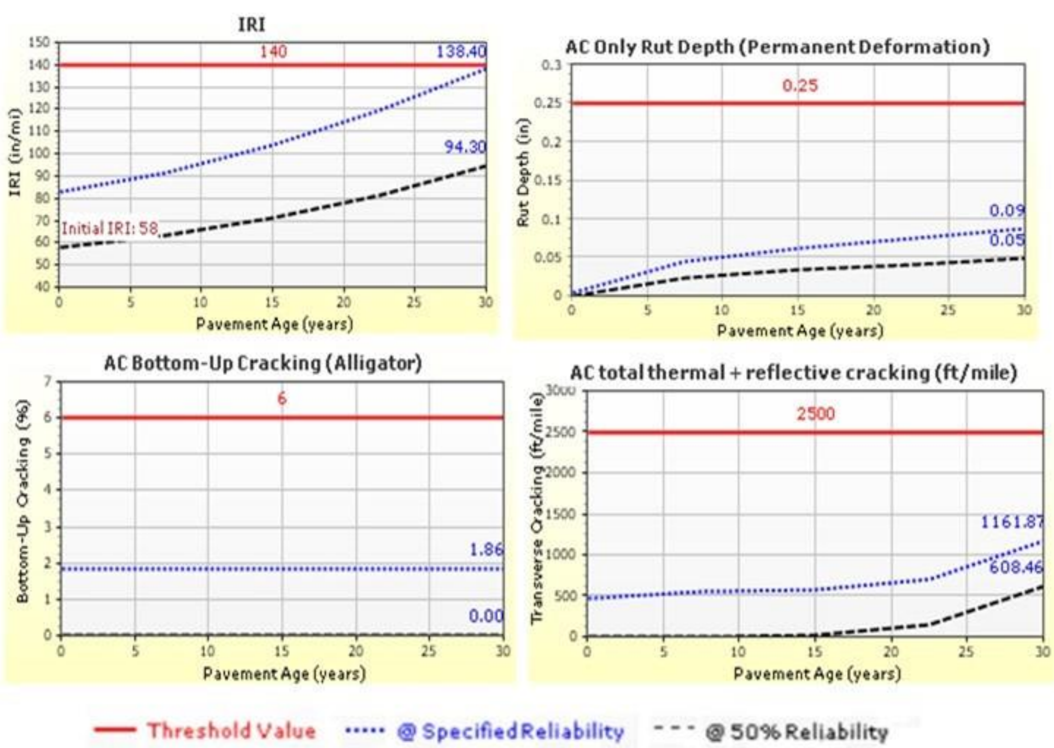

Figure 19. ME Predicted Distresses for US 60 Project With Age: (a) International Roughness Index (IRI); (b) rut depth; (c) alligator crack; (d) thermal crack. ME = mechanistic-empirical; AC = asphalt concrete.

In an SHRP2 report, Rao et al. (2013) explained the relative importance of different cracks in a composite pavement analysis. According to Rao et al., low-temperature thermal cracking is a minor issue and is not likely to occur in HMA/PCC composite pavements. The chances of surface-initiated cracks (AC top-down cracking) for composite pavements are also small because the tensile strains at the surface of the HMA layer are small. AC bottom-up cracking is also not a concern for composite pavements since it does not usually initiate at the bottom of the HMA layer for composite pavements because the HMA is almost always in compression unless there is a loss of friction between the HMA and concrete layers. Fatigue cracks in HMA/PCC composite pavements normally initiate at the bottom and top of the PCC layer and propagate to the surface with continued traffic applications. Composite pavement can develop fatigue-related distress in the form of punchouts, which should be an important design consideration (punchouts are also the main distress criteria for CRCP). Moreover, the concrete layer is the primary load-carrying component of the composite pavement system (AC over CRCP). When a heavy load is applied and repeated on a composite pavement, the HMA layer may undergo some permanent deformation, which is considered to be an important design criterion. The thickness of the HMA layer will affect rutting potential. Rutting and the number of punchouts are expected to increase when truck traffic increases.

The transverse and reflective crack prediction models in the version of Pavement ME Design used for this project analysis have been revised. The latest version of Pavement ME Design has incorporated the revised models, but it is not yet available for VDOT's use because it needs local calibration. Therefore, these distresses will not be considered in the subsequent analysis in this report. However, with adequate thickness and steel reinforcement (cracks in the CRCP are held tight by the reinforcing steel) in the CRCP, transverse cracks can be reduced as can the reflection of transverse cracks. Moreover, SMA mixtures rich in binder content and the use of polymer modified binder should make SMA perform well against reflective cracking. Hence, reflective cracking is not a big concern for AC over CRCP.

Based on the discussion, CRCP punchouts, AC rutting, and terminal IRI values are major design criteria for AC over composite CRCPs.

Pavement ME Analysis With Typical Interstate Traffic

Pavement ME analysis for a 30-year design was also conducted using statewide truck traffic distribution (Table 17) and higher truck traffic counts: 5,000 and 8,000 two-way AADTT. The pavement structure used for these analyses is similar to that of US 60. Results are summarized in Table 21. As expected, rutting and punchouts increased when truck traffic increased, indicating the sensitivity of the design to AADTT.

Table 21. Results of Pavement ME Analysis for Higher AADTT

Distress Type		Composite Pavement (Same as US 60)	
		AADTT: 5,000	**AADTT: 8,000**
Terminal IRI (in/mi)		156	159
Permanent deformation, AC only (in)	30 years	0.28	0.35
	15 years	0.18	0.22
CRCP punchouts (No./mi)		4.02	6.2

ME = mechanistic-empirical; AADTT = annual average daily truck traffic; IRI = International Roughness Index; AC = asphalt concrete; CRCP = continuously reinforced concrete pavement.

Further analyses using Pavement ME Design were conducted on a structure similar to that on US 60 with only 6 in instead of 12 in of CTA, which is the typical VDOT practice. A higher traffic growth (3%) and statewide traffic distribution and higher truck traffic (5,000 and 8,000 two-way AADTT) were used. Results were compared with a CRCP (no asphalt overlay, 8-in bare CRCP) in Table 22. It can be seen that the predicted punchouts for an 8-in CRCP was higher than that for a composite pavement for both traffic levels and do not meet the VDOT design criterion of six punchouts per mile. On the other hand, the punchouts for the composite pavement structure met the criteria for 5,000 AADTT but not for 8,000 AADTT.

A few more pavement ME analyses were conducted with several combinations of thickness in order to meet VDOT's punchout criteria, as presented in Table 22. The thickness for exposed (bare) CRCP needs to be increased to 9 in for an AADTT of 5,000 (punchouts reduced to 4.7) and to 10 in for an AADTT of 8,000 (punchouts reduced to 3.3). For a composite section with 8,000 AADTT, punchouts can be reduced either by increasing the CRCP thickness to 9 in (punchouts, 3.8) or by increasing the asphalt thickness to 3 in (punchouts, 4.3). However, rutting increased when a 3-in SMA layer was used, but the increase was minor over 15 years. Therefore, a 1-in and 2-in reduction in thickness for CRCP was observed when a 2-in SMA surface was used as a composite structure over CRCP for an AADTT of 5,000 and 8,000, respectively. This clearly shows that CRCP thickness can be reduced when composite pavement is used. Rao et al. (2013) stated that the HMA layer reduces temperature and moisture gradients in the PCC slab, which reduces slab curvature and related load and thermal stresses. They also suggested that the CRCP thickness can be reduced by 1 to 3 in depending on factors such as traffic, climate, and material properties when compared with a bare CRCP.

Table 22. Comparison of MEPDG-Predicted Distresses Between Composite Pavement and CRCP for a Design Life of 30 Years

	Layer Thickness, in		**Predicted Distress**			
Pavement Type	**CRCP**	**SMA**	**Terminal IRI (in/mi)**	**Rutting (in) (Asphalt Only), 15 Years**	**Punchouts (No./mi), 30 Years**	**Criteria Satisfied**
AADTT: 5,000						
CRCP	8	0	120	-	15.8	No
	9	0	102	-	4.7	Yes
	10	0	100	-	2.0	Yes
Composite	8	2	156	0.18	5.5	Yes
AADTT: 8,000						
CRCP	8	0	130	-	20.2	No
	9	0	105	-	6.8	No
	10	0	101	-	3.3	Yes
Composite	8	2	159	0.22	7.7	No
	8	3	165	0.31	4.3	No
	9	2	159	0.22	3.8	Yes

MEPDG = *Mechanistic-Empirical Pavement Design Guide;* CRCP = continuously reinforced concrete pavement; SMA = stone matrix asphalt; IRI = International Roughness Index; AADTT = annual average daily truck traffic; - = not applicable.

Additional analyses with the design lives of 40 and 50 years were also performed, and the results are presented in Tables 23 and 24, respectively. It can be seen that the analysis met the criteria for a higher design life except for a few cases. In general, the reduction in thickness of CRCP was observed in composite pavement even with a higher design life.

Table 23. Comparison of MEPDG-Predicted Distresses Between Composite Pavement and CRCP for a Design Life of 40 Years

Pavement Type	Layer Thickness (in)		Predicted Distress			
	CRCP	SMA	Terminal IRI (in/mi)	Rutting (in) (Asphalt Only), 15 year	Punchouts (No./mi), 40 year	Criteria Satisfied
AADTT: 5,000						
CRCP	8	0	130	-	20.4	No
	9	0	107	-	6.9	No
	10	0	102	-	3.3	Yes
Composite	8	2	183	0.19	7.6	No
	9	2	183	0.19	3.8	Yes
AADTT: 8,000						
CRCP	8	0	140	-	25.4	No
	9	0	110	-	9	No
	10	0	104	-	4.3	Yes
Composite	8	2	186	0.21	9.9	No
	8	3	194	0.31	5.6	No
	9	2	186	0.21	5	Yes

MEPDG = *Mechanistic-Empirical Pavement Design Guide;* CRCP = continuously reinforced concrete pavement; SMA = stone matrix asphalt; IRI = International Roughness Index; AADTT = annual average daily truck traffic; - = not applicable.

Table 24. Comparison of MEPDG-Predicted Distresses Between Composite Pavement and CRCP for a Design Life of 50 Years

Pavement Type	Layer Thickness (in)		Predicted Distress			
	CRCP	SMA	Terminal IRI (in/mi)	Rutting (in) (Asphalt Only), 15 year	Punchouts (No./mi), 50 year	Criteria Satisfied
AADTT: 5,000						
CRCP	8	0	140	-	24.8	No
	9	0	111	-	8.8	No
	10	0	105	-	4.2	Yes
Composite	8	2	212	0.19	9.4	No
	9	2	212	0.19	4.8	Yes
AADTT: 8,000						
CRCP	8	0	152	-	30.5	No
	9	0	115	-	11.1	No
	10	0	106	-	5.3	Yes
Composite	8	2	215	0.21	11.9	No
	8	3	224	0.31	6.8	No
	9	2	215	0.21	6.2	No
	9	3	225	0.31	3.7	No
	10	2	216	0.25	3.5	Yes

MEPDG = *Mechanistic-Empirical Pavement Design Guide;* CRCP = continuously reinforced concrete pavement; SMA = stone matrix asphalt; IRI = International Roughness Index; AADTT = annual average daily truck traffic; - = not applicable.

Pavement Condition and Performance

Pavement conditions were visually observed a few days after construction and 2 years after opening to traffic. There were no visible distresses on the surface, as shown in Figure 20. VDOT's PMS also rated this section of pavement in 2019 after 2 years of service. The average

Critical Condition Index (CCI) and rutting from the 2019 PMS data were 98 and 0.12 in, respectively. The average IRI was 65 in/mi compared to 57 in/mi right after construction. Therefore, the composite pavement on US 60W is performing satisfactorily as judged by visual appearance.

Figure 20. Composite Pavement SMA Surface Condition After 2 Years of Traffic. SMA = stone matrix asphalt.

Crack Survey on Exposed Concrete Surface

A CRCP is supposed to crack at 3- to 8-ft spacings, but the cracks usually stay tight because of the presence of reinforcing steel. The section of CRCP on US 60W was surveyed for cracks after 3 weeks of curing, as shown in Figure 21, before it was overlaid with asphalt. Two 500-ft sections of the right lane were randomly selected to collect crack width and spacing information. These sections were surveyed between 10 a.m. and 12 p.m. Most cracks were smaller than 0.3 mm at 10 a.m. and 0.4 mm at 11 a.m., with a range between 0.1 and 0.6 mm. The crack spacing was 3 to 8 ft, as expected. Histograms of crack spacing and width at 11 a.m. are presented in Figure 22.

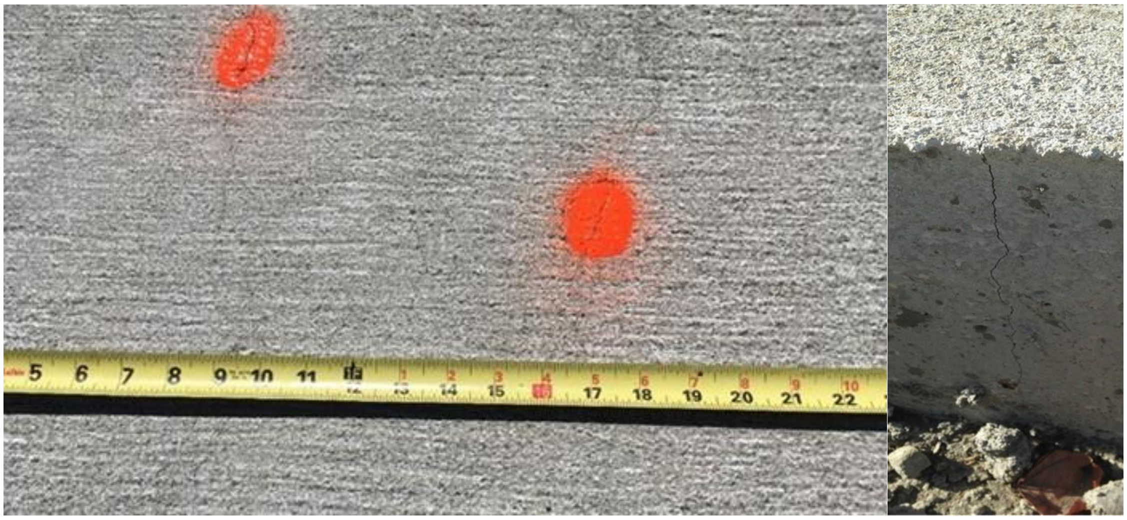

Figure 21. Crack Survey on US 60 CRCP Exposed Surface After 3 Weeks of Curing. CRCP = continuously reinforced concrete pavement.

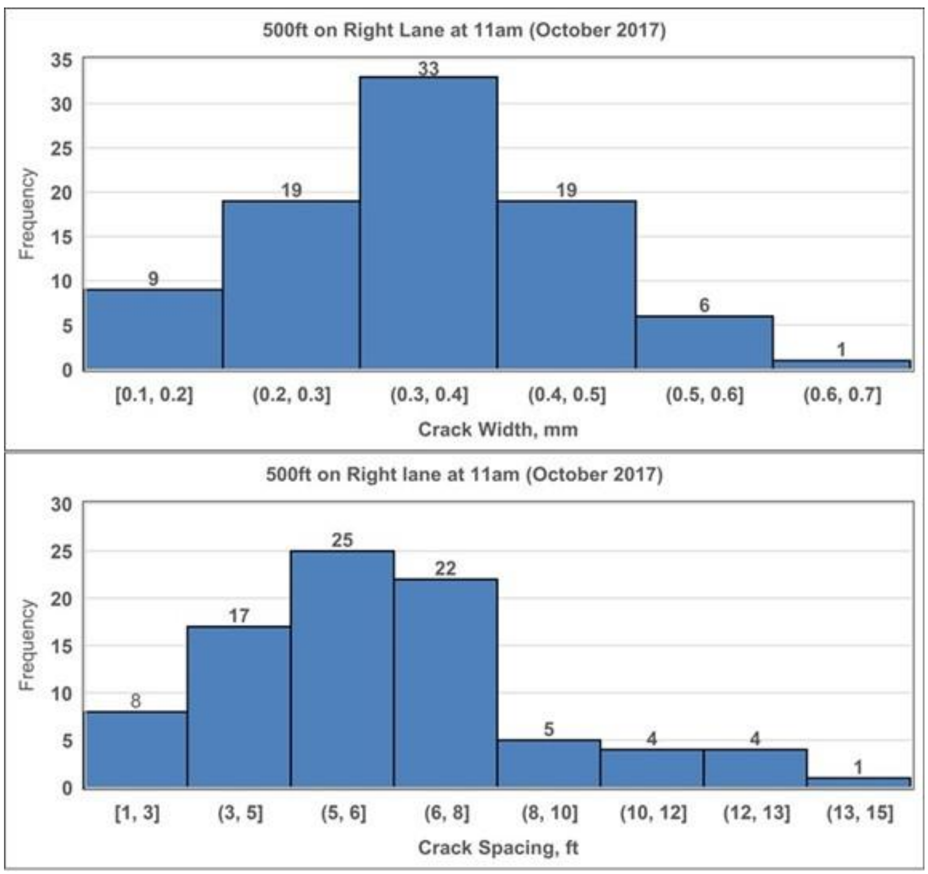

Figure 22. Frequency Distribution of CRCP Crack Width and Spacing on US 60. CRCP = continuously reinforced concrete pavement.

Temperature Distribution in CRCP

The temperature inside the pavement was monitored at different depths for 6 months. The first few weeks of data represent the exposed concrete surface before the asphalt overlay was placed. Complete sets of data were not available for all six locations because batteries inside some of the sensors started to die after 2 months. The temperature data at different depths and air temperatures for the first and third weeks of October 2017 are presented in Figure 23. The CRCP surface was exposed during the first week because it was not overlaid with SMA until October 11.

The maximum daily swings in temperature at three locations/depths are listed in Table 25: (1) air, (2) at the top of the concrete 1 in below the surface, and (3) at the bottom of the concrete 7 in below the surface. The maximum difference in temperature between the top and bottom of the concrete in a 1-week period is also listed in the table.

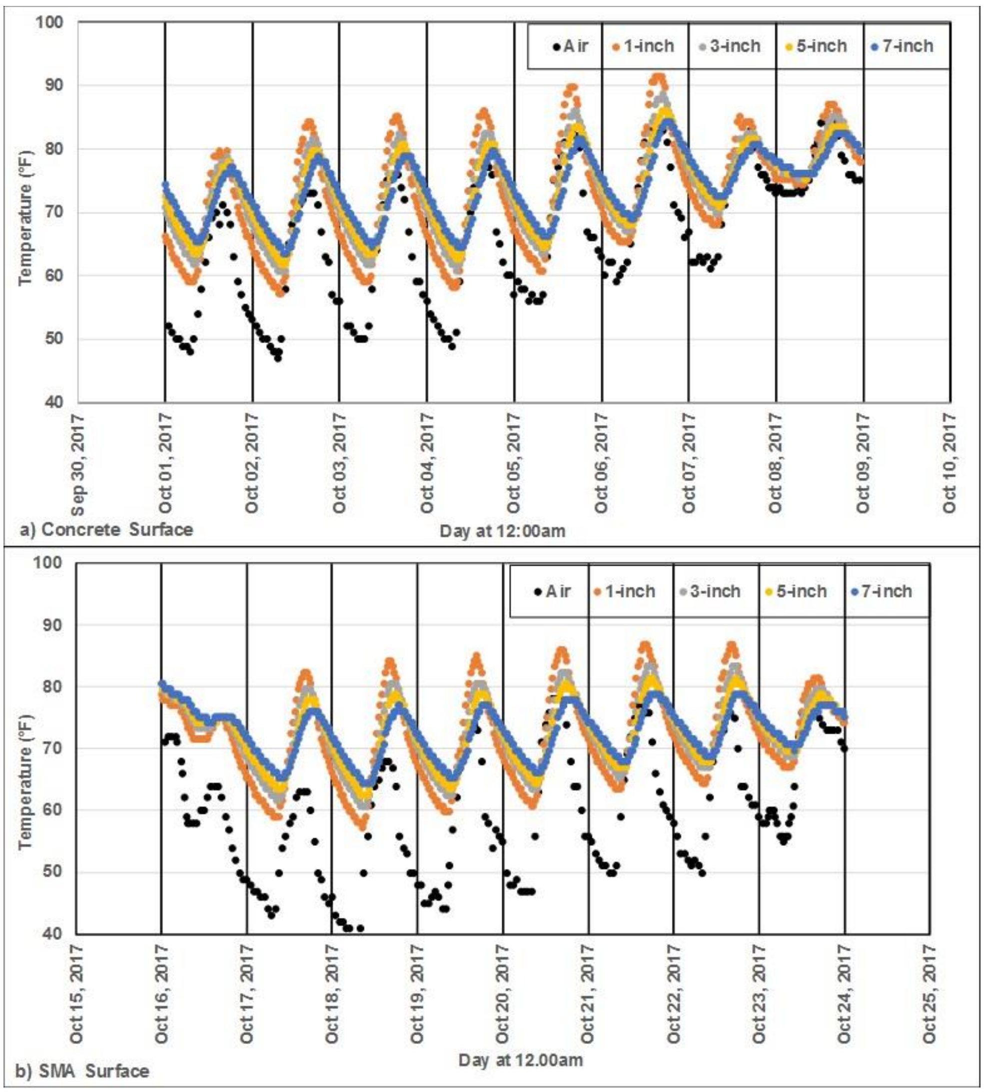

Figure 23. Temperature Distribution Inside CRCP: (a) exposed concrete surface (before SMA placement); (b) when overlaid with SMA (a few days after SMA placement). CRCP = continuously reinforced concrete pavement; SMA = stone matrix asphalt.

Table 25. Temperature Cycles and Gradients in CRCP With and Without SMA Overlay

Maximum Temperature Difference (Daily Swing or Cycle) in 1 Day	Exposed Concrete Surface, °F	Asphalt (SMA) Overlay, °F
Air	27	31.2
1 in into the concrete	28.8	26.8
7 in into the concrete	15.3	12.0
Difference between top and bottom (gradient)	12.6	10.8
Observation days	October 1-8, 2017	October 16-23, 2017

CRCP = continuously reinforced concrete pavement; SMA = stone matrix asphalt.

Although the air temperature swing (31.2 °F vs. 27 ºF cycles) was higher for the week (October 16-23, 2017) with SMA overlaid compared to the week with exposed concrete (October 1-8, 2017), the temperature swing at the top and bottom of the CRCP was lower when SMA was present. Also, the temperature gradient was steeper (12.6 °F vs. 10.8 °F) in exposed concrete. Both the lower swings and the flatter gradient clearly indicate an insulating effect of the asphalt layer. This insulating effect is expected to lower curling and associated stresses in the concrete pavement, which may result in a lower concrete thickness requirement in the design and/or better performance.

Pavement Drainage Condition

Although a pavement edge drain (VDOT UD-7) was recommended for this project, it was omitted during construction because of a constructability issue in the field. During a site visit to collect temperature data in February 2018, a significant amount of water was found at the edge of the pavement inside the data collection box and tubes, as shown in Figure 24. In a subsequent visit in May 2018, these locations were dry.

Figure 24. Standing Water at the Edge of the Pavement in February 2018

FWD Testing

FWD testing was conducted on top of the SMA surface in June 2019 after about 20 months of service. This testing also included another 1-mile-long section on the east side of the I-295 Bridge. Although this additional section is a composite pavement, asphalt was overlaid on top of old (37-year) concrete pavement. Four load levels (6, 9, 12, and 16 kips) were used with four drops per load. The testing was performed on asphalt overlays. The deflection profiles under a 9 kips load are plotted in Figure 25; the deflection values at the new composite section are consistently lower than for the old pavement (east side of I-295 Bridge), indicating that the new composite pavement was stronger, as expected. In 2015, similar FWD testing was conducted on the old CRCP before this rehabilitation work, and deflection data for a 9 kips load are included in the same figure; deflection values are similar to those for the east side old composite pavement. Average deflections under the 9 kip load are presented in Table 26. The pavement condition data from VDOT's PMS are included in the table for comparison purposes. Despite a high CCI (90), FWD deflection values clearly show that the old composite pavement is structurally weaker than the new composite pavement, as expected.

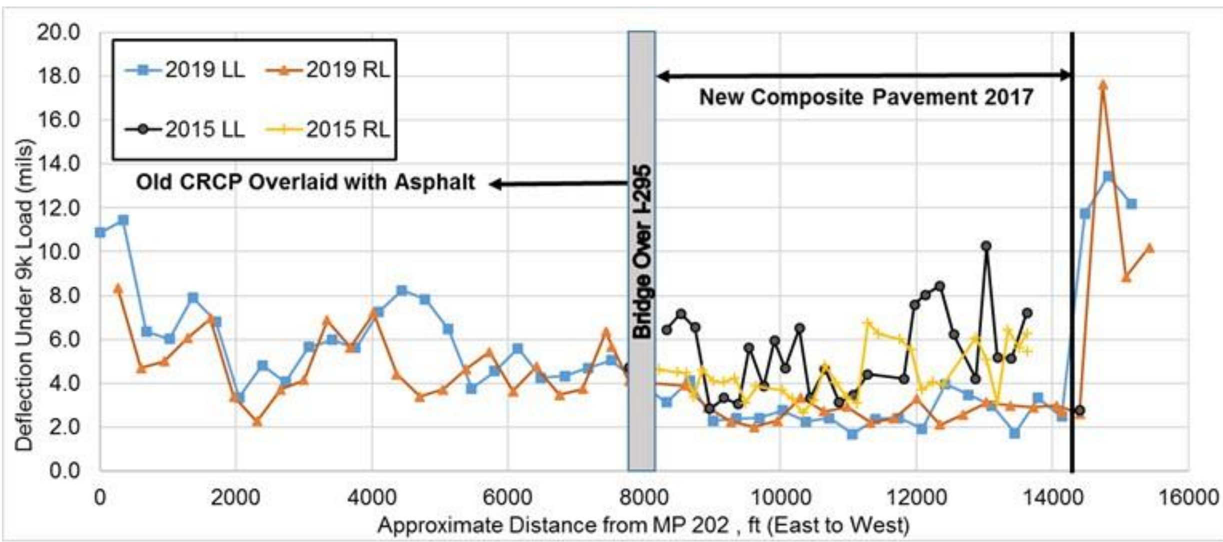

Figure 25. Falling Weight Deflectometer Deflection Under 9 Kip Load on US 60

Table 26. Average Deflection (mils) Under 9 Kip Load of FWD and VDOT PMS Data

Year	Surface	Deflection Under 9 Kips (mils)		Pavement Condition	
		Right Lane	Left Lane	IRI	CCI
2019	New composite (asphalt over CRCP)	2.75	2.7	65	98
	Old composite (asphalt over old CRCP)	4.71	5.7	131	90
2015	37-year-old CRCP	4.46	5.44	-	31[a]

FWD = falling weight deflectometer; PMS = Pavement Management System; CRCP = continuously reinforced concrete pavement; IRI = International Roughness Index; CCI = Critical Condition Index; - = data not available.
[a] This rating was for concrete pavement surface with no asphalt overlay.

Maintenance Needs Assessment

VDOT PMS data were used to assess the maintenance needs of a composite pavement. A total of 514 lane-miles of CRCP sections built from 1967-1992 on I-64, I-664, I-95, I-295, I-85, US 60, US 58, SR 76, and SR 288 were used for this analysis. They all served their respective design life of 20 years and were then overlaid with flexible surfacing, causing them to function as composite pavements. A complete list of these sections is provided in Table 27, which includes location, lane-miles, AADTT, original construction year of CRCP, and year it was first overlaid. Figure 26 shows the relative location of these sections, most of which are along the I-64 corridor.

A list of these (currently) composite pavement sections along with some of the construction history and maintenance records is provided in the Appendix. The construction and maintenance history information was collected from the Pavement Structure Table in VDOT's PMS. The pavement surface condition rating information was also collected from VDOT's PMS. It is important to note that these maintenance records do not include any patching or minor repair; only overlay or surface treatment information is available. Many of the pavements have gone through two or more maintenance (overlay) cycles, and they are performing satisfactorily with a total pavement service life of as much as 52 years (average 40 years with a standard deviation of 8.7 and a minimum of 27). Table 28 summarizes some of the statistics regarding CRCP and asphalt overlay life. On average, the CRCP was overlaid with asphalt at the age of 26 years (lane-mile weighted average) with a standard deviation of 8.5 (maximum of 48 years and minimum of 11 years).

The following general observations from Tables 27 and 28 and the information in the Appendix were made:

- The oldest functionally composite pavement (at present) was built in 1967 and the newest was built in 1992 as CRCP; average in-service pavement life is 40 years; CRCP lasted about 26 years before needing an overlay (the only fully integral composite pavement, built as composite, on US 60 was constructed in 2017).

- Some of these sections have had two or more cycles of maintenance (overlay or surface treatment) between the year of original construction and 2019; no records of concrete pavement patching are available as so many of these pavements might have been patched before the overlay treatment.

- Of these pavements, 67% (345 lane-miles) have had only one overlay and 33% (169 lane-miles) have had two or more overlays; 15% (75 lane-miles) have had three or more overlays.

- Although there is a general decrease in overlay life as CRCP is aging, average overlay life is more than 10 years; the average life of the first overlay was 13.6 years; the second lasted about 10 years.

Table 27. Asphalt-Overlaid CRCP (Composite Pavement) Sections on VDOT Network

Route	County	Mile Post (direction)	Lane-miles	AADTT (approximate)	Year CRCP Constructed[a]	Year First Overlay or Surface Treatment
I-64	Albemarle	107-130 East	35	3000	1970 (22-26)	1992, 1994 and 1996
		107-127 West	35	3000	1970 (23-26)	1993 and 1996
	Louisa	131-136 East	10	3000	1970 (24)	1994
		135-136 West	2	3000	1987 (18)	2005
	Henrico	178-181 East	5	1000	1967 (38)	2005
		183-186 East	8	1400	1968 (42)	2010
		191-196 East	13	2000	1968 (35)	2003
		178-187 West	18	1000	1968 (36-42)	2004, 2005 and 2010
		191-195 W	11	2000	1968 (35)	2003
	New Kent	206-215 East	18	3000	1973 (20)	1993
		215-225 East	18	3000	1972 (18)	1990
		221-223 West	3	3000	1972 (18)	1990
	Norfolk	274-277 East	5	1500-2000	1975 (40)	2015
		293-300 East	14	2900-5600	1969 (42)	2011
		274-277 West	5	1750-2000	1975 (40)	2015
I-664	York	1-3 East	7	-	1983 (32)	2015
	Nansemond	11-14 East	10	2000	1991 (22)	2013
	Norfolk	14-18 East	8	2300-2600	1991 (22)	2013
I-295 (North of I-64)	Henrico	29-32 North	11	2350	1980 (19)	1999
	Hanover	32-36 North	17	2750	1980 (16)	1996
		38-42 North	15	3150	1980 (27)	2007
	Henrico	42-52 North	28	1200	1980 (27)	2007
	Hanover	32-36 South	17	2750	1980 (16)	1996
		36-42 South	19	-	1980 (23-30)	2003, 2005 and 2010
	Henrico	46-47 South	4	1700	1980 (18)	1998
I-295 (South of I-64)	Prince George	0-12 North	23	2600	1992 (35)	2017
		9-13 South	24	2900	1992 (34)	2016
	Chesterfield	15-17 North	6	2876	1990 (27)	2017
		15-17 South	6	-	1990 (26)	2016
I-85	Dinwiddie	44-46 North	4	2100	1969 (48)	2017
		55-62 North	11	2500	1969 (48)	2017
I-95	Sussex	17-22 North	10	3200	1982 (17)	1999
		17-22 South	10	3200	1982 (17)	1999
US 58	Southampton	423-432 East	17	1200-1750	1988 (24)	2012
US 60	Henrico	200-202 West	2	600	1979 (37)	2016
US 60	*Henrico*	*199-200 West*	*2*	*600*	*2017*	*New*[a]
SR 288	Chesterfield	0-8 North	16	1100	1990 (26)	2016
		12-16 North	7	1300	1988 (26)	2014
		1-8 South	13	1100	1990 (26)	2016
		13-15 South	3	1300	1988 (27)	2015
SR 76	Chesterfield	1-7 North	13	<300	1988 (29)	2017
		1-5 South	6	<300	1988 (27)	2015
		5-8 South	6	<300	1988 (28)	2016

The number in parentheses is the age of the CRCP (continuously reinforced concrete pavement) at the time of the first overlay; AADTT = annual average daily truck traffic.

[a] Constructed as a composite pavement (first pavement in VDOT).

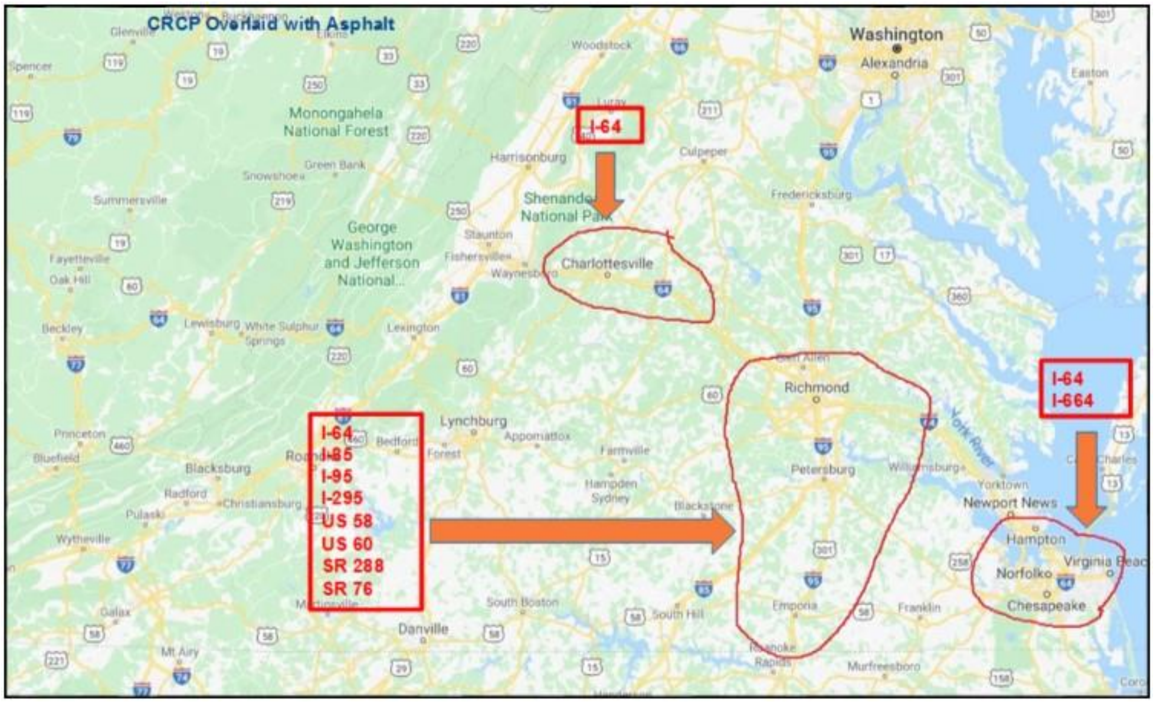

Figure 26. Locations of Composite Pavements in VDOT System. Courtesy of Google Maps.

Table 28. Average CRCP and Asphalt Overlay Life for Composite Pavements in VDOT's PMS

Statistics	Pavement Age in 2019, years	Age of CRCP Before Any Rehabilitation Work, years	First Overlay Life (Before Second Cycle Of Maintenance), years	Second Overlay Life (Before Third Cycle of Maintenance), years
Average	40	26	13.6	10.0
Standard deviation	8.7	8.5	5.3	5.2
Maximum	52	48	22	18
Minimum	27	11	6[a]	2[b]
Lane-miles rehabilitated	514	514	169 (33%)	75 (44%)
Lane-miles still in service	514	514	345 (2-23 years)	94 (2-11 years)

PMS = Pavement Management System; CRCP = continuously reinforced concrete pavement.
[a] A few older hot mix asphalt overlays in the 1990s lasted only 6 to 8 years, but subsequent stone matrix asphalt overlays performed fine.
[b] Only 1 overlay needed a latex surface treatment (0.3 in) in 2 years, and it is still performing fine after 7 years.

For further analysis of the effectiveness of any particular treatment, the composite pavements were grouped into the following nine categories (six overlay types, two surface treatments, and one combination):

1. *Multi-lift HMA overlay (3.5 to 6.5 in).* Around 119 lane-miles of pavement in 14 sections received a multi-lift HMA overlay that consisted of 1.5 in of surface mixture (SM) over 2.0 to 5.0 in of base mixture (BM) or intermediate mixture (IM). Most of them were constructed as the first overlay from 1992-1999. Some sections have thicker bases than others; 76 lane-miles have greater than or equal to 3.0 in BM and/or IM, and 43 lane-miles have less than 3.0 in. In general, the AADTT on these

sections is about 3,000; only 15 lane-miles have 275 to 2,700 AADTT. Most of them, 104 lane-miles, received the second overlay in an average of 12 years (standard deviation of 5, maximum of 22, minimum of 6). The remaining 15 lane-miles are still in service with the first overlay after 4 to 21 years. The pavement surface condition data right before construction of the second overlay are available for only four sections; the average IRI was 100, and the Pavement Condition Index or CCI varied from 21 to 59.

2. *Single-lift HMA overlay (1.5 to 2.0 in).* Only 38 lane-miles of composite pavement in six sections were milled and filled with a single layer of 1.5 to 2.0 in of HMA as a second or third overlay except for 2 lane-miles. All of this construction took place after 2000. Two sections with 20 lane-miles received a subsequent treatment in 10 and 12 years; their IRIs were 61 and 68, and their CCIs were 71 and 43. The rest of the sections with 18 lane-miles are still in service after 0 to 6 years; their IRIs vary from 77 to 131, and their CCIs vary from 78 to 91. The AADTT is 3,000 with the exception of 2 lane-miles, which have an AADTT less than 600.

3. *Base HMA and Surface SMA (3.5 to 4.0 in).* A combination of HMA and SMA was applied on seven sections of old CRCP totaling 106 lane-miles as the first overlay. Five sections (62 lane-miles) were placed in 1996, and two sections (44 lane-miles) in 2007. These overlays consisted of 1.5 to 2.0 in of SMA SM over 2.0 to 2.5 in of HMA IM. The AADTT on these sections is 2,700 to 3,200 with the exception of one section (29 lane-miles), which has 1,300. These sections are performing well, with 78 lane-miles still in service after 12 to 23 years; their average IRI and CCI are 70 and 91, respectively. The remaining 28 lane-miles received the second treatment in 18 to 23 years, and their average IRI and CCI are 94 and 71, respectively.

4. *Single-lift SMA (9.5 or 12.5 mixture) Overlay (1.5 to 2.0 in).* Ten sections totaling 92 lane-miles have received single-lift SMA overlays mostly as mill and fill for the second or third treatment after 2006 except for one section (9 lane-miles), which received the first overlay in 1999. After 18 years, this one section was overlaid in 2017 with an IRI of 107 and a CCI of 54; AADTT is around 3,300. The remaining sections are still in service after 0 to 7 years with an average IRI and CCI of 73 and 91, respectively. For one of these in-service sections, this is actually the fourth treatment. The average AADTT for all sections is around 3,000 except for two with 1,300 and 2,300.

5. *Two-lift SMA overlay (3.5 in).* Fifteen sections totaling 120 lane-miles of CRCP were overlaid (first treatment) with a two-lift SMA from 2003-2017. These overlays consisted of 1.5 in of SMA 9.5 or 12.5 over 2.0 in of SMA 19.0. The AADTT on these sections ranges from 1,000 to 3,700. Most of them (all but two sections, 104 lane-miles) are still in service after 2 to 16 years with an average IRI and CCI of 82 and 89, respectively. Two sections (16 lane-miles) were overlaid in 8 years (IRI of 113 and CCI of 70) and 16 years (IRI of 116 and CCI of 87).

6. *Multi-lift SMA overlay (4.5 to 6.5 in).* A multi-lift SMA overlay is not a common treatment, but it was used intermittently from 2007-2015. Five sections totaling about 42 lane-miles have received this treatment as a first, second (mill and fill), or even third (mill and fill) overlay. These overlays consisted of 1.5 to 2.0 in of SMA 12.5 over 3.0 to 4.0 in of SMA 19.0. All of them are still in service after 3 to 12 years with an average IRI and CCI of 92 (73 to 141) and 81 (68 to 97), respectively. The AADTT varies from 1,000 to 3,500, with an average of 2,000.

7. *Latex-modified surface treatment (slurry seal) (0.2 to 0.3 in).* As a second or third treatment, about 47 lane-miles of pavement in four sections received latex-modified slurry seal; most of them were constructed after 2006. All of them were overlaid in 5 to 11 years (average 9 years) with an average IRI of 87 and CCI of 65 right before the overlay. AADTT on these sections was around 3,000.

8. *Thin HMA concrete overlay (THMACO) (0.7 to 0.75 in).* Nine sections of pavement totaling about 100 lane-miles were treated with a THMACO, a thin gap-graded plant mixture. Most of these treatments were applied directly over CRCP except for 29 lane-miles for which it was the second treatment. All THMACOs were applied after 2013. The AADTT was around 2,700 except for 18 lane-miles with 200 to 300. Seven sections with 82 lane-miles are still in service after 0 to 5 years; the remaining two sections were overlaid in 4 years. The performance of these sections is good, with an IRI of 75 and a CCI of 89.

9. *THMACO (0.75 in) + SMA overlay (<4.0 in: 2.0 in of SMA 19.0 + 1.5 to 1.75 in of SMA 12.5).* A few (seven) sections received a two-lift SMA over THMACO from 2013-2017. The total lane-miles was only 45, and two sections with 18 lane-miles actually had exposed THMACO for 4 years before receiving SMA. All of them are still in service after 2 to 6 years with an average IRI and CCI of 93 (61 to 130) and 91(83 to 98), respectively. The AADTT of these sections varies from 2,000 to 3,000 with an average of 2,300.

All nine treatment options have provided additional service life to CRCP, with SMA overlay being the best with as much as 23 years. The performance of these nine treatment options is summarized in Table 29.

Most of the old CRCPs were overlaid with multiple lifts of asphalt as a first treatment, which indicates the need for additional structure to make up for the deterioration in old CRCP. The HMA base with SMA surface performed the best with more than 18 years of life. The second and subsequent treatments are usually mill and fill of HMA or SMA SM, and they also provided more than 10 years of service. Although the data were limited, an SMA surface as a second treatment lasted for 18 years.

Table 29. Performance of Different Treatment Options on CRCP

Treatment	Total LM	AADTT	Overlaid or Next Treatment				Still in Service			
			LM	Average Life, yr	Pavement Condition		LM	Average Life, yr	Pavement Condition	
					IRI	CCI			IRI	CCI
First Overlay/Treatment Directly Over CRCP										
Multi-lift HMA	119	3000	104	12 (6-22)	100	21-59	15	4-21	-	-
Base HMA + Surface SMA	106	3000	28	18-23	94	71	78	12-23	70	91
Two-lift SMA	120	1000-3700	16	8 and 16	114	78	104	2-16	82	89
THMACO	100	2700	18	4	-	-	82	0-5	75	89
THMACO + SMA	45	2300	None				All	2-6	93	91
Second or Third Overlay/Treatment (Mill and Fill)										
Single-lift HMA	38	3000	20	10 and 12	61 and 68	70 and 43	18	0-6	77-131	78-91
Single-lift SMA	92	3000	9	18	107	54	83	0-7	73	91
Latex-Modified ST	47	3000	All	9	87	65	None			
First (Directly Over CRCP) or Second or Third Overlay/Treatment (Mill and Fill)										
Multi-lift SMA	42	1000-3500	None				All	3-12	92	81

CRCP = continuously reinforced concrete pavement; LM = lane-miles; AADTT = annual average daily truck traffic; IRI = International Roughness Index; CCI = Critical Condition Index; HMA = hot mix asphalt; SMA = stone matrix asphalt; THMACO = thin hot mix asphalt concrete overlay; ST = surface treatment; - = not available

It is reasonable to assume a minimum 10-year mill and fill maintenance cycle for composite pavement; it could be even longer. The literature, in particular Flintsch et al. (2009), also supports an assumption of maintenance activity consisting of mill and overlay on a continuous 10-year cycle. On average, the first overlay lasted 13.6 years (Table 26) on top of an old deteriorated pavement so it is reasonable to assume that an asphalt overlay on a new composite pavement may last 15 years or so. Therefore, a schedule of maintenance and rehabilitation suitable for composite pavement with a CRCP base would be simply a functional mill and fill of asphalt surface (preferably SMA) every 10 to 15 years where asphalt (SMA surface) is built into the composite pavement.

Since two of the major distresses on a composite pavement are crack/joint reflection and rutting, a suggested structure would be CRCP overlaid with SMA. Since CRCP does not have joints, crack/joint reflection through the asphalt layer will not occur if the CRCP is designed appropriately for fatigue damage; reinforcement should keep the cracks tight. In addition, SMA mixtures are typically rut resistant. Moreover, the performance of an SMA surface as an overlay for CRCP bases in Table 27 further suggests the durability advantages of SMA; most lasted more than 16 years except for one small section (less than 3 lane-miles) with a two-lift SMA, which needed to be overlaid in 8 years.

Cost and Benefit Assessment

The SHRP2-R21 report (Rao et al., 2013) indicated that an asphalt overlay reduces temperature and moisture gradients (both magnitude and nonlinearity) in an underlying PCC slab. Similar observations were made on the US 60 project, as mentioned previously. These properties of an asphalt overlay mediate PCC slab curvature and related load and thermal stresses. The insulation provided by the asphalt overlay is substantial enough to reduce the PCC slab thickness by 1 to 3 in, depending on the traffic, climate, support conditions, materials properties, etc., while allowing the thinner but asphalt-overlaid PCC to have the structural capacity of a thicker but exposed PCC pavement This reduction in slab thickness and expense will at least partially offset the added expense of the asphalt overlay; in other words, the construction cost for a composite pavement may be comparable to that of a CRCP from the design perspective.

McGhee and Clark (2007) and Habib (2017) found that the pavement overlaid with SMA on top of CRCP tends to provide a longer service life compared to other types of pavement overlay. McGhee and Clark (2007) mentioned these comparisons as a "bird's eye" perspective and recommended against any sweeping conclusion. Their comparison table using VDOT 2006 windshield survey data is shown in Figure 27.

The 2019 PMS data were used to update Habib's plot of CCI vs. age for both HMA and SMA overlays on three types of pavements: bituminous, jointed concrete, and CRCP. The Pavement Condition Index was used as the performance measure and plotted against overlay age for different types of pavement. Figure 28 shows all six plots, and it indicates that the composite pavement with SMA over CRCP has a better pavement condition with a longer service life. The SMA overlay is showing better performance, as indicated by a CCI of 80 or higher for most of the SMA overlays as old as 15 years or so. It is important to note that both comparisons used the performance data, which simply reflect the overall network conditions without reflecting the varying conditions at different sections. Such grouping, as mentioned by McGhee and Clark (2007), is an oversimplification of the actual situation, which varies over a wide range. Appropriate filtering/grouping of the conditions such as pavement type and structure, mixture type and thickness, traffic count and distribution, and climatic condition, etc., is needed for any meaningful comparative analysis, which is outside the scope of this study.

	Underlying Surface		
Mix	BIT	BOJ	BOC
SM 9.5A	10.5		
SM 9.5D	8.2	7.6	12.1[a]
SM 12.5A	11.3		
SM 12.5D	7.6	7.6	18.2[a]
SMA 9.5	22.2		
SMA 12.5	17.7	9.4	23.1

BIT = bituminous; BOJ = bituminous over jointed concrete; BOC = bituminous over continuously reinforced concrete.
[a]Only 1 representative section per year in database.

Figure 27. Estimated Life Expectancy of Different Types of Asphalt Overlay. From McGhee and Clark (2007).

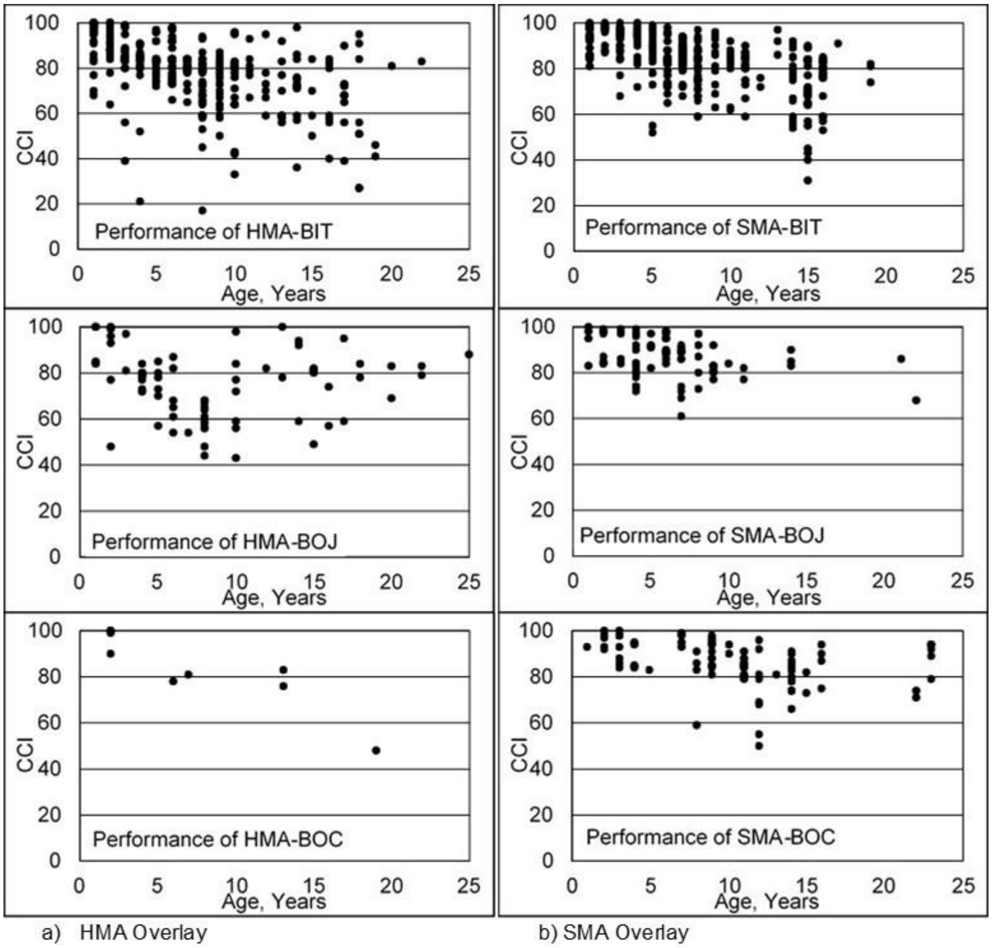

Figure 28. Overlay Performance for Different Types of Pavement: (a) HMA overlay; (b) SMA overlay. HMA = hot mix asphalt; SMA = stone matrix asphalt; BIT= bituminous over bituminous pavement; BOJ = bituminous over jointed concrete pavement; BOC = bituminous over continuously reinforced concrete pavement. Adapted from Habib (2017) and updated with 2019 data.

Any cost information to build a composite pavement was not readily available, as none was built by VDOT before this project. Table 30 contains published unit costs for composite pavement items from the VDOT website (VDOT, 2020c) calculated as statewide averages of the last 2 years of bid tabulation. The quote for Item Code 10981, however, illustrates a problem with using these figures: the scarcity of projects using a specific construction item can suggest improbable point estimates for unit costs. A more realistic cost could be estimated based on the last 5 or 10 years of bid information adjusted for inflation and reflecting bid quantities.

Table 30. VDOT Bid Tabulation Cost Information for Composite Pavement Items

VDOT Item Code	Item Description	Statewide Bid Prices: Jan 2017-Feb 2019		
		Minimum	Maximum	Average
10981	8 in CRCP (square yard)	$79.81	$79.81	$79.81
16401	SMA 9.5E (ton)	$88.00	$109.00	$98.72
16403	SMA 12.5E (ton)	$84.73	$147.50	$112.77
16405	SMA 19.0E (ton)	$76.46	$121.30	$90.87
16360	HMA 12.5E (ton)	$64.75	$565.00	$75.66
16523	Flexible pavement planing 2-4 in (square yard)	$0.83	$25.00	$3.36

CRCP = continuously reinforced concrete pavement; SMA = stone matrix asphalt; HMA = hot mix asphalt.

The advantages of a new composite pavement notwithstanding, as detailed in the SHRP2 report, such pavements may have relatively high initial costs followed by a lower stream of maintenance costs. Therefore, a life cycle cost analysis (LCCA) may provide a meaningful comparison against any other type of pavement. Currently none is available for composite pavement. A feasibility-level LCCA (Flintsch et al., 2009) suggested that a composite pavement with a CRCP base may become more cost-effective for very high volumes of traffic. Although an LCCA is outside the scope of this study, a brief general discussion of factors that will affect a LCCA for composite pavement is provided here.

The cost pattern of composite pavements mentioned—higher construction cost in exchange for longer service life-- could create a distinct disadvantage in an LCCA "fit" into a 50-year analysis period. Specifically, the higher-initial-cost pavements with service lives that easily surpass 50 years are systematically penalized if remaining pavement value at the end of the analysis period is disregarded. Neglect of remaining pavement value in an LCCA will certainly result in incomplete LCCA results with a systematic bias against composite pavements. As discussed in a previous section, many in-service so-called "composite pavements" in the VDOT system have provided more than 50 years of service and are still serving the transportation need.

The treatment of remaining pavement or salvage value does not enjoy a consensus opinion in the literature. Salvage value can be treated in several ways, each of which has well-known shortcomings. Yet the imposition of a 30- or 50-year analysis period is so artificial a constraint for high-volume highways that will likely be in indefinite use that it is preferable to choose a method with known shortcomings than to neglect salvage value. The simplicity and low cost of the maintenance schedule for a high-initial-cost composite pavement consisting of a CRCP base overlaid with SMA will certainly require salvage value to be incorporated into the LCCA in order for pavement alternatives to be evaluated fairly.

The analytical impact of the discount rate is also significant. The discount rate is meant to be a reflection of current economic conditions in order to compare alternative uses of limited DOT funds. Misunderstanding of the basic function of the discount rate leads to the typical use of a fixed middling rate bearing no relation to the current business climate or profitability of alternative uses of DOT funds, therefore adding nothing of value to analyses undertaken to determine the best use of DOT funds among several possibilities.

Another important factor is the growth in traffic volumes that may compress maintenance activities or otherwise drive up lifetime costs, again to the disadvantage of composite pavements, which may be less maintenance-sensitive to traffic volume growth. Along this line, climatic factors may contribute more than traffic volume increases to the distresses in need of composite pavement maintenance.

To summarize, an LCCA that evaluates new composite pavement fairly against other alternatives would require some in-depth analysis with regard to unit costs, salvage value, the discount rate, and potentially forecast growth in traffic volume. In addition, highway user cost may be considered an additional criterion when life cycle costs to the agency among top alternatives are similar.

Summary of Findings

Construction of US 60 Composite Pavement

- Composite pavement on US 60 was designed in accordance with the 1993 AASHTO guide following a two-step process. In Step 1, a CRCP was designed for the expected traffic to be 8.42 in, and in Step 2, an asphalt overlay (2 in) was designed on top of reduced-thickness CRCP (8 in) to compensate for the reduction in thickness.

- This project was designed and planned to be constructed on top of the old CTA. Therefore, the old CTA was expected to be preserved from any construction-related damage. Although the existing CTA from the old pavement was found mostly intact in cores obtained before construction, an elaborate base course stabilization was performed, resulting in a significant cost increase. In addition to the actual deterioration of the CTA from 37 years of use, many factors might have contributed to the damage:

  — The old concrete was crushed in-place by using a guillotine breaker before removal, which can easily damage the CTA.

  — A back hoe while operating on top of the broken pavement was used to remove the broken pieces.

  — The haul truck carrying the broken pieces was driven over the exposed CTA.

  — Multiple storm events and heavy rain during these operations might have exacerbated the situation because of the presence of clay subgrade and poor drainage.

- The CRCP was paved one lane (12 ft) at a time. In order to keep both lanes together, the tie bars were drilled and epoxied before the next lane was paved.

- CRCP was successfully constructed using a concrete mixture that provided a 28-day compressive strength of more than 4,000 psi and a permeability of less than 1615 C.

- A wax-based curing compound over the concrete surface was used for curing and did not need any removal for asphalt overlay; just a tack coat was used. No debonding of SMA overlay has been observed in the last 2 years of service.

- CRCP was paved at a rate of 1,000 to 1,500 lane-feet per night, and eight cold joints were formed. Although the VDOT standard calls for doubling the reinforcement at a cold joint, it was not followed for the US 60 project. There are some indications in the literature that doubling the reinforcement actually hinders the consolidation of concrete because of congested reinforcement. It will be important to monitor the long-term performance of these cold joints.

- CRCP cracked at 3- to 8-ft spacings as expected after 3 weeks of curing, and crack widths were mostly less than 0.4 mm, with a range of 0.1 to 0.6 mm. These cracks are held tight with the reinforcement. Although the 28-day shrinkage in the laboratory sample was around 0.05%, field shrinkage is expected to be less since the concrete was overlaid with asphalt in 3 weeks.

- Two inches of SMA was placed in a single lift after at least 3 weeks of CRCP construction; no traffic was allowed over the CRCP.

  — One lane at a time was overlaid, and a longitudinal joint was 3 in off-center from a concrete lane joint.

  — For a very small section, the superelevation was corrected by using 4 in of SMA.

  — The average thickness for the SMA core was 2.21 in, and the density was above 95.75%, satisfying the requirement of 94% density.

- Additional samples for both concrete and SMA were collected and characterized at the VTRC laboratory for pavement ME design. Concrete properties are summarized in Table 10: 28-day compressive strength = 4,700 psi; beam flexural strength = 710 psi; and modulus of elasticity = 4.2 million psi. SMA properties are summarized in Tables 11 through 15: they satisfied VDOT requirements for volumetric properties and gradation, SMA stone-on-stone contact, a binder grading of PG 76-22, and the presence of polymer.

- SMA mixture was further evaluated at the VTRC laboratory for cracking and rutting:

  — The APA test indicated a rut-resistant mixture.

  — The MSCR test confirmed the rut resistance of the polymer modified binder.

  — The Texas overlay test and the IDEAL-CT indicated increased resistance to cracking.

- According to VDOT's yearly traffic data, AADT on this section of roadway was 14,140, with 6% truck traffic. Actual traffic data were collected during 2 weeks in May 2018, which came out to be around 13,000 AADT. The truck distribution from these actual traffic counts was used for analysis using Pavement ME Design: the percentage distributions of truck Classes 4 through 13 were 27, 19, 17, 1, 15, 20, 1, 0, 0, and 0, respectively.

  — VDOT statewide truck traffic distribution data show that Class 9 trucks are predominant (77%). However, for the US 60 project, Class 9 truck distribution was only 20%.

  — Because of the low percentage of truck traffic from the actual traffic count, the pavement ME analysis showed very low distresses for this composite section.

- The 2-in SMA overlay provided an insulating effect on the CRCP. When temperatures for 1 week in October 2017 were compared, the temperature gradient (difference between top and bottom of concrete) was 2 ºF steeper (12.6 °F in exposed concrete vs. 10.8 °F in SMA-covered concrete) whereas daily maximum air temperature cycles were 4 ºF lower (27 °F in the case of the exposed concrete surface vs. 31.2 °F in the case of the SMA-covered surface) in exposed CRCP than in the SMA-covered section. This insulating effect of asphalt is expected to lower curling and associated stresses in the concrete pavement, which may result in a lower concrete thickness requirement in the design and/or longer service life.

- The US 60 composite pavement is performing satisfactorily after 2 years of traffic: 2019 VDOT PMS data show a CCI of 98 and an IRI of 65. A visual observation did not indicate any distresses.

- An edge drain was recommended in the design but was omitted during construction. There was some evidence of a lack of drainage during the first year of service near the instrumentation boxes at the median side of the edges, but those spots were dry in the second year of service.

- The FWD deflection under a 9-kip load for the new composite section was 2.7 mils, indicating that it was structurally stronger than a similar "old composite" pavement (37-year-old CRCP overlaid with 2-in SMA for 3 years) with an FWD defection of 5.2 mils.

Pavement ME Analysis

- The pavement ME design procedure using the AC overlay over CRCP can be used to design a new composite pavement.

- Rao et al. (2013), in an SHRP2 report, indicated that AC bottom-up cracking, low temperature thermal cracking, and AC top-down cracking are less significant for composite pavements (AC over CRCP), and hence these distresses are not required as design criteria.

- Pavement ME analysis indicated that reflection cracking prediction results are not reasonable. Moreover, current transverse and reflective models used in Pavement ME Design are revised in the recent versions. VDOT still uses the older version (V2.2) and has not conducted local calibration for the new reflection cracking models. With adequate thickness and steel reinforcement (cracks in the CRCP are held tight by the reinforcing steel) in the CRCP, transverse cracks and thereby the reflection of transverse cracks can be reduced.

- Pavement ME analysis indicated that punchouts and rutting in an AC layer are significant distresses for composite pavement (AC over CRCP) design.

- Pavement ME analysis was conducted for typical composite structures (8- to 10-in CRCP + 2- to 3-in SMA) for VDOT statewide average traffic. For a similar level of distresses, 1 to 2 in of CRCP thickness can be reduced when composite pavement is used compared to bare CRCP. Rao et al. (2013) also suggested that concrete pavement thickness can be reduced by 1 to 3 in, depending on factors such as traffic, climate, and material properties, when compared with a bare concrete pavement.

Performance of the Rehabilitated (Overlaid) Old CRCP

- A properly designed CRCP successfully served the design life of 20 years and an average of 26 years of service when overlaid with asphalt. A total of 514 lane-miles of such CRCPs in VDOT's system are now composite pavement, and most of them are still in service after an average total life of 40 years.

- Of these pavements, 67% (345 lane-miles) have had one (first) overlay and 33% (169 lane-miles) have had at least two or more overlays; 15% (75 lane-miles) have had three or more overlays.

- Although there is a general decrease in overlay life as CRCP ages, the average overlay life is more than 10 years; the average life of the first overlay is 13.6 years, and that of the second is about 10 years.

- In general, the first overlay is two lifts of asphalt and the second is a single-lift mill and fill asphalt overlay. The use of two lifts during initial rehabilitation indicates that the old CRCP needed structural improvement.

- A schedule of maintenance and rehabilitation suitable for composite pavement with a CRCP base would be simply a functional mill and fill of the asphalt layer every 10 years at a minimum. It is important to note that most pavement sections considered in this analysis had a high AADTT of around 3,000. The literature, in particular Flintsch et al. (2009), also supports an assumption of maintenance activity consisting of mill and overlay on a continuous 10-year cycle for a new composite pavement.

- Some of the SMA overlays lasted for more than 18 years or so. Therefore, a 15-year mill and fill cycle may be appropriate for a properly constructed SMA surface.

- Some of the old composite pavements are still in service after more than 50 years. Therefore, a new composite pavement is expected to last at least 50 years if not more.

Cost and Benefit Assessment

- A composite pavement can provide a long service life of 50 years or more and could be cost-competitive:

  — With only 10 to 15 years of mill and fill of asphalt as maintenance, the use of SMA may facilitate a 15-year cycle.

  — Asphalt overlay provides thermal insulation to CRCP so there will be less curling and reduced stresses; a reduction of 1 to 3 in of CRCP and an associated cost reduction are possible.

  — A predominant distress in composite pavement is crack reflection; this could be minimized or even eliminated if CRCP were used as a base. The reinforcement in

CRCP and the insulation of the asphalt layer will keep the cracks in CRCP tight and they will not reflect through.

— Another important distress in composite pavement is rutting; the use of an SMA surface layer could reduce this distress in composite pavement since SMA is usually rut resistant.

- Although a LCCA would be the appropriate tool to assess the cost and benefit of any pavement system, it was outside the scope of this study. The development of an LCCA would need a careful evaluation of discount rate, salvage value, service life, analysis period, and maintenance need based on traffic volume.

CONCLUSIONS

- *Current VDOT specifications and standards were sufficient for construction of a composite pavement except for a few details.*

  — Only a minor detail in the end anchorage was missing from the VDOT standard.

  — Doubling the reinforcement at a cold joint in CRCP was not followed.

- *During the removal of old concrete pavement, significant base damage occurred. A. heavy rain event exacerbated the situation, resulting in a cost increase because of the need for base stabilization.*

- *The laboratory performance evaluation showed that the SMA mixture used in the US 60 project was highly crack and rut resistant.*

- *The AC overlay design procedure in Pavement ME Design could be used for new composite pavement (SMA over CRCP) design.*

- *Pavement ME analysis shows that punchouts and rutting in an AC layer are major distresses for composite pavements (AC over CRCP).*

- *Based on pavement ME analysis, in the case of higher traffic (truck) volumes, predicted distress levels for composite pavements were low compared to bare CRCP.*

- *For a similar performance on high truck traffic designs, the thickness of concrete pavement could be reduced 1 to 3 in depending on the traffic and environment when an asphalt overlay is included (i.e., a composite pavement).* This may partially reduce the initial cost of the pavement design, thus making it more cost-competitive.

- *It is reasonable to assume 10- to 15-year mill and fill cycles of the asphalt layer as the maintenance for composite pavement.*

- *A composite pavement is expected to provide a long service life when properly constructed.*

  — Existing CRCP that has become composite (overlaid with asphalt) in the VDOT system is still in service after more than 50 years and is performing very well.

  — One of the predominant modes of distresses of the existing composite pavement (AC over CRCP) is the reflection of the distresses (punchouts, cracks, widened longitudinal joints) from the old CRCP that might not have been adequately repaired during overlay. A new composite pavement is not expected to have these distresses, ensuring further better performance.

  — The predominant mode of failure in a composite pavement is crack reflection (punchout reflection with CRCP) and rutting; SMA is designed to resist both types of distress.

  — The reinforcement in a CRCP keeps the cracks in the pavement tight. Moreover, the insulating effect of an asphalt overlay will reduce distress development in the CRCP, further reducing the chances of reflection through a crack-resistant overlay.

  — A newly built composite pavement with a CRCP base and an SMA surface, a design that preserves the base CRCP from the beginning, can reasonably be expected to provide even better service (for longer).

RECOMMENDATIONS

1. *VDOT's Materials Division and Location and Design Division should update anchorage details for CRCP in the respective VDOT standard.*

2. *VDOT's Materials Division and Construction Division should explore ways to minimize the construction damage to the pavement bases during construction when the existing base materials are designed to remain in place.*

3. *VDOT's Materials Division should use AASHTOWare Pavement ME Design for designing new composite pavements (AC over CRCP option in Pavement ME Design). The current design criteria for CRCP punchouts (maximum of 6 per mile) and rutting in an AC layer (0.26 in for 15-year design) for composite pavements should be used.*

4. *VDOT's Materials Division and Maintenance Division, along with VTRC, should monitor the long-term performance of the composite pavement on US 60, including the performance of the cold joints in CRCP.*

5. *VDOT should consider using composite pavements (SMA over CRCP) among its options for high truck traffic areas.*

IMPLEMENTATION AND BENEFITS

Implementation

With regard to Recommendation 1, the VDOT Road and Bridge Standard Sheet PR-3 and other similar sheets should be updated with a detailed instruction for end anchorage of CRCP. Since such construction is not common in VDOT, the VDOT Materials Division would undertake this task when a new CRCP construction is expected.

With regard to Recommendation 2, project-specific instructions in the contract document should be added to minimize the chances of base damage during construction. VDOT's Materials Division and Construction Division would undertake such a task when an existing concrete pavement is scheduled to be removed and replaced.

With regard to Recommendation 3, VDOT's *AASHTOWare Pavement ME User Manual* (VDOT, 2017) and the VDOT Materials Division's *Manual of Instructions* (VDOT, 2019) will be updated to include provisions for new composite pavement design. The update is expected to be completed within 1 year after the publication of this report.

With regard to Recommendation 4, VTRC will coordinate and collect performance data for this section for 20 years from VDOT's regular PMS data collection effort and share the data with the Materials Division.

With regard to Recommendation 5, VTRC and VDOT's Materials Division will continue to promote composite pavement through different venues such as the Materials Division's Pavement Forum.

Benefits

Updating the VDOT Road and Bridge Standard Sheet PR-3 with end anchorage details would eliminate any future issue during construction of a CRCP. Similarly, providing a clear instruction in the contract document to preserve the base during old concrete pavement removal would eliminate any unexpected and expensive base repair. Updating the Material Division's *Manual of Instructions* with a composite pavement design instruction would facilitate the use of long-life composite pavement as another option for VDOT engineers. A continuous monitoring of this composite pavement for 20 years would validate the long-life performance of such a system. A composite pavement system provides very good structural support through the rigid base of concrete and at the same time provides a good riding surface from asphalt. When CRCP is used as the base, it will take a long time for cracks to reflect in the overlay, as cracks stay tight in CRCP because of the use of reinforcement. Asphalt overlay provides a good control of temperature variations and will minimize the distresses in a pavement. SMA is usually less susceptible to rutting and cracking, so SMA over CRCP would be a good composite system. Although the reduction in thickness is possible because of the insulation of asphalt, composite pavement systems may initially cost more; however, they are durable, reduce traffic interruptions, and minimize inconveniences to the traveling public, making them cost-competitive.

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APPENDIX

VDOT CRCP PAVEMENT MAINTENANCE RECORD AND STRUCTURE DATA

Site No.	County	Route	Direction	State Milepost	CRCP Built, Year	Pavement Age (yr)	Overlay No.	Pavement or Overlay Age (yr)	Overlay Type	Overlay Life (yr)
1	Albemarle	I-64	East	107.42-114.33	1970	49	1	22 (1992)	2.9 in BM-2 + 1.4 in SM-2C	16
							2	16 (2008)	1.5 in SMA-9.5	11+
2	Albemarle	I-64	East	119.01-126.73	1970	49	1	26 (1996)	2.5 in IM-1A + 1.5 in SMA Surface	18
							2	18 (2014)	0.75 in THMACO	5+
3	Albemarle	I-64	East	126.75-129.89	1970	49	1	24 (1994)	3.0 in BM-2 + 1.5 in SM-2A	16
							2	16 (2010)	1.5 in SMA-9.5 (Mill & Fill)	2
							3	2 (2012)	0.3 in Latex Modified ST	7+
4	Albemarle	I-64	West	126.67-123.43	1970	49	1	26 (1996)	2.5 in IM-1A + 1.5 in SMA	22
							2	22 (2018)	2.0 in SMA 12.5 (Mill & Fill)	1+
5	Albemarle	I-64	West	123.43-120.23	1970	49	1	26 (1996)	2.5 in IM-1A + 1.5 in SMA	23+
6	Albemarle	I-64	West	119.00-114.35	1970	49	1	23 (1993)	3.0 in B-3 + 1.4 in SM-2C	6
							2	6 (1999)	1.5 in SMA Surface	18
							3	18 (2017)	2.0 in SMA 12.5 (Mill & Fill)	2+
7	Albemarle	I-64	West	114.3-107.48	1970	49	1	23 (1993)	3.0 in B-3 + 1.4 in SM-2C	7
							2	7 (2000)	1.5 in SM-9.5D	12
							3	12 (2012)	1.5 in SMA 9.5 (Mill & Fill)	7+
8	Louisa	I-64	East	131.16-136.06	1970	49	1	24 (1994)	3.0 in BM-2 + 1.4 in SM-2A	8
							2	8 (2002)	1.5 in SM-9.5A	10
							3	10 (2012)	0.3 in Latex Modified	5
							4	5 (2017)	2.0 in SMA 12.5 (Mill & Fill)	2+
9	Louisa	I-64	West	136.31-135.05	1987	32	1	18 (2005)	2.0 in SMA 19.0 + 1.5 in SMA 9.5	12
							2	12 (2013)	2.0 in SM-12.5D (Mill & Fill)	6+
10	Henrico	I-64	East	178.84-180.48	1967	52	1	38 (2005)	2.0 in SMA 19.0 + 1.5 in SMA 12.5	14+
11				183.42-185.91	1968	51	1	41 (2010)	2.0 in SMA19.0 + 1.5 in SMA 9.5	9+
12				191.51-195.89	1968	51	1	35 (2003)	2.0 in SMA 19.0 + 1.7 in SMA 12.5	16+
13	Henrico	I-64	West	181.54-178.58	1967	52	1	38 (2005)	2.0 in SMA 19.0 + 1.5 in SMA 12.5	14+
14				183.40-187.59	1968	51	1	42 (2010)	2.0 in SMA19.0 + 1.5 in SMA 9.5	9+
15				191.48-195.21	1968	51	1	35 (2003)	2.0 in SMA 19.0 + 1.7 in SMA 12.5	16+
16	New Kent	I-64	East	206.05-214.91	1973	46	1	20 (1993)	2.5 in IM-1B + 1.4 in SM-2C	13
							2	13 (2006)	0.2 Latex Modified	9
							3	9 (2015)	3.0 in SMA 19.0 + 1.5 in SMA 12.5 (M &F)	4+
17	New Kent	I-64	East	215.73-224.69	1972	47	1	18 (1990)	4.0 in IM-1B + 1.4 in SM-2C	16
							2	16 (2006)	0.2 Latex Modified	9
							3	9 (2015)	3.0 in SMA 19.0 + 2.0 in SMA 12.5 (M &F)	4+
18	New Kent	I-64	West	223.16-221.60	1972	47	1	18 (1990)	4.0 in IM-1B + 1.5 in SM-2C	22
							2	22 (2012)	3.0 in SMA 19.0 + 2.0 in SMA 12.5 (M &F)	7+

19	Norfolk	I-64	East	273.85-276.44	1975	44	1	40 (2015)	0.75 in THAMCO + 2.0 in SMA 19.0 + 1.75 in SMA 12.5	4+
20	Norfolk	I-64	East	293.10-300.01	1969	50	1	41 (2011)	2.0 in SMA 19.0 + 1.5 in SMA 12.5	8+
21	Norfolk	I-64	west	273.90-274.64	1974	45	1	41 (2015)	0.75 THMACO+2.0 in SMA 19.0 + 1.75 in SMA 12.5	4+
22	Norfolk	I-64	west	274.64-276.54	1975	44	1	40 (2015)	0.75 THMACO+2.0 in SMA 19.0 + 1.75 in SMA 12.5	4+
23	Dinwiddie	I-85	North	44.00-45.80	1969	50	1	48 (2017)	2.0 in SMA 19.0 + 1.5 in SMA 12.5	2+
24	Dinwiddie	I-85	North	55.73-61.44	1969	50	1	48 (2017)	2.0 in SMA 19.0 + 1.5 in SMA 12.5	2+
25	Sussex	I-95	North	17.14-22.29	1982	37	1	18 (1999)	3.0 in BM-3 + 2.0 in IM-1B + 1.4 in SM-12.5D	9
							2	9 (2008)	0.2 in Latex Modified	11+
26	Sussex	I-95	South	17.14-22.04	1982	37	1	18 (1999)	3.0 in BM-3 + 2.0 in IM-1B + 1.4 in SM-12.5D	
							2	9 (2008)	0.2 in Latex Modified	11+
27	Prince George	I-295	North	0.17-11.6	1992	27	1	25 (2017)	0.7 in THAMCO	2+
28	Prince George	I-295	North	12.99-13.83	1992	27	1	11 (2003)	2.0 in SMA 19.0 + 1.2 in SMA 12.5	16+
29	Prince George	I-295	South	12.63-9.17	1992	27	1	24 (2016)	0.7 in THAMCO + 2.0 in SMA 19.0 + 1.5 in SMA 12.5	3+
30	Chesterfield	I-295	North	15.14-17.24	1990	29	1	27 (2017)	0.7 in THAMCO	2+
31	Chesterfield	I-295	South	15.01-17.06	1990	29	1	26 (2016)	0.7 in THAMCO + 2.0 in SMA-19.0 + 1.50 in SMA-12.5	3+
32	Hanover	I-295	North	31.92-36.15	1980	39	1	16 (1996)	2.0 in IM-1A + 1.50 in SMA-Surface	23+
33	Hanover	I-295	North	38.20-42.07	1980	39	1	27 (2007)	2.0 in IM-19.0D + 2.0 in SMA 12.5	12+
34	Hanover	I-295	South	36.07-31.79	1980	39	1	16 (1996)	2.0 in IM-1A + 1.50 in SMA-Surface	23+
35	Hanover	I-295	South	37.47-36.21	1980	39	1	25 (2005)	2.0 in IM-19.0D + 1.5 in SM 12.5D	14+
36	Hanover	I-295	South	39.76-37.89	1980	39	1	30 (2010)	2.0 in SMA 19.0 + 1.5 in SMA 9.5	9+
37	Hanover	I-295	South	42.03-40.31	1980	39	1	23 (2003)	2.0 in SMA 19.0 + 1.5 in SMA 12.5	16+
38	Henrico	I-295	North	28.86-31.64	1980	39	1	19 (1999)	2.0 in IM-1D + 1.5 in SM-2D	17
							2	17 (2016)	2.0 in SMA 12.5 (Mill & Fill)	3+
39	Henrico	I-295	North	42.45-51.93	1980	39	1	27 (2007)	2.0 in IM-1D + 2.0 in SMA-12.5D	12+
40	Henrico	I-295	South	47.35-45.98	1980	39	1	18 (1998)	2.0 in IM-1D + 1.5 in SM-2D	21+
41	York	I-664	East	1.08-3.26	1983	36	1	32 (2015)	0.7 in THAMCO	4+
42	Nansemond	I-664	East	10.89-14.23	1991	28	1	22 (2013)	0.7 in THAMCO	4
							2	4 (2017)	2.0 in SMA 19.0 + 1.5 in SMA 12.5	2+
43	Norfolk	I-664	East	14.25-18.04	1991	28	1	22 (2013)	0.7 in THAMCO	4
							2	4 (2017)	2.0 in SMA 19.0 + 1.5 in SMA 12.5	2+
44	Chesterfield	SR 76	North	0.87-7.21	1988	31	1	29 (2017)	0.75 in THAMCO	2+
45	Chesterfield	SR 76	South	1.13-3.30	1988	31	1	27 (2015)	2.0 in IM-19.0D + 1.5 in SM-9.5E	4+
46	Chesterfield	SR 76	South	3.70-4.50	1988	31	1	27 (2015)	2.0 in IM-19.0D + 1.5 in SM-9.5E	4+
47	Chesterfield	SR 76	South	4.97-7.78	1988	31	1	28 (2016)	0.7 in THMACO	3+
48	Chesterfield	SR 288	North	0.63-8.52	1990	29	1	26 (2016)	1.5 in SMA 9.5	3+
49	Chesterfield	SR 288	North	12.51-13.56	1990	29	1	24 (2014)	4.0 in SMA 19.0 + 1.5 in SMA 12.5	5+

50	Chesterfield	SR 288	North	13.56-16.11	1988	31	1	26 (2014)	4.0 in SMA 19.0 + 1.5 in SMA 12.5	5+
51	Chesterfield	SR 288	South	1.33-7.85	1990	29	1	26 (2016)	2.0 in 19.0 SMA + 1.5 in SMA 9.5	3+
52	Chesterfield	SR 288	South	13.63-15.21	1988	31	1	27 (2015)	2.0 in 19.0 SMA + 1.5 in SMA 9.5	4+
53	Southampton	US 58	East	423.17-431.88	1990	29	1	22 (2012)	3.0 in Milling+ 3.0 in 19.0 SMA + 2.0 in SMA 12.5	3+
54	Henrico	US 60	West	200.64-201.56	1979	40	1	37 (2016)	1.5 in SM-12.5E	3+
55	Henrico	US 60	West	199.29-200.39	2017	2	New Comp		15 in CTA + 8.0 in CRCP + 2.0 in SMA 12.5	