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I-81 In-Place Pavement Recycling Project

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16. Abstract:

During the 2011 construction season, the Virginia Department of Transportation (VDOT) completed an in-place pavement recycling project to rehabilitate a section of pavement on I-81 near Staunton, Virginia. The project consisted of a 3.66-mile section of southbound I-81 in Augusta County. VDOT employed three in-place pavement recycling techniques and a unique traffic management plan to accomplish the work. The recycling processes included full-depth reclamation (FDR), cold in-place recycling (CIR), and cold central-plant recycling (CCPR). This project marked the first time in the United States that these three recycling techniques were combined in one project on the interstate system.

Materials for both the CIR and CCPR were produced using hydraulic cement and foamed asphalt. A combination of hydraulic cement and lime kiln dust was chosen for the FDR process.

The purpose of this research portion of this construction project was threefold: (1) to allow VDOT personnel to gain experience with the specific laboratory mix designs, field evaluation, and quality assurance procedures; (2) to characterize the structural properties of the materials used in the recycling project; and (3) to document the performance of the entire rehabilitated section during its initial 3-year service period.

Various laboratory tests were conducted on materials collected before, during, and after construction to characterize the materials. These tests included gradation, resilient modulus, indirect tensile strength, dynamic modulus, and flow number. Additional tests to document the performance of the project included ride quality testing and rut-depth measurements collected via a traffic-speed profiler; pavement layer thickness measurements by ground penetrating radar; and structural capacity measurements by the falling weight deflectometer.

From the results of this study, the combined structural layer coefficient for the CCPR and FDR materials was calculated as 0.37. The structural layer coefficient for the CIR material was calculated as 0.39. The structural layer coefficient for the CCPR material was calculated to have a likely range of 0.37 to 0.44. Laboratory testing showed that the performance of the CCPR and CIR materials is expected to be similar. The field performance tests demonstrated that the section of pavement rehabilitated by the three in-place recycling methods continues to perform well after nearly 3 years of high-volume interstate traffic.

This study recommends that VDOT pursue in-place recycling where it is most suitable. The study also recommends that VDOT consider increasing the structural layer coefficients used in the design for recycled materials. Further, VDOT should continue to monitor the performance of the I-81 project and other in-place recycling projects in an effort to develop long-term performance data. Finally, the study recommends that VDOT consider using long-term lane closure strategies similar to those employed in this project on other major pavement rehabilitation projects.

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FINAL REPORT

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ABSTRACT

During the 2011 construction season, the Virginia Department of Transportation (VDOT) completed an in-place pavement recycling project to rehabilitate a section of pavement on I-81 near Staunton, Virginia. The project consisted of a 3.66-mile section of southbound I-81 in Augusta County. VDOT employed three in-place pavement recycling techniques and a unique traffic management plan to accomplish the work. The recycling processes included full-depth reclamation (FDR), cold in-place recycling (CIR), and cold central-plant recycling (CCPR). This project marked the first time in the United States that these three recycling techniques were combined in one project on the interstate system.

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This study recommends that VDOT pursue in-place recycling where it is most suitable. The study also recommends that VDOT consider increasing the structural layer coefficients used in the design for recycled materials. Further, VDOT should continue to monitor the performance of the I-81 project and other in-place recycling projects in an effort to develop long-term performance data. Finally, the study recommends that VDOT consider using long-term lane closure strategies similar to those employed in this project on other major pavement rehabilitation projects.

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INTRODUCTION

In-place pavement recycling includes various paving technologies that can restore the structural capacity of distressed pavements while using little to no virgin materials. In general, in-place pavement recycling re-mixes the in-situ pavement material and reuses it in the final pavement structure. Various in-place recycling techniques have been successfully demonstrated by many highway agencies (Mallick et al., 2002a, b; Mohammad et al., 2003; Romanoschi et al., 2004; Saleh, 2004; Wen et al., 2004; Lane and Kazmierowski, 2005; Bemanian et al., 2006; Lewis et al., 2006; Berthelot et al., 2007; Guthrie et al., 2007; Maurer et al., 2007; Hilbrich and Scullion, 2008; Diefenderfer and Apeagyei, 2011a, b). Benefits include reduced use of virgin materials, reduced fuel consumption, reduced lane closures, and reduced emissions related to construction (Nataatmadja, 2001; Thenoux et al., 2007; Stroup-Gardiner, 2011) and large cost savings resulting in the ability of highway agencies to stretch available funding for pavement rehabilitation.

The Nevada Department of Transportation, in particular, has shown large cost savings by employing cold in-place recycling (CIR) and full-depth reclamation (FDR) on their pavement network. Bemanian et al. (2006) reported cost savings in the hundreds of millions of dollars when in-place recycling techniques were compared to traditional reconstruction practices.

Despite the wealth of experience by many agencies (including some on higher volume facilities), in-place recycling techniques have primarily been viewed as a pavement rehabilitation program suitable for only lower volume roadways. In addition, only a few studies document the performance beyond the first year of construction. The lack of reliable performance data has contributed to making in-place recycling techniques unlikely to be specified by pavement engineers.

During the 2011 construction season, the Virginia Department of Transportation (VDOT) completed an in-place pavement recycling project to rehabilitate a section of pavement on I-81 near Staunton, Virginia. The project consisted of a 3.66-mile section of I-81 in Augusta County. A construction contract was awarded in December 2010 with a value of $7.64 million and a timeframe of approximately 8 months. The construction project used three in-place pavement recycling techniques (CIR, CCPR, and FDR) and a unique traffic management plan to accomplish the work. CIR and FDR are the more commonly used in-situ pavement recycling techniques; CCPR is a process wherein milled pavement materials can be processed in a mobile plant at a central location (Asphalt Recycling and Reclaiming Association [ARRA], 2001). The CCPR and CIR materials were produced using a hydraulic cement content of 1.0% and a foamed asphalt content of 2.0%. A combination of hydraulic cement and lime kiln dust was chosen as the stabilizing agent for the FDR portion of the project at a dosage rate of 3.0%. The in-place recycling portion of the work was completed in fewer than 20 workdays spanning 6 weeks (from April 16 to May 24, 2011) and marks the first time in the United States that these three recycling techniques were combined on one project on the interstate system. The contracted work also consisted of adjusting guardrail to meet new standards; adding longitudinal edge-drains (prefabricated vertical fin drains) on each side of the pavement; and correcting the pavement surface cross slope.

The project was located on southbound I-81 between Mileposts 217.66 and 214.00. The interstate in this section is a four-lane divided highway having a paved 2- to 3-ft inside shoulder and a paved 8- to 10-ft outside shoulder. The original pavement was constructed in 1967 and 1968 and consisted of asphalt concrete (AC) over an aggregate base. During original construction, the natural subgrade was capped by locally available borrow material. The project location is situated in an area of rolling hills; thus, there are numerous cut and fill sections within the project limits. The "before" pavement structure consisted of approximately 12 in of AC over an aggregate base having a thickness ranging from 10 to 12 in. In 2008, VDOT showed a directional traffic volume of approximately 23,000 vehicles per day with 28% trucks (85% of the truck traffic consisted of five- and six-axle tractor-trailer combination vehicles) (VDOT, 2007a). Prior to construction, VDOT's annual network-level condition survey (using continuous digital imaging and automated crack detection technology) identified this section of I-81 as having frequently recurring structurally related distresses resulting in deep patching and AC mill and inlays, despite regular periodic maintenance. A thorough project-level investigation of this section confirmed that it was a candidate for reconstruction (Weaver and Clark, 2007).

PURPOSE AND SCOPE

The purpose of this study on pavement recycling methods was threefold: (1) to allow VDOT personnel to gain experience with the laboratory mix design, field evaluation, and quality assurance (QA) procedures; (2) to characterize the structural properties of the materials used; and (3) to document the performance of the project during its initial service (3-year period following rehabilitation). The information collected from the laboratory and field evaluation was expected to establish a baseline for future performance evaluation and populated an initial materials characterization database. In addition, this information will help VDOT's understanding of the materials behavior for the design and specification of future projects.

The scope of this study consisted of the section of southbound I-81 located between Mileposts 217.66 and 214.00 in Augusta County, Virginia, that was rehabilitated during VDOT's 2011 construction season. Laboratory testing prior to construction, during the mix design phase, was completed using indirect tensile strength (ITS) testing. Following construction, cores were collected within approximately 3 months and again approximately 20 months after construction and were tested to determine gradation, resilient modulus, ITS, dynamic modulus, and flow number (FN). Additional tests to document the performance of the project included ride quality testing and rut depth measurements by a traffic-speed profiler; layer thickness measurements by ground penetrating radar (GPR); and structural capacity measurements by a falling weight deflectometer (FWD).

METHODS

Literature Review

A literature review was conducted by searching various databases related to transportation engineering such as the Transport Research International Documentation (TRID) bibliographic database, the catalog of Transportation Libraries (TLCat), the Catalog of Worldwide Libraries (WorldCat), and the Transportation Research Board Research in Progress (RiP) and Research Needs Statements (RNS) databases.

Preconstruction Testing and Construction Description

Details of the pavement condition prior to construction and the laboratory mix design procedure are presented. Representatives from the Virginia Center for Transportation Innovation and Research (VCTIR) were present during most of the construction of the I-81 in-place pavement recycling project. The processes used to construct the pavement are described in further detail in this report.

Laboratory Evaluation

The laboratory testing program consisted of testing that was conducted on specimens (1) fabricated in the laboratory from loose materials collected during construction, and (2) obtained from cores collected after construction. During construction, laboratory-prepared specimens were primarily tested for ITS to monitor construction quality, with a limited number of FN tests included. Within the first 3 months after construction of the recycled layers was completed (but prior to the application of the final AC overlay), a total of 61 cores were collected from the right and left lanes and were tested to determine gradation, resilient modulus, ITS, and FN of the CCPR and CIR materials. In addition, 13 cores were collected approximately 20 months after construction. Cores collected at 20 months were tested to measure the dynamic modulus and FN of the CCPR and CIR materials. Different tests were performed on the second set of cores as new equipment became available in the VCTIR laboratory.

Laboratory-Prepared Specimens: Compaction and Curing

During construction of the right and left lanes, loose samples of the CIR and CCPR materials were collected on May 2 and 21, 2011 (right lane), and June 4, 2011 (left lane). The collected materials were sealed in plastic buckets and brought to the laboratory. The loose mixture was compacted in a gyratory compactor using a 6-in-diameter mold to a density that was similar to the in-place compacted density. The specimens were compacted to a density, rather than a particular number of gyrations, in an effort to simulate better the actual field conditions from construction. For certain specimens, particles larger than ¾ in were removed by sieving prior to specimen fabrication. This was done because only materials smaller than ¾ in were used by the contractor during the mix design stage and preparation of 4-in-diameter test specimens would be facilitated. Care was taken to compact the mixtures within 3 to 5 hours after field sampling to ensure that the sampled materials did not dry out and the cement did not completely hydrate prior to fabrication of the test specimens.

Once compacted, the 6-in-diameter samples were placed in a forced-draft oven at 40°C for 72 hours prior to creation of the 4-in-diameter specimens. The specimens were not weighed to determine if constant mass had been achieved. This test procedure is typically used for materials stabilized with foamed asphalt (Asphalt Academy, 2009). This oven drying step was done in an effort to prevent damage to the specimen during fabrication. Since the coring for specimen fabrication was performed using water, the completed test specimen was again placed in an oven at 40°C for 72 hours after the ends were trimmed but prior to testing. It is unknown if the two rounds of oven drying changed the test properties of the cores. The cured specimens were stored at room temperature until testing (ranging from approximately 2 to 4 weeks to several months). Accelerated curing of foamed asphalt specimens at 40°C is standard industry practice; however, there is no specimen fabrication specification.

Field Core Specimen Preparation

Field cores were obtained approximately 3 months and 20 months after construction. The sampling locations for the 3-month cores were chosen to coincide with those areas that were found deficient in the QA testing (primarily having deficient ITS from specimens produced by collecting loose materials during construction). Cores collected 3 months after construction were taken between the wheel paths of the right and left lanes. The timing of these cores was approximately 4 weeks after the recycled layer had been overlaid with the AC intermediate layer but prior to application of the surface AC course. Cores collected approximately 20 months after construction were collected from between the wheel paths in both the right and left lanes.

During core collection at 3 months after construction, a 4-in-diameter core barrel was initially used to obtain cores, as most tests required a 4-in-diameter specimen. This was problematic, as the lower portions of the core began to disintegrate around the edges. Similar problems have been reported in the past (Chen et al., 2006), and the cause of sample disintegration has been attributed to the pressure of water used during coring and the size of the core drilling bit. After a few unsuccessful attempts, a 6-in-diameter core barrel was used to collect the remaining cores at 3 and 20 months after construction using reduced water pressure.

With the 6-in-diameter core barrel, it was easier to obtain sound samples of sufficient size to fabricate the required 4-in-diameter test specimens in the laboratory.

All the cores that were retrieved included the overlying AC layers. All test specimens obtained from cores were prepared by placing the cores in a forced-draft oven at 40°C for 72 hours prior to creating the 4-in-diameter specimens from the 6-in-diameter cores. This was done in an effort not to damage the core while fabricating the test specimen. Since the coring was performed using water, the completed test specimen was again placed in a forced-draft oven at 40°C for 72 hours after the ends were trimmed, but prior to testing. The same accelerated curing procedure was followed for the laboratory-prepared and field core specimens.

The primary concern at the onset of laboratory testing was that the recycled materials could be susceptible to deterioration by rutting. As a result, much of the early testing centered on assessing the rutting susceptibility by determining the FN. ITS testing was also performed for comparison with the results of construction QA tests and the results obtained during the mix design phase. Subsequent cores collected at approximately 20 months after construction focused on dynamic modulus and FN testing of the materials after nearly 2 years of service under traffic.

Laboratory-Prepared and Field Core Specimen Testing

Gradation and Binder Content Tests

The binder content was measured using the ignition oven method, conducted in accordance with AASHTO T 308-10, Standard Method of Test for Determining the Asphalt Binder Content of Hot-Mix Asphalt (HMA) by the Ignition Method (American Association of State Highway and Transportation Officials [AASHTO], 2013), which was also used to produce material for gradation analysis. Gradation analysis was conducted in accordance with AASHTO T 27-11, Standard Method of Test for Sieve Analysis of Fine and Coarse Aggregates (AASHTO, 2013). For each sample, about 2,400 grams of materials were used for the ignition oven test. The results of aggregate gradation tests were evaluated with respect to recommendations by Wirtgen GmbH (Wirtgen) (2010), although those recommendations prescribe pre-stabilized materials and the gradation presented herein was determined following stabilization. According to Wirtgen (2010), aggregate for cold recycling should pass certain gradation requirements, the most critical being the percent passing the 0.075-mm sieve. The materials passing the 0.075-mm sieve are necessary for dispersion of the tiny foam droplets that are critical for binding the foamed material together. Wirtgen (2010) recommended that 2% to 9% of material pass the 0.075-mm sieve. The binder content was determined as the difference between the mass of material before and after ignition. The measured binder content included both the original binder and the binder added during the recycling process.

Bulk Density

The bulk density of test specimens was calculated by dividing the mass of the specimen by the nominal volume and was used to compare the level of compaction achieved on the CIR and CCPR sections of the project and to compare the influences of bulk density on the ITS and M_R test results. The variation of bulk density with depth was evaluated by comparing the bulk density of the upper portions (top) and lower portions (bottom) of each core. Top and bottom portions of the core were obtained by sawing the core.

Indirect Tensile Strength Tests

ITS tests were conducted in accordance with AASHTO T 283-07, Standard Method of Test for Resistance of Compacted Hot Mix Asphalt (HMA) to Moisture-Induced Damage (AASHTO, 2013). Test specimens measuring 4 in in diameter by 63 mm in thickness (approximately 1.2 in) were fabricated by coring 4-in-diameter specimens from the center of the 6-in-diameter field cores using a wet core drill bit. The ends were trimmed with a dry masonry saw. The cored and trimmed specimens were then dried in a forced-draft oven maintained at 40°C for 72 hours. The specimens were weighed, their height and diameter were measured, and the results were used to estimate the bulk density of the cores. From each core, only a single ITS specimen was created from the upper portion of the core because the cores were not thick enough to make two 63-mm-thick specimens.

Resilient Modulus (M_R) Tests

Resilient modulus tests were conducted in accordance with ASTM D7369-09, Standard Test Method for Determining the Resilient Modulus of Bituminous Mixtures by Indirect Tension Test (ASTM, Inc. [ASTM], 2013a), using 36 2-in-thick by 6-in-diameter specimens produced from 18 cores. The M_R tests were conducted using the same specimens tested for density as previously described. All the M_R tests were conducted using a servo-hydraulic universal testing machine with a 100 kN capacity. The tests were conducted at three temperatures, 4°C, 20°C, and 38°C, at a loading frequency of 10 Hz. Test specimens were created from the field cores by slicing the cores into two approximately 2-in-thick specimens. The cut specimens then were placed in a forced-draft oven at 40°C for 72 hours to dry. The specimens were weighed, their height and diameter were measured, and the results were used to estimate the bulk density of the cores.

The test specification calls for applying a load along the diametral axis of a disk-shaped specimen. The magnitude of the applied load was selected to ensure the resilient strain that developed in each specimen stayed within the linear viscoelastic range (approximately 75 to 125 microstrain). Four displacement sensors were used to monitor vertical and horizontal displacements resulting from the applied load. Two displacement sensors (one vertical and one horizontal) were affixed to each side of the test specimen. A gauge length of 38 mm was used. The M_R was computed using the peak applied load and measured vertical and horizontal displacements over the 38-mm gauge length.

Flow Number Tests

Rutting susceptibility was assessed by determining the FN in accordance with AASHTO TP 79-09, Standard Method of Test for Determining the Dynamic Modulus and Flow Number for Hot Mix Asphalt (HMA) Using the Asphalt Mixture Performance Tester (AMPT) (AASHTO, 2013), using a universal testing machine on laboratory-compacted specimens and specimens fabricated from field cores. Kim et al. (2009) showed that the FN test was able to evaluate the rutting potential of recycled asphalt mixtures. All tests were conducted at a temperature of 54°C using a repeated haversine axial compressive load pulse of 0.1 sec followed by a 0.9-sec rest period. This test temperature represents the 50% reliability high pavement temperature as determined using LTPPBind software (LTTPBind, 2005) for Virginia. The tests were conducted to 10,000 load cycles or until a permanent strain in the test specimen reached 50,000 microstrain, whichever occurred first. During the test, permanent axial strain and cumulative number of load cycles were recorded automatically, and the results were used to calculate the FN.

FN tests were performed in the unconfined mode with 30 psi deviator stress and in the confined mode with 70 psi deviator stress plus 10 psi confining pressure. Similar testing conditions have been used for FN testing of AC (Apeagyei et al., 2011; Bonaquist, 2010; Mohammad et al., 2006). Therefore, the loading conditions used in the current study were selected so that a comparison could be made with previously conducted AC mixture testing. In addition, cored samples were tested at 70 psi deviator stress with 0 psi confining pressure.

Dynamic Modulus (|*E**|) *Tests*

The dynamic modulus describes the stress-strain relationship for a linear viscoelastic material. The test is conducted by subjecting a cylindrical specimen to an axial compressive sinusoidal load at a range of temperatures and loading frequencies. The dynamic modulus of specimens cut from field cores was measured using an asphalt mixture performance tester (AMPT). Testing was conducted in accordance with AASHTO TP 79-09, Standard Method of Test for Determining the Dynamic Modulus and Flow Number for Hot Mix Asphalt (HMA) Using the Asphalt Mixture Performance Tester (AMPT) (AASHTO, 2013), at temperatures of 4.4, 21.1, 37.8, and 54.4°C; at each temperature, testing was conducted at frequencies of 0.1, 0.5, 1, 5, 10, and 25 Hz. Testing was performed on specimens obtained by coring after construction.

Field Evaluation

Rut Depth and Ride Quality

Through a contract with a third-party vendor, rutting and ride quality data were simultaneously collected with vehicle-mounted sensors on an inertial profiler operated at highway speeds. Data were collected in accordance with ASTM E950, Standard Test Method for Measuring the Longitudinal Profile of Traveled Surfaces with an Accelerometer Established Inertial Profiling Reference (ASTM, 2013a); AASHTO R 43-07, Standard Practice for Determination of International Roughness Index (IRI) to Quantify Roughness of Pavements (AASHTO, 2013); and AASHTO R 48-10, Standard Practice for Determining Maximum Rut Depth in Asphalt Pavements (AASHTO, 2013). The data were reported from the vendor at 0.01- and 0.1-mile intervals. These data were collected at approximately 5, 9, 12, 16, 23, 28, and 34 months after construction.

Layer Thickness

GPR was used to assess the layer thickness of the recycling project. This technique has been shown to be an effective means for nondestructively determining the pavement layer thickness (Maser and Scullion, 1992; Maser, 2002). Testing was conducted in accordance with ASTM D4748-10, Standard Test Method for Determining the Thickness of Bound Pavement Layers Using Short-Pulse Radar (ASTM, 2013a).

The GPR system used in this study consisted of a 2.0-GHz air-launched horn antenna and an SIR-20 controller unit, both manufactured by Geophysical Survey Systems, Inc. (GSSI). The pulse rate of the antenna was maintained at a rate of 1 scan per 2 ft, regardless of the vehicle speed, using an integrated distance measuring instrument. All data were processed by the software RADAN (Version 6.6) developed by GSSI. The software allows the user to view the collected data and identify the layer boundaries. The thickness to each layer boundary is automatically calculated.

Structural Capacity

Deflection testing to assess structural capacity was performed using a Dynatest Model 8000 FWD in accordance with ASTM D4694-09, Standard Test Method for Deflections with a Falling-Weight-Type Impulse Load Device (ASTM, 2013a). Testing was conducted in the right and left lanes at approximately 4, 15, and 28 months after construction. The FWD load plate was located within the right wheel path of each lane during testing. The FWD was equipped with nine sensors at radial distances of 0, 8, 12, 18, 24, 36, 48, 60, and 72 in from the center of a load plate. Deflection testing was conducted at approximately 250-ft intervals and at three load levels (6,000; 9,000; and 12,000 lbf). Following two unrecorded seating drops, three deflection basins were recorded at each load level.

The deflection data were analyzed in accordance with the 1993 AASHTO *Guide for Design of Pavement Structures* (AASHTO, 1993). Deflection data were analyzed using ModTag, Version 4.1.9 (VDOT, 2007b). The pavement sections were analyzed by evaluating the effective structural number (SN_{eff}) values, the temperature-corrected deflection (D_0), the subgrade resilient modulus (M_R), and the pavement modulus (E_p). The central deflections (D_0) were corrected for temperature using the previous day's average air temperature (average of high and low) that was obtained from a nearby weather station by Weather Underground (n.d.). The pavement was modeled in ModTag as a flexible pavement structure. In addition, the structural layer coefficient was determined for the recycled materials in both the right and left lanes.

RESULTS AND DISCUSSION

Literature Review

Pavement recycling is a technology that can be used to rehabilitate or reconstruct pavements by pulverizing the existing pavement material, mixing it with a recycling or stabilizing agent (or agents), and reusing it in the final pavement in the form of a stabilized base. Some of the most commonly cited benefits of using pavement recycling techniques to rehabilitate and repair AC pavements include reduction in the use of virgin materials, reduced fuel consumption, reduced lane closures, and reduced emissions related to construction (Nataatmadja, 2001; Thenoux et al., 2007; Stroup-Gardiner, 2011). Pavement recycling methods include the following processes: cold planing, hot in-place recycling, cold recycling, and FDR. Cold recycling includes the techniques CIR and CCPR. The two principal asphalt-based recycling technologies used to produce a recycled layer are asphalt emulsion and foamed asphalt. The Asphalt Academy (2009) terms the material stabilized by asphalt emulsion or foamed asphalt *bitumen stabilized material* (BSM), and this term is used herein generally to describe asphalt-based FDR, CIR, and CCPR.

FDR is used to correct severe structural deficiencies and defects that are deep within the pavement structure. The depth of pulverization depends on the thickness of the bound layers of the existing pavement but is typically 4 to 12 in (ARRA, 2001). FDR is performed on the bound layers and a portion of the underlying unbound materials. FDR may consist of simply pulverizing and remixing the roadway foundation, termed *mechanical stabilization,* but it most often also includes introducing one or several stabilizing agents, termed *chemical stabilization.* A list of typical FDR stabilizing agents includes active fillers (e.g., lime, fly ash, cement, cement and lime kiln dust) and asphalt-based recycling agents (e.g., asphalt emulsion and foamed asphalt) (ARRA, 2001). The most commonly used stabilizing agents are foamed or emulsified asphalt binder, lime, or cement (Wirtgen, 2010). Active fillers may often be combined with asphalt-based stabilizers to improve the resistance to the detrimental effects of moisture and improve the early strength. For higher volume routes, an AC overlay is usually applied after the FDR layer has been allowed to cure. FDR has been successfully demonstrated by highway agencies in several states and countries (Mallick et al., 2002a, b; Mohammad et al., 2003; Romanoschi et al., 2004; Saleh, 2004; Bemanian et al., 2006; Lewis et al., 2006; Berthelot et al., 2007; Guthrie et al., 2007; Loizos, 2007; Maurer et al., 2007; Hilbrich and Scullion, 2008; Diefenderfer and Apeagyei, 2011a, b).

CIR is a process used to rehabilitate the upper portions of the bound layers of an asphalt pavement and is typically performed at depths of 2 to 6 in (ARRA, 2001). The CIR process is most commonly performed using a train of equipment that often includes the following: a tanker, a cold recycler, a paver, and rollers. Typical recycling agents for CIR include lime, fly ash, cement, cement and lime kiln dust, asphalt emulsion, and foamed asphalt (ARRA, 2001). Active fillers may often be combined with asphalt-based stabilizers to improve the resistance to the detrimental effects of moisture and to improve the early strength. On higher volume routes, an AC overlay is typically applied, but non-structural treatments (such as chip seals) may be used on lower volume facilities (Bemanian et al., 2006; Maurer et al., 2007). CIR has been successfully demonstrated by many agencies in projects worldwide (Crovetti, 2000; Forsberg et al., 2002; Lane and Kazmierowski, 2005; Bemanian et al., 2006; Loizos et al., 2007; Diefenderfer et al., 2012).

CCPR is a process in which the recycled material is milled from a roadway and brought to a centrally located recycling plant that incorporates the recycling agents into the material. The benefits of this process are primarily two-fold. First, material can be removed from the roadway and stockpiled to be used as a recycled layer while at the same time the underlying foundation can be either stabilized or replaced if needed. Second, existing stockpiles of reclaimed asphalt pavement (RAP) can be treated and used in the construction of new pavement or in the rehabilitation of existing pavement.

Stabilizing Mechanisms

The CCPR and CIR materials produced on the I-81 pavement recycling project employed foamed asphalt as the primary recycling agent. For this reason, this section focuses more on foamed asphalt than on asphalt emulsion.

The strength-related properties of foamed asphalt BSMs differ from those of an AC as a result of the mechanisms of stabilization. Since the binder droplets are not linked and the larger aggregate particles are not always coated, BSMs retain the strength characteristics of the parent granular materials, albeit with improved cohesion and moisture resistance (Asphalt Academy, 2009; Wirtgen, 2010). The interparticle bonding of a foamed asphalt BSM could be described as being "non-continuous"; thus, the material acts more like a granular material and is often treated as such during and even after construction. Recent studies have begun to show, however, that regarding BSMs as simply an improved granular pavement component may be selling their full potential short (Thomas and May, 2007; Apeagyei and Diefenderfer, 2013; Schwartz and Khosravifar, 2013).

The Asphalt Academy (2009) described foamed asphalt as a material that is produced by injecting water and air into a hot asphalt binder such that spontaneous foaming results. Foam is produced when hot asphalt binder turns the water into a vapor and traps thousands of tiny bubbles within the binder. When these bubbles burst during the process of mixing with aggregate/RAP, the foam droplets are dispersed by adhering to the finer particles to form a mastic. The process of compaction forces the mastic to be pressed against larger aggregate particles to form localized non-continuous points of bonding. The Asphalt Academy (2009) describes the bonding of particles in a foamed asphalt–based BSM as "spot welds" consisting of a mastic of foam droplets and fines.

Fillers

Fillers are often used in combination with asphalt emulsion or foamed asphalt to improve the engineering properties of BSMs (Hodgkinson and Visser, 2004; Saleh, 2004, 2006; Fu et al., 2008; Franco et al., 2009; Halles and Thenoux, 2009; Fu et al., 2010a). Fillers are classified as either *active* or *natural* fillers, with active fillers being those that chemically affect the properties of the BSM. Active fillers typically include hydraulic cement, hydrated lime, and fly ash; natural fillers include rock dust. The Asphalt Academy (2009) stated that active fillers are included in BSMs to promote the adhesion of the asphalt binder to the aggregate; improve the dispersion of the foam droplets within foamed asphalt BSMs; improve (i.e., reduce) the plasticity index of certain materials (especially by pretreating with hydrated lime); increase the stiffness at early ages; increase the rate of strength gain; and increase the resistance to moisture.

Curing

BSMs, whether produced in the laboratory or in the field, undergo a process in which the materials develop stiffness with respect to time. Many researchers have worked to describe the results of this process either in the field or the laboratory (e.g., Kazmierowski et al., 1999; Mallick et al., 2002b; Loizos and Papavasiliou, 2006; Fu et al., 2010b; Bocci et al., 2011; Diefenderfer and Apeagyei, 2011a, b). Those studies describing testing on in-service construction projects have shown that BSMs can achieve large gains in stiffness from early ages up to approximately 2 years after construction. Other laboratory studies have investigated the feasibility of replicating the field curing processes in the laboratory setting using accelerated methods.

The processes related to curing of a BSM following construction are also not well understood, neither are the potential ramifications of accelerated curing in the laboratory on the strength-related properties of BSMs. Fu et al. (2010b) stated that bonding using foamed asphalt develops as mixing/compaction water evaporates (or as curing progresses). If moisture is reintroduced after the bonds are formed, their strength appears to be only partially damaged. However, if this moisture is prevented from evaporating (thus, preventing the BSM from curing properly), the bonds may not develop, even after extended periods of time.

Laboratory Testing

The most common laboratory tests for foamed asphalt and asphalt emulsion BSMs during mix design include density and moisture relationships in accordance with AASHTO T 180-10, Standard Method of Test for Moisture-Density Relations of Soils Using a 4.4-kg (10-lb) Rammer and a 457-mm (18-in.) Drop, Method D (AASHTO, 2013); ITS in accordance with AASHTO T 283-03, Standard Method of Test for Resistance of Compacted Hot-Mix Asphalt (HMA) to Moisture-Induced Damage (AASHTO, 2013); and Marshall stability in accordance with ASTM D5581-07a, Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus (6 in-Diameter Specimen) (ASTM, 2013a), and AASHTO T 245-13, Standard Method of Test for Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus (4 inch-Diameter Specimens) (AASHTO, 2013). Additional tests may include gradation analysis in accordance with AASHTO T 27-11, Standard Method of Test for Sieve Analysis of Fine and Coarse Aggregates (AASHTO, 2013); moisture susceptibility (as measured by the retained ITS [AASHTO T 283-03] or Marshall stability [ASTM D5581-07a] after soaking); and unconfined compressive strength in accordance with ASTM D1633-00, Standard Test Method for Compressive Strength of Molded Soil-Cement Cylinders (ASTM, 2013b). These tests have been used for many years and thus have the advantage of a substantial history that can be used to estimate whether a given project will perform as anticipated after construction.

The main drawback is that most of these test methods do not provide direct inputs for currently used pavement design procedures, whether they are empirically based or mechanistic-empirical. Commonly reported mechanistic tests for BSMs are similar to those now used for AC materials and include dynamic modulus and permanent deformation resistance in accordance with AASHTO TP 79-09, Standard Method of Test for Determining the Dynamic Modulus and Flow Number for Hot Mix Asphalt (HMA) Using the Asphalt Mixture Performance Tester

(AMPT) (AASHTO, 2013); resilient modulus in accordance with ASTM D7369-11, Determining the Resilient Modulus of Bituminous Mixtures by Indirect Tension Test (ASTM, 2013a); and creep compliance in accordance with AASHTO T 322-03, Standard Method of Test for Determining the Creep Compliance and Strength of Hot-Mix Asphalt (HMA) Using the Indirect Tensile Test Device (AASHTO, 2013). However, the results of only some of these tests provide direct inputs for pavement structural design procedures.

It is difficult to report representative examples since the results of strength-related testing can vary widely depending on many parameters, including source materials, recycling/stabilizing agent type and percentage, and curing regime employed. From the literature, it is seen that ITS and resilient modulus have been the most popular tests to describe the strength-related properties of BSMs. However, the results of dynamic modulus and permanent deformation resistance tests have been regularly reported over the past 6 to 7 years. As the briefest of examples, average test results for dynamic modulus of BSMs at 21°C and 10 Hz have been reported as ranging from approximately 530,000 psi to 630,000 psi (Lee and Im, 2008; Lee et al., 2009a). The average resilient modulus of BSMs at 20°C from materials produced in Virginia have been reported as ranging from approximately 430,000 psi to 630,000 psi (Diefenderfer and Apeagyei, 2011b; Apeagyei and Diefenderfer, 2013).

Field Performance

The primary method used to characterize the field performance of BSMs was the FWD. Analysis of data from the FWD was used to determine the structural layer coefficient (Marquis et al., 2003; Romanoschi et al., 2004; Sebaaly et al., 2004; Wen et al., 2004; Kroge et al., 2009; Diefenderfer and Apeagyei, 2011a, b) or layer stiffness by back-calculation (Forsberg et al., 2002; Marquis et al., 2003; Mohammad et al., 2003; Morian et al., 2004; Romanoschi et al., 2004; Wen et al., 2004; Lane and Kazmierowski, 2005; Loizos and Papavasiliou, 2006; Mallick et al., 2006; Miller et al., 2006; Chen and Jahren, 2007; Fu and Harvey, 2007; Jahren et al., 2007; Loizos, 2007; Loizos et al., 2007; Lee et al., 2009b; Gonzalez et al., 2009, 2011). As the results from a particular project are influenced by many parameters, a listing of the properties found is not presented here. However, layer coefficient values are typically reported as ranging from approximately 0.25 to 0.35 (with FDR and CIR/CCPR tending to be on the lower and upper end of this range, respectively) with layer stiffness values similar to those reported from laboratory testing.

There are relatively few published studies describing the long-term performance of pavements constructed with recycled materials, which is a drawback to the recycling process, as discussed by Stroup-Gardiner (2011). Those studies that are available predominantly point to the use of recycled materials on roadways carrying less than approximately 15,000 vehicles per day.

After summarizing more than 10 years of performance in Ontario, Kazmierowski et al. (1999) concluded that both hot in-place recycling and CIR were effective pavement rehabilitation options with deterioration rates that were similar to those for conventional pavement rehabilitation practices. Morian et al. (2004) reviewed projects located in Pennsylvania on roadways with annual average daily traffic volumes between 8,000 and 10,000 vehicles per day. They found that all reviewed CIR projects greatly exceeded their anticipated

10-year design life and provided two to three times the reflective crack resistance exhibited by conventionally resurfaced control sections. In addition, the cost of the CIR sections was shown to be between one-third and two-thirds less than that of conventionally resurfaced control sections.

Sebaaly et al. (2004) presented 10 years' worth of experience with CIR mixtures in Nevada. Based on their experiences, they reported that CIR was an effective rehabilitation process for low- and medium-volume facilities (defined as being between 30 and 300 equivalent single-axle loads [ESALs] per day). CIR was found to be effective in reducing the development of reflective and thermal cracking and rutting.

Chen and Jahren (2007) described a study of 24 projects in Iowa that were constructed between 1986 and 2004 with traffic volumes ranging from approximately 130 to more than 5,800 vehicles per day. They reported that the stiffness (as measured by the FWD) and air-void content were found to be the most influential parameters in the performance of CIR for the higher traffic volume sections. Conversely, the saturated ITS was found to be the most significant indicator of performance for lower traffic volume sections.

Preconstruction Testing

Project-Level Structural Testing, 2006 and 2011

For the current study, a detailed project-level survey to support project design was conducted in the right lane in 2006 and included FWD testing and core sampling (Weaver and Clark, 2007). Core samples showed that the asphalt layer thickness ranged from 10 to 12 in and included a surface layer AC mixture (having a 12.5 mm nominal maximum aggregate size [NMAS]) with a thickness between 1.75 and 2 in). The remaining thickness consisted of a base layer AC mixture (having a 25 mm NMAS). More than one-half of the cores showed layer debonding; either the surface layer was debonded from the underlying material or debonding was found within the base AC layer. FWD testing indicated that the effective structural number (SN_{eff}) ranged from approximately 4.5 to 6.9 and the subgrade resilient modulus values ranged from approximately 15,000 to 40,000 psi. Material collected from subgrade borings indicated the subgrade consisted of AASHTO classification A-7-6 to A-6 material (determined in accordance with AASHTO M 145-91, Standard Specification for Classification of Soils and Soil-Aggregate mixtures for Highway Construction Purposes [AASHTO, 2013]) having a water content of approximately 20% to 28%. Cracking, noted in certain areas within the wheel paths at the pavement surface, showed evidence of fines pumped from the subgrade, which is suggestive of structural failure.

A second round of FWD testing was conducted on March 3, 2011, approximately 3 months prior to construction. The testing was conducted in the right lane only with the load plate located within the right wheel path. At this time, the FWD testing indicated the effective structural number (SN_{eff}) ranged from approximately 3.3 to 4.5 with an average value of approximately 3.89. The subgrade resilient modulus values ranged from approximately 13,000 to 35,000 psi, with an average value of approximately 22,800 psi.

Pavement Layer Thickness by Ground Penetrating Radar

GPR testing was conducted using a 2 GHz horn antenna at approximately 3 months prior to construction. Testing was conducted at traffic speed with the antenna located approximately in the center of each lane. The results of the GPR testing were analyzed to determine the layer thicknesses for use with FWD analysis. For the right lane, the average depth of all asphalt and aggregate layers was approximately 10.8 and 6.6 in, respectively. For the left lane, the average depth of all asphalt and aggregate layers was approximately 10.3 and 6.3 in, respectively.

Automated Distress Survey Results, 2007-2011

Table 1 shows the results of the automated distress survey from 2007 through 2011 (the 2011 measurement was completed approximately 2 months prior to the start of the in-place recycling project) for the project area. The table shows the project average and 0.1-mile segment maximum or minimum values for the critical condition index (CCI), International Roughness Index (IRI), rut depth, patched area (expressed as a percentage of the wheel path), and alligator cracking (including all severity levels, expressed as a percentage of the entire pavement surface area) for the right lane.

The CCI parameter is a unitless composite indicator (having a range from 0 to 100) that quantifies all of the various distresses into a single value and is determined as the lesser of the load-related distress rating (LDR) and the non–load-related distress rating (NDR). The LDR incorporates load-related distresses such as wheel-path cracking, patching, rutting, etc.; the NDR includes non–load-related distresses such as transverse and longitudinal cracking (observed outside the wheel path), bleeding, etc. VDOT describes the pavement condition, in terms of CCI, as follows: excellent, ≥90; good, 70 to 89; fair, 60 to 69; poor, 50 to 59; and very poor, <50 (VDOT, 2012). Table 1 shows that although the average project condition was not poor during the time period shown, there were very deteriorated local areas as evidenced by a minimum 0.1-mile segment CCI of 50 and a maximum 0.1-mile segment IRI of 226 in per mile (VDOT [2012] stated that an interstate pavement is considered to have a poor ride quality when the IRI value is greater than 140 in per mile). At the time of data collection, patches were considered to have largely improved the pavement condition despite not addressing the actual cause of the deterioration.

Table 1 also shows the results of pavement rehabilitation efforts conducted at various times between 2007 and 2011. As an example, the data indicated that the average CCI increased from 86 in 2007 to 99 in 2008 (however, it can be seen that by 2010, the CCI value had fallen back to 86). Further, Table 1 shows the 0.1-mile segment minimum CCI improved in three instances (2007 to 2008, 2008 to 2009, and 2010 to 2011), possibly indicating that three instances of spot repairs were completed in the 5-year period.

Table 1. 2007-2011 Automated Distress Survey Results

Condition/Distress Measured	2007	2008	2009	2010	2011
Critical Condition Index, average/0.1-mile segment minimum	86/50	99/75	96/84	86/70	85/76
International Roughness Index (inches per mile), average/0.1-mile segment maximum	89/130	55/114	57/101	67/226	72/113
Rut Depth (inches), average/0.1-mile segment maximum	0.09/0.15	0.09/0.13	0.13/0.20	0.10/0.27	0.11/0.27
% Wheel-Path Patched Area, average/0.1-mile segment maximum	8.0%/45.4%	0.3%/12.2%	0.0%/0.0%	0.5%/11.5%	0.6%/11.3%
% Pavement Surface Having Alligator Cracking (all severity levels), average/0.1-mile segment maximum	0.0%/0.2%	0.0%/0.0%	0.0%/0.1%	1.0%/7.9%	1.6%/14.3%

Mix Design

Prior to construction, a mix design procedure was performed in the VCTIR laboratory to determine the optimum moisture content, the density at optimum moisture content, and the optimum recycling agent content for the CCPR and CIR materials (a mix design was also completed for the FDR materials by a third party). These processes began when the contractor collected milled materials from the project site in January 2011. The contractor selected foamed asphalt as the recycling agent for the CIR and CCPR materials, and a Wirtgen WLB10S laboratory-scale foamed asphalt plant was used to mix the materials in the laboratory. The CIR and CCPR mixtures were designed in accordance with the *Cold In-Place Recycling Manual* (Wirtgen, 2006) to determine the recycling agent content that would meet the minimum strength criteria for the project in terms of the ITS. Wirtgen (2006) stated that an ITS of 51 psi is equivalent to an AASHTO layer coefficient of 0.3 per in and is suitable for a high-volume highway (i.e., traffic greater than 5 million ESALs).

The laboratory-scale foamed asphalt plant was used to determine the foaming characteristics of the asphalt binder and also to produce foamed asphalt for the mix design process. A laboratory-scale pug mill was used to mix the foamed asphalt with milled materials obtained from test pits to determine the mix design parameters in accordance with AASHTO T 180-10, Standard Method of Test for Moisture-Density Relations of Soils Using a 4.4-kg (10-lb) Rammer and a 457-mm (18-in.) Drop, Method D (AASHTO, 2013), and ITS in accordance with AASHTO T 283, Standard Method of Test for Resistance of Compacted Hot-Mix Asphalt (HMA) to Moisture-Induced Damage (AASHTO, 2013).

Several foaming water contents were investigated to determine the optimum foaming characteristics of the performance grade (PG) 64-22 asphalt binder procured for the project. The results showed that to produce foamed asphalt with adequate foaming characteristics (defined by Wirtgen [2010] as having a minimum expansion ratio of 11 and a half-life of 8 sec), 2% water by weight of asphalt binder should be added at a pressure of 80 psi to the hot (320°F) PG 64-22 asphalt binder.

The foamed asphalt mix design was produced at three different binder contents: 2.00%, 2.25%, and 2.50% by weight of total mixture. For each mixture, 1% hydraulic cement (by weight of mixture) and 1% moisture (by weight of mixture) were added and mixed before the foamed asphalt was added. The main purpose of the cement was to increase the percentage of material passing the No. 200 sieve. This is critical for proper dispersion of the foamed asphalt in the mixture. However, Fu and Harvey (2007) and ARRA (2001) stated that adding cement also improves the moisture resistance of foamed asphalt mixtures.

The trial mixtures were prepared by compacting the stabilized mixtures in a 4-in-diameter mold using a gyratory compactor to a predetermined density of 125 lb/ft^3 after the materials were screened so that all particles passed the ¾-in sieve. This compacted density was based on the compaction level achieved when preparing specimens to 75 blows per face using a Marshall compactor. The compacted specimens, 4 in in diameter by approximately 2.5 in tall, were cured at 105°F for 72 hours before ITS testing. The results indicated that 2.0% foamed asphalt plus 1.0% hydraulic cement was adequate to achieve the required minimum ITS of 45 psi.

A combination of hydraulic cement and lime kiln dust was chosen as the stabilizing agent for the FDR portion of the project. Specimens were prepared from materials milled within the test pits to determine the optimum stabilizing agent content for the FDR process. The optimum was defined as the dosage rate that resulted in a mixture with an unconfined compressive strength of approximately 300 psi. This maximum unconfined compressive strength value was chosen with the intention of reducing cracking within the FDR layer. The dosage rate that achieved this desired value was 3.0%.

Construction Description

The project was constructed in two phases, one for the right lane and one for the left lane. The completed cross section for each lane was different since the level of deterioration differed in the right and left lanes, as shown in Figure 1. During the design phase, the original design for the right lane consisted of 8 in of paver-laid CCPR followed by a 4-in AC overlay consisting of a 2-in course of a 19 mm NMAS intermediate AC with a PG 70-22 asphalt binder followed by a 2-in stone-matrix asphalt (SMA) surface course having a 12.5 mm NMAS with a polymer-modified PG 76-22 asphalt binder. The project was bid according to this original design. However, concerns were raised over the local relative inexperience with the processes and the decision was made to modify the design just prior to construction. As a result, the first approximately 2,150 ft of the right lane was constructed according to the original design (12-in FDR, 8-in CCPR, 4-in AC) and the remainder of the right lane was constructed according to the modified design (12-in FDR, 6-in CCPR, 6-in AC).

The AC overlay for the 6-in AC section in the right lane was composed of 4 in of AC having a 19 mm NMAS with a PG 70-22 asphalt binder (placed in two lifts) and a 2-in SMA overlay having a 12.5 mm NMAS with a polymer-modified PG 76-22 asphalt binder. The AC overlay for the 4-in AC section in the right lane consisted of 2 in of AC having a 19 mm NMAS with a PG 70-22 asphalt binder and a 2-in SMA overlay having a 12.5 mm NMAS with a polymer-modified PG 76-22 asphalt binder.

The final cross section in the left lane was composed of 5 in of CIR over approximately 5 in of existing AC, surfaced by a 4-in AC overlay. The AC overlay for the left lane was composed of 2 in of AC having a 19 mm NMAS with a PG 70-22 asphalt binder and a 2-in SMA overlay having a 12.5 mm NMAS with a polymer-modified PG 76-22 asphalt binder. The left lane was profile-milled (at the choice of the contractor) prior to the final riding surface being paved. Table 2 shows the approximate construction schedule including lane closure dates for the recycled layers and the initial AC overlay. The 4-in AC overlay section in the right lane was completed during Closure 1; the 6-in AC overlay section in the right lane was completed during Closures 2 through 4. The final SMA surface course in both lanes was added at a later date after all other activities were completed.

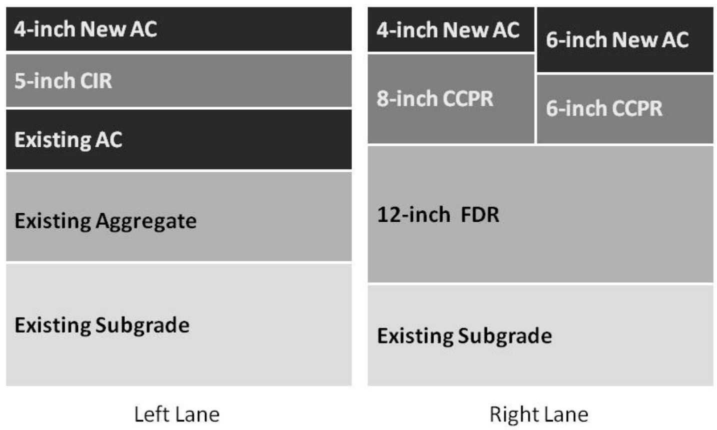

Figure 1. Completed Cross Section. AC = asphalt concrete; CIR = cold in-place recycling; CCPR = cold central-plant recycling; FDR = full-depth reclamation.

Table 2. Construction Schedule for FDR, CCPR, and CIR Courses and Initial Asphalt Concrete Overlay

Phase (Lane)	Closure No.	Dates	Distance Completed, ft	Cumulative Distance Completed, ft
Phase 1 (Right)	1	April 16-19	2,150	0-2,150
	2	April 30-May 2	6,500	2,150-8,650
	3	May 7-9	7,900	8,650-16,550
	4	May 21-24	2,775	16,550-19,325
Phase 2 (Left)	5	June 4-6	19,325	0-19,325

FDR = full-depth reclamation; CCPR = cold central-plant recycling; CIR = cold in-place recycling.

Right Lane

The work in the right lane began with milling on the first night of each lane closure and continued for approximately 3,000 ft (this distance varied on subsequent nights depending on production rates and weather forecasts). The milling process removed all but the bottom approximately 1 in of the existing AC to a depth of approximately 10 in. The following morning, the FDR process started (shown in Figure 2) and was followed later the same day by paving of the CCPR material. The CCPR process was employed in the right lane so the bound layers could be removed to give unimpeded access to the foundation materials for the FDR process. The milled asphalt material was stockpiled near the CCPR mobile plant (a Wirtgen KMA 220 mobile cold-recycling mixing plant) located at the northern end of the project. Using the FDR process allowed the contractor to address weak support issues manifested in the distresses observed at the pavement surface and identified in the FWD testing results. The entire operation for the right lane was completed in four multi-day closure periods, as shown in Table 2.

Figure 2. Full-Depth Reclamation Process Used in Right Lane

The FDR work was completed using a Wirtgen WR 2400 reclaimer. The lime kiln dust / cement combination was placed ahead of the reclaimer by a tanker truck with a distributor bar and was hydrated by a light water spray from a water truck. The hydration released a light fog and the Virginia State Police assisted to coordinate a temporary rolling closure until the process was completed. The rolling closures were approximately 10 minutes in duration. Compaction of the FDR material was accomplished by use of a 6-ton vibratory soil compactor with a padfoot drum as shown in Figure 3. The compacted FDR material was shaped by a motor grader and recompacted prior to placement of the CCPR material.

Figure 3. Compaction of Full-Depth Reclamation Layer

Following completion of the FDR, the CCPR material was produced at a mobile plant (see Figure 4) using foamed asphalt and hydraulic cement as the recycling agents. The material was produced, trucked back to the right lane, and paved using conventional paving practices (the CCPR material was dumped directly into the paver hopper as shown in Figure 5). No prime or tack coat was applied to the FDR prior to placing the CCPR. Approximately 4 to 6 hours passed between placement of the FDR and placement of the CCPR material at any one location. The contractor planned to cover the FDR material quickly to allow it to cure slowly. The decision on when to cover the FDR material was based on nuclear gauge density; the contractor also proof rolled with a loaded dump truck to ensure stability of the FDR layer. Compaction of the CCPR material was accomplished by the use of a 12-ton and a 14-ton double steel-drum vibratory roller and a 10-ton vibratory rubber-tire roller. A roller pattern was established based on nuclear density gauge measurements.

The 4-in overlay was applied to the CCPR after approximately 8 to 16 hours; no tack material was placed on the CCPR prior to overlay. The decision regarding when to pave the AC overlay was based on nuclear density gauge measurements showing a moisture content that was less than one-half the as-placed moisture content (which was approximately 75% of the optimum value determined during the mix design process). The moisture content measurements provided by the nuclear gauge were corrected by determining the moisture content in accordance with ASTM D4643-08, Standard Test Method for Determination of Water (Moisture) Content of Soil by Microwave Oven Heating (ASTM, 2013b). The pavement was opened to traffic following the initial two-course AC overlay, and then the 2-in SMA wearing course was added after approximately 8 weeks.

Figure 4. Cold Central-Plant Recycling Plant

Figure 5. Paving the Cold Central-Plant Recycling Material

Left Lane

The entire work in the left lane was completed within 3 workdays. During the first 2 days, an average of approximately 8,000 ft was completed each day. The final approximately 3,000 ft was intentionally held for the third day so that a scheduled tour of highway agency personnel could be accommodated.

The work began by milling approximately 2.75 in of the existing asphalt material. Then, a Wirtgen 3800 CR cold recycler (shown in Figure 6) completed the CIR process to a depth of approximately 4.25 in (accounting for fluff of the recycled materials, processing 4.25 in resulted in the desired 5-in-thick CIR layer) and incorporated hydraulic cement (placed ahead of the recycler via a distributer truck) and the foamed asphalt recycling agent in a single pass covering the full width of the lane. The cold recycler was equipped with a paving screed to place the CIR material. Compaction of the CIR layer was accomplished by the use of two 16-ton double steel-drum vibratory rollers and a 25-ton vibratory rubber-tire roller. A roller pattern was established based on nuclear density gauge measurements. A 2-in AC intermediate course was applied to the CIR after approximately 2 days, and the pavement was opened to traffic; no tack material was added to the CIR layer prior to the AC intermediate course. The 2-in SMA wearing course was added after approximately 6 weeks.

Figure 6. Cold In-Place Recycling Process Used in Left Lane

Traffic Management Plan

A unique traffic plan was developed for the in-place recycling project. During construction, the southbound direction of I-81 was reduced from two lanes to one lane for the entire approximate 4-mile length; these closures occurred 5 times over a 2-month period and were allowed as a continuous closure from 9 P.M. on Fridays to 7 A.M. the following Thursday. This unique closure window was determined based on traffic data that showed the highest traffic volumes on southbound I-81 measured in the vicinity of the project occurred Thursday and Friday afternoons. U.S. Route 11, a two-lane undivided primary route that normally carries approximately 3,300 vehicles per day, was used as a detour route for passenger vehicles traveling through the work zone. U.S. 11 runs parallel to I-81 and is located approximately 0.5 mile west of the north end of the construction project and intersects I-81 at the south end of the construction project.

Although a major public service announcement campaign warned travelers as far north as Pennsylvania upstream of the work zone, VDOT and the contractor intentionally provided limited warning of the right-lane detour to alleviate late-merge conflicts at the start of the work zone. During the right-lane closure, changeable message signs directed trucks to use the left lane and to continue on I-81. As vehicles approached the work zone, Group 2 channelizing devices divided the highway, forcing vehicles in the right lane onto the detour route for U.S. 11. An example of the passenger vehicle / truck split is shown in Figure 7. Generally, for lighter volumes, an upstream changeable message sign displayed the message *Trucks use left lane/right lane exits*, alerting drivers to the detour and allowing them the option to stay on I-81. When mainline volumes became heavy and queues developed, the message was changed to *Trucks use left lane/cars use right lane*. The left lane work was largely completed over one weekend and trucks were again directed to remain on I-81 while passenger vehicles were directed to exit and use the detour.

Figure 7. Result of Lane Closure and Instruction for Through Trucks to Remain on I-81

VDOT conservatively estimated that only 10% of through traffic would exit the freeway and use the detour. Field results showed that during construction, approximately 40% of traffic exited the freeway and used the detour (27% were through vehicles, and 13% was local traffic). Ninety percent of all trucks remained on I-81 through the work zone, and 10% exited the freeway (Gallo et al., 2012).

CIR/CCPR Construction Quality Control and Acceptance Testing

The project was subdivided into lots for testing purposes. Each lot consisted of 2,500 linear feet of full lane width. The acceptance criteria included depth of recycled layer, gradation, recycling agent dosage, ITS results, and compacted density. Only the results for gradation, ITS, and density are discussed in this report.

Gradation

Table 3 shows summarized results of the washed gradation tests conducted during construction of the project. These materials were collected by VCTIR and are not the gradation results used for acceptance. The mean percent passing each sieve size and the coefficient of variation (COV) are shown. Also shown are the specified gradation ranges based on the approved mix design for the project. The results are based on samples from the right (CCPR) and left (CIR) lanes. The results shown in Table 3 represent the average values of the gradation from samples collected in 18 lots (10 lots from the right lane and 8 lots from the left lane).

The individual gradation results showed that all but 2 lots passed the specified gradation controls for the right lane. However, none of the gradation controls passed for the left lane. One reason for the gradation failures in the left lane could be that samples were taken after stabilization and thus the fines were likely bound up with the coarser particles, which negatively influenced the results. A finding from this project was that all gradation sampling should occur prior to stabilization. One means to accommodate this process might include gradation sampling at the start of each day’s construction with additional checks near mid-day when the recycling process needs to stop so that the recycling agent tankers can be switched.

Table 3. Gradation Results

Sieve Size	Percent Passing					
	Right Lane, CCPR		Left Lane, CIR		Approved Mix Design Tolerance	
	Mean	COV	Mean	COV	Lower	Upper
1.5 in	100	0.0	100	0.0	100	100
1.0 in	99.1	1.0	95.9	4.0	94.6	100
¾ in	97.3	1.2	86.8	6.7	92.5	100
No. 8	32.8	11.3	9.9	37.8	25.0	44.0
No. 200	3.5	32.3	0.3	177.0	3.0	9.0

CCPR = cold central-plant recycling; CIR = cold in-place recycling; COV = coefficient of variation.

Indirect Tensile Strength

The main purpose of the ITS tests was to evaluate the construction quality and moisture susceptibility of the CCPR and CIR materials. All specimens were prepared using field-mixed, laboratory-compacted materials. The tests were performed on 4-in-diameter specimens having a height of approximately 2.5 in that were manufactured in a field laboratory using a Marshall hammer with 75 blows per face.

ITS testing was a major component of the quality control and QA procedures undertaken during the project. A total of 54 laboratory-compacted specimens were made from foamed asphalt materials sampled during construction. To manufacture the samples for quality control testing, the materials were screened over a ¾-in sieve to remove larger particle sizes and were compacted using a Marshall hammer. This process was followed during QA testing since it was performed during the mix design. The mass of material required to manufacture the specimens was determined by preparing multiple specimens and recording the weight that resulted in a specimen of the proper height. The remaining specimens for that lot were compacted using the same mass of material.

Of all specimens fabricated, one-half were designated for dry ITS testing and the other half for conditioning in a water bath and subsequent ITS testing after saturation. A modified version of AASHTO T 283, Standard Method of Test for Resistance of Compacted Hot-Mix Asphalt (HMA) to Moisture-Induced Damage (AASHTO, 2013), was used such that conditioned samples were not subjected to freeze-thaw cycling. For this study, specimens were held in an oven at 105°F for 72 hours and then conditioned specimens were held in a water bath at 77°F for an additional 24 hours. A single testing temperature of 77°F was used for all ITS tests. A loading rate of 2 in per minute was used. For dry specimens, a minimum ITS value was specified as 95% of the approved mix design value. The minimum ITS from the approved mix design value was 51 psi, making the ITS requirement for the average from each lot 48.5 psi. The tensile strength ratio (TSR), a measure of moisture susceptibility of asphalt mixtures, was computed as the ratio of the ITS test result after saturation to the dry ITS test result. The TSR value was reported but was not part of the acceptance parameters.

The average ITS values for the right and left lanes were 54.9 psi and 51.2 psi, respectively, with standard deviations of 9.3 psi and 6.5 psi, respectively. The individual results showed that 16 of the 19 lots in the right lane and 12 of the 16 lots in the left lane had passing ITS results (greater than or equal to 48.5 psi). The average TSR values for the right and left lanes were 73.4% and 66.4%, respectively, with standard deviations of 6.0% and 8.5%, respectively. The individual results also showed that 12 lots in the right lane and 4 lots in the left lane had a TSR value greater than 70%. For those sections that did not pass the ITS requirements, cores were collected to confirm the properties of the material produced in the field (see section on "Laboratory Evaluation" in the "Methods" section).

Field Density

The density of the constructed layer was measured by the nuclear density gauge. The contractor started the project using the direct transmission method but was allowed to switch to the backscatter mode. However, it is known that the backscatter mode may not report the density from the bottom portions of deeper layers. This should be taken into consideration if backscatter is allowed for density measurements on future projects having a sufficiently thick recycled layer.

During the laboratory mix design process, a maximum density for the CIR/CCPR material was determined to be 125.0 lb/ft^3. This value was based on compacting a 63-mm-thick specimen using the Marshall hammer at 75 blows per face. In addition, AASHTO T 180-10, Standard Method of Test for Moisture-Density Relations of Soils Using a 4.4-kg (10-lb) Rammer and a 457-mm (18-in.) Drop, Method D (AASHTO, 2013), was used to determine a modified Proctor value of 127.8 lb/ft^3. Field densities averaged 127.4 lb/ft^3 with a range of 124.7 to 130.0 lb/ft^3. During construction, a decision was made to refer to the modified Proctor results for acceptance; the contractor was required to achieve a density value of 98% of the modified Proctor value (125.2 lb/ft^3). Cores were collected to confirm the density of the material produced in the field (see section on "Laboratory Evaluation" in the "Methods" section).

Additional Laboratory Testing

A program of additional laboratory testing was performed using specimens that were (1) fabricated in the laboratory from loose materials collected during construction and (2) obtained from cores collected after construction. Cores were collected on the dates and at the approximate locations listed in Table 4. Not all of the cores were used for testing by VCTIR. The cores collected in 2011 were generally used for determination of gradation and binder content, density, ITS, resilient modulus, and FN. The cores collected in 2013 were generally used for dynamic modulus and FN testing.

Table 4. Coring Dates and Locations

Date	VCTIR Lab ID	Lane	Approximate Location (Milepost)	No. of Cores
June 22, 2011	11-1009	Right	217.1-216.4	6
			216.3	11
August 4, 2011	11-1025	Right	214.2	6
			214.0	6
August 16, 2011	11-1026	Left	217.4	9
			216.5	7
August 31, 2011	11-1037	Right	215.0	16
April 1, 2013	13-1004	Right	216.3-216.2	7
	13-1005	Left	216.3-215.0	6
			Total	74

VCTIR = Virginia Center for Transportation Innovation and Research.

Gradation and Binder Content Tests

Figure 8 shows the gradation results for the CCPR and CIR field cores from testing conducted in accordance with AASHTO T 27-11, Standard Method of Test for Sieve Analysis of Fine and Coarse Aggregates (AASHTO, 2013). The gradations shown in Figure 8 are the averages of six and four cores from the CCPR (right lane) and CIR (left lane) materials, respectively. From Figure 8 it is seen that the average gradations are similar but the CCPR materials are slightly finer than the CIR materials. Figure 8 also shows the variability of the gradation in terms of the COV. The COV of the cores from the CCPR materials is less than 5% and is generally seen to increase as the sieve size becomes finer. The COV of the cores from the CIR materials approaches 16% and also increases as the sieve size becomes finer. The results from one of the four cores collected from the CIR were more variable than those from the other three. If the data from this core were removed, the COV for the CIR materials would range from 0% to 8%, although the same trend of increasing variability with decreasing particle size would be found. These results show that the gradation of the materials obtained from the CCPR materials had relatively less variability than the gradation of the materials obtained from the CIR materials. During construction, the only additional processing of the CCPR materials as compared to the CIR materials was that the CCPR materials were passed through a 1.5-in screening deck before being stabilized within the mobile plant. The results of the aggregate gradation tests were also evaluated by comparing them with the recommendations from Wirtgen (2010) as shown in Figure 9. The gradation of the cores appears to be on the finer side of the recommended values, especially on the coarser sieve sizes. Gradation acceptance sieves included the 1.5-in and 3/8-in sieves.

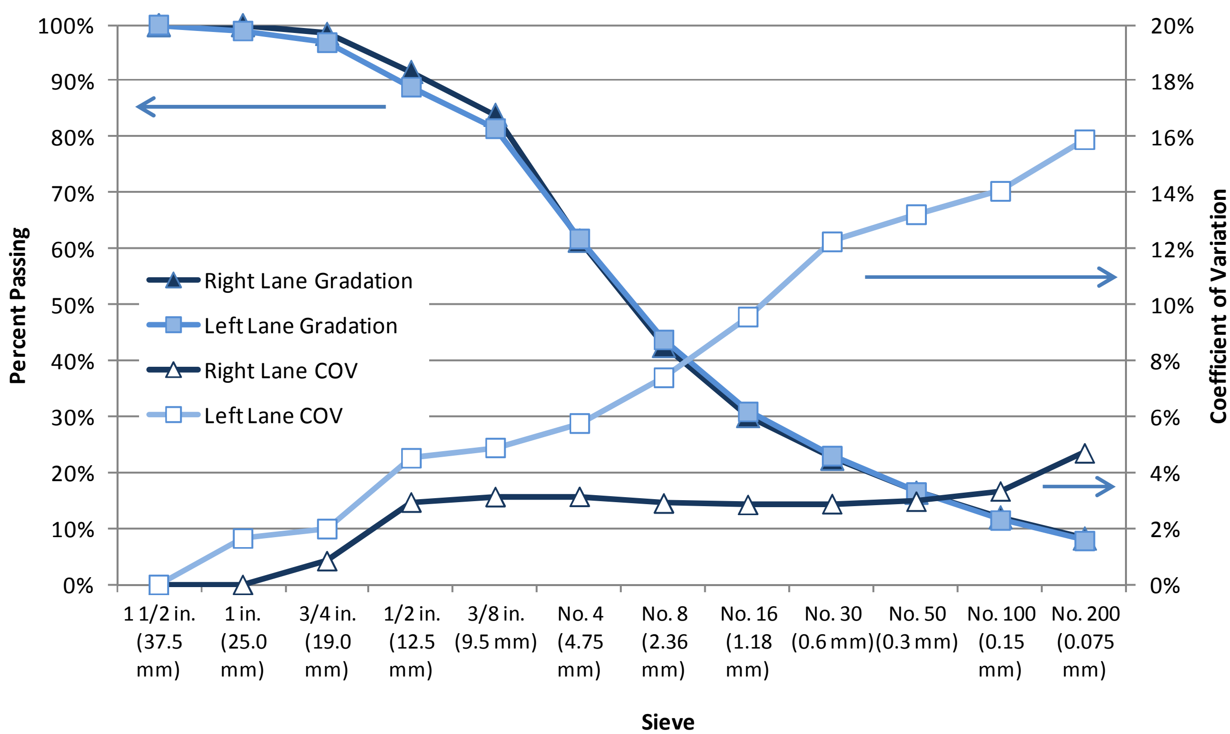

Figure 8. Gradation Results From Field Cores. COV = coefficient of variation.

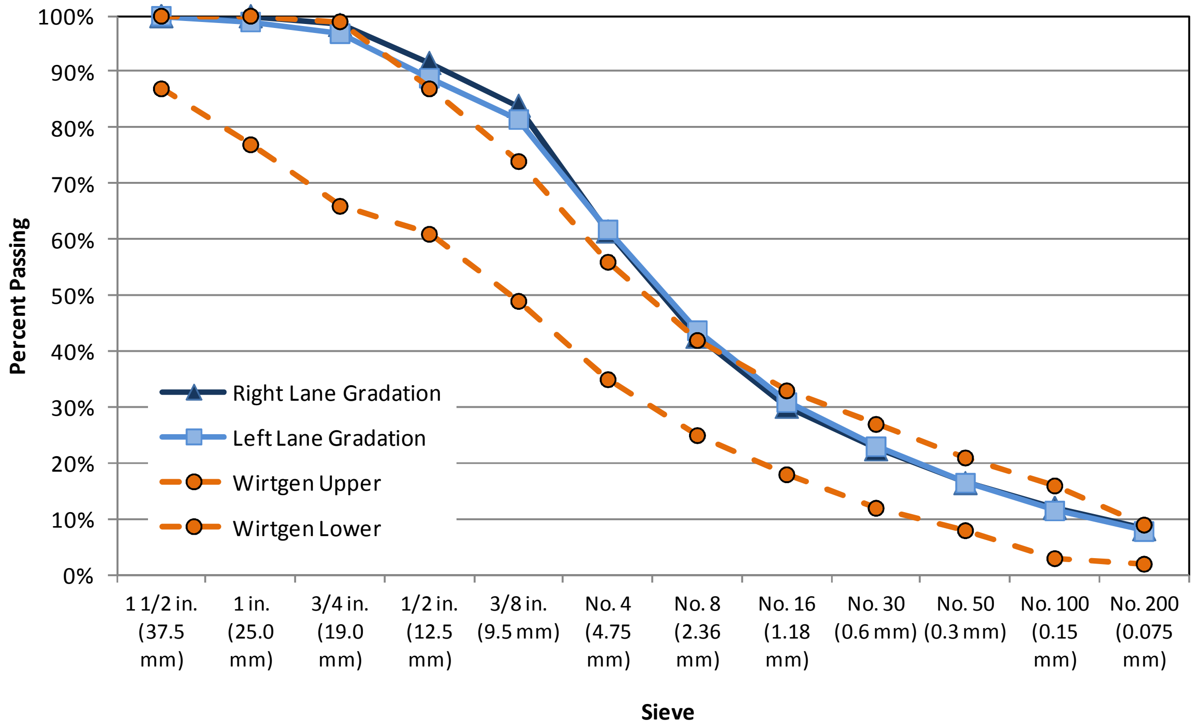

Figure 9. Gradation Results From Field Cores Compared With Recommendations From Wirtgen (2010)

The average binder content of the CCPR materials was 7.58%, with a COV of 3.01%. The average binder content of the CIR materials was 8.00%, with a COV of 5.52%. The slightly higher binder content for the CIR materials could be attributed to the fact that the CIR process was limited to the upper portions of the pavement and therefore included a higher proportion of original AC surface mixtures that usually contain higher binder contents. The measured binder content included both the original binder from the parent in-place materials and the foamed asphalt added during the recycling process. Since the foamed binder content for both the CCPR and CIR materials was 2%, the results obtained were considered reasonable since an average binder content of 5% to 5.5% is typical for AC produced in Virginia.

Bulk Density

The cores were cut into two specimens (approximately 2 in thick each) and labeled as "top" and "bottom"; the density of each was measured in the laboratory as previously discussed. A summary of the results is shown in Table 5, with additional details included in Table A1, Appendix A. A *t*-test was performed to determine whether differences in mean bulk density values between the CCPR and CIR process were statistically significant. The differences in mean bulk density for all the CCPR density results (averaging top and bottom) versus all the CIR density results (averaging top and bottom) were not significant ($p = 0.24$). *T*-tests were performed to determine whether differences in the mean bulk density between the "top" specimens and the "bottom" specimens were statistically significant. The differences in bulk density between the top and bottom of the CCPR and CIR specimens were statistically significant ($p < 0.01$ for both cases). A *t*-test was also used to evaluate differences in bulk density between the top portions of the CCPR and CIR cores and also the bottom portions of the

CCPR and CIR cores. The differences in bulk density from the top portions of the cores from the two processes were not statistically significant ($p = 0.09$). Further, the differences in bulk density from the bottom portions of the cores from the two processes were not statistically significant ($p = 0.11$). The results did show that the achievable bulk density at the bottom of a recycled layer should be considered in the placement and compaction of relatively thick layers.

In addition, the average bulk density values shown in Table 5 are greater than the maximum density based on the Marshall hammer found during the mix design, i.e., 125 lb/ft^3. Table A1, Appendix A, shows that of the 25 bulk density test results, only 1 was less than the maximum density. Future research should document the range of field densities achieved so the mix design process can more accurately represent actual construction.

Table 5. Summary of Density Testing Results

Location	Top of Core		Bottom of Core	
	Bulk Density Average, lb/ft^3	**COV**	**Bulk Density Average, lb/ft^3**	**COV**
CCPR (right lane)	139.2	1.0%	128.6	1.4%
CIR (left lane)	141.8	2.1%	132.4	4.3%

COV = coefficient of variation; CCPR = cold central-plant recycling; CIR = cold in-place recycling.

ITS Tests

A summary of the ITS test results are presented in Table 6; additional details are shown in Table A2, Appendix A. Similar to density, differences in ITS results were found when specimens obtained from the top of the cores were compared with specimens obtained from the bottom of the cores. The data showed that specimens obtained from the top of the cores generally were stronger than specimens obtained from the bottom of the cores for both CIR and CCPR. These results agreed with those reported by Loizos (2007), who also found that the upper portions of field cores had higher ITS values than the middle or lower portions. Loizos (2007) attributed the difference to compaction achieved during construction.

A *t*-test was performed to determine whether differences in mean ITS values between the CCPR and CIR processes were statistically significant. There was no significant difference between the mean ITS value for all the CCPR results (averaging top and bottom) and all the CIR results (averaging top and bottom) ($p = 0.40$). Additional *t*-tests were performed to determine whether differences in the mean ITS between the top specimens and the bottom specimens were statistically significant. The differences in density between the tops and bottoms of the CCPR specimens were statistically significant ($p = 0.01$) but were not statistically significant for the CIR specimens ($p = 0.10$). A *t*-test was also used to evaluate differences in ITS between the top portions of the CCPR and CIR cores and the bottom portions of the CCPR and CIR cores. The differences in ITS from the top portions of the cores from the two processes were not statistically significant ($p = 0.60$). The differences in ITS between the bottom portions of the cores from the two processes were statistically significant ($p = 0.01$). It is not surprising that the ITS was different for the two processes since the layer thicknesses were different. The layer thickness for the CIR process was thinner than for the CCPR process, and thus the CIR core could be expected to have a higher ITS value at the bottom of the layer. The results indicated that the ITS value can vary greatly between the top and bottom of a recycled layer when relatively thick layers are placed and compacted. The results also suggest that the ITS-derived performance does not distinguish between the two techniques used in the recycling process when the top of the layer is considered; the performance of CCPR-produced materials and CIR should be similar when ITS is used as the determining factor.

Moisture susceptibility was assessed by assigning additional specimens randomly as either moisture-conditioned (wet) or un-conditioned (dry) ITS specimens. The specimens assigned to the wet group were soaked in a water bath at 25°C for 24 hours before testing to simulate the effect of moisture on the recycled materials. The TSR was computed as the ratio of the average ITS wet to ITS dry results. Table 7 provides a summary of the moisture susceptibility results; Table A3, Appendix A, includes additional details. Table 7 also shows the TSR calculated for each process.

Table 6. Summary of Indirect Tensile Strength Testing Results (psi)

	Top of Core		Bottom of Core	
Location	**Average**	**COV**	**Average**	**COV**
CCPR (right lane)	80.8	22.6%	48.7	20.4%
CIR (left lane)	75.6	14.2%	65.0	17.4%

COV = coefficient of variation; CCPR = cold central-plant recycling; CIR = cold in-place recycling.

Table 7. Summary of Indirect Tensile Strength Testing Results (psi) for Moisture Susceptibility

	Dry Specimens		Wet Specimens		
Location	**Average**	**COV**	**Average**	**COV**	**TSR**
CCPR (right lane)	77.2	13.9%	54.2	28.6%	70.2%
CIR (left lane)	71.6	10.9%	56.6	17.4%	79.1%

COV = coefficient of variation; TSR = tensile strength ratio; CCPR = cold central-plant recycling; CIR = cold in-place recycling.

Resilient Modulus (M_R) Tests

Table 8 shows the M_R test results for the CCPR and CIR processes; additional details are provided in Tables A4 and A5, Appendix A, for the CCPR and CIR process, respectively. Table 8 shows that the M_R can vary widely within the same test temperature. The researchers believe that the variability is attributable to a combination of the variability within the test and the inherent variability of the recycling process itself. Similar variability was found for FDR specimens by Diefenderfer and Apeagyei (2011b).

With regard to the average M_R values in Table 8, both the CIR and CCPR exhibited a temperature-dependent behavior; i.e., the materials become less stiff at higher temperatures, similar to AC. This indicates that the interparticle bonding is controlled in part by the asphalt binder and is not entirely based on the aggregate skeleton, as would be expected from an untreated aggregate material. It also indicates that viscoelastic analysis may be appropriate for CIR and CCPR materials. Similar results suggesting temperature sensitivity of asphalt-stabilized mixtures were found by Fu and Harvey (2007) when conducting cyclic triaxial loading tests on foamed asphalt mixtures.

A *t*-test was performed to determine if differences in the M_R between the top and bottom specimens from the CCPR and CIR were statistically significant with respect to temperature. For the CCPR cores, the differences were significant for the 4°C and 20°C test temperatures but not for the 38°C test temperature. The *p*-values were 0.02, 0.04, and 0.69 for the test temperatures of 4°C, 20°C, and 38°C, respectively. Differences for the CIR cores were not significant for the test temperatures considered. The *p*-values were 0.44, 0.68, and 0.51 for the test temperatures of 4°C, 20°C, and 38°C, respectively. A *t*-test was also performed to study the differences in the M_R values across all CCPR and CIR cores to compare the two processes. The differences in the average M_R of all CCPR cores versus all CIR cores were not statistically significant with respect to any of the test temperatures. The *p*-values were 0.51, 0.15, and 0.45 for the test temperatures of 4°C, 20°C, and 38°C, respectively. The results suggested that the average M_R did not depend on which recycling technique was used and the CCPR and CIR materials for this project could be expected to perform similarly when M_R is the determining factor.

Table 8. Summary of Resilient Modulus Testing (psi)

Temperature, °C	Location	CCPR (right lane)		CIR (left lane)	
		Average Resilient Modulus	COV	Average Resilient Modulus	COV
4	Top	1,331,084	41.0%	1,169,512	19.2%
	Bottom	744,644	20.6%	1,066,353	23.4%
	All	1,015,309	47.1%	1,110,564	21.2%
20	Top	574,035	27.5%	554,745	19.3%
	Bottom	411,555	21.4%	594,727	35.1%
	All	486,546	30.1%	577,597	29.1%
38	Top	235,567	17.2%	250,021	23.8%
	Bottom	253,567	35.4%	323,597	79.5%
	All	246,105	29.0%	292,064	67.1%

CCPR = cold central-plant recycling; CIR = cold in-place recycling; COV = coefficient of variation.

Flow Number Tests

The flow number (FN) was determined from laboratory-compacted specimens fabricated from field-produced materials and from specimens obtained by coring following construction. The results reported herein are the mean FN of at least three replicates.

Specimens Compacted in the Laboratory From Field-Produced Materials. The mean FN and COV at two combinations of confining and deviator stress are shown in Table 9 for the CCPR and CIR materials; additional details are provided in Tables A6 and A7, Appendix A, for the CCPR and CIR process, respectively. The results were highly variable, as indicated by COV values that ranged up to approximately 46%. It is unclear if the cause for this high variability was related to the sampling technique, the compaction process, or the test itself. The level of variability found in this study was higher than that reported for conventional AC mixtures by Apeagyei et al. (2011) and Apeagyei and Diefenderfer (2011) but within that reported by Bonaquist (2010).

Table 9 also indicates that some of the FN specimens produced in the laboratory were sieved, as discussed previously. This was performed to study the influence of larger particles on the test results. As the test specimens were fabricated, materials retained on the ¾-in sieve were removed prior to compacting the test specimens. This procedure was performed to make specimens that were similar to those produced during the mix design phase in accordance with AASHTO T 180-10, Standard Method of Test for Moisture-Density Relations of Soils Using a

4.4-kg (10-lb) Rammer and a 457-mm (18-in.) Drop, Method D (AASHTO, 2013). This practice could potentially affect the FN comparison between field-produced laboratory-compacted specimens and those obtained by coring; however, there were no studies on this particular topic in the literature. It is unclear if removal of the larger particles contributed to a high sample-to-sample variability or if the materials are inherently variable regardless. Additional studies may be needed to investigate this further.

The confined FN test results were higher than the unconfined FN test results for the CCPR materials. A t-test showed the difference was statistically significant when all CCPR unconfined and confined test results were averaged together ($p = 0.03$). However, when each date was considered separately, the difference between the unconfined and confined FN results was significant (p-value less than 0.05) for all dates except for those materials produced on 5/21/2011. The p-values were 0.01, 0.02, and 0.07 for materials produced on 5/2/2011, 5/11/2011, and 5/21/2011, respectively (note the p-value of 0.07 is not highly significant). The difference between the unconfined and confined FN for the CIR material was not statistically significant, with a p-value of 0.001. It is not clear why the test results from the materials produced on 5/11/2011 were so dramatically greater than those produced on the other dates. Air-void content measurements were not completed on these test specimens. The results suggested that application of realistic test conditions based on actual stress conditions to which the material would be subjected in the field is of critical importance when evaluating rutting susceptibility of recycled materials. Although the strain at FN is not shown here, it was generally found that the strain at FN was higher for the confined tests than for the unconfined tests.

Table 9. Results of Flow Number Test for Field-Produced Laboratory-Compacted Specimens

VCTIR Specimen ID (Date)	Sieved?	Confining/ Deviator Stress, psi	Flow Number Average	Coefficient of Variation
CCPR				
5/2/2011	No	0/30	49	33.5%
		10/70	824	15.6%
5/11/2011	Yes	0/30	1416	32.9%
		10/70	10000[a]	0.0%
5/21/2011	Yes	0/30	66	28.3%
		10/70	1915	46.3%
CIR				
6/4/2011	Yes	0/30	39	29.6%
		10/70	772	7.4%

VCTIR = Virginia Center for Transportation Innovation and Research; CCPR = cold central-plant recycling; CIR = cold in-place recycling.
[a] Tests were stopped at 10,000 cycles.

Specimens Obtained From Field Cores. Cores were collected approximately 3 months after construction of the recycled and the intermediate AC layers and again after approximately 20 months. The mean FN and COV at two combinations of confining and deviator stress are shown in Table 10 for the CCPR and CIR materials; additional details are provided in Tables A8 and A9, Appendix A, for the CCPR and CIR process, respectively. The results were also highly variable, as indicated by the high COV for certain cases. The effect of confinement was statistically significant for the materials denoted as Lab ID 11-1009 (the VCTIR Specimen ID is given as xx-yyyy where xx signifies the last two digits of the calendar year in which the mixture was sampled and yyyy signifies the consecutive number of the mixture as recorded in the laboratory starting with 1,000); the p-value was 0.004. The testing conducted on cores collected after 20 months was performed at the higher deviator stress, but no confinement was used. The difference between the FN of CCPR and CIR cores was considered by performing a t-test for those materials denoted as Lab ID 13-1004 and those denoted as Lab ID 13-1005. The p-value was 0.34, indicating the difference was not statistically significant.

As seen in Table 10, tertiary flow was achieved very quickly when a high deviator stress was used without confinement. This high pressure level without confinement may not accurately represent the in-service conditions expected for recycled materials and suggests the importance of selecting stress conditions that accurately simulate field loading conditions. The FN data for specimens obtained from cores were in good agreement, qualitatively, with data obtained in previous studies (Kim et al., 2009; Apeagyei et al., 2011).

Table 10. Results of Flow Number Test for Specimens From Field Cores

VCTIR Specimen ID	Confining/Deviator Stress, psi	Flow Number Average	Coefficient of Variation
CCPR			
11-1009	0/30	2761	48.8%
	10/70	9158	9.3%
13-1004	0/70	309	140.4%
CIR			
13-1005	0/70	62	21.6%

VCTIR = Virginia Center for Transportation Innovation and Research; CCPR = cold central-plant recycling; CIR = cold in-place recycling.

Dynamic Modulus (|E|)*

The dynamic modulus was assessed on specimens fabricated from cores collected approximately 20 months after construction; the cores were collected from between the wheel paths in each lane. Results of GPR testing were used to sample from those locations in the right lane (CCPR process) that were most likely to have the thickest recycled materials since the test protocol for the AMPT requires a 6-in-tall specimen. However, the cores retrieved from the right lane were marginally tall enough for testing in accordance with AASHTO TP 79-09, Standard Method of Test for Determining the Dynamic Modulus and Flow Number for Hot Mix Asphalt (HMA) Using the Asphalt Mixture Performance Tester (AMPT) (AASHTO, 2013). With an effort to conduct similar testing on the right lane (CCPR) and left lane (CIR) materials (where the CIR layer was placed less than 6 in), the researchers decided to conduct the dynamic modulus tests using the indirect tensile geometry as described by Kim et al. (2004).

A total of 13 cores were collected, 7 from the right lane and 6 from the left lane. The average height of the recycled layer from the cores from the right and left lanes was 5.0 and 4.2 in, respectively. All cores were retrieved intact, and there were no signs of deterioration from trafficking. Testing was conducted at temperatures of 4.4, 21.1, 37.8, and 54.4°C at frequencies of 25, 10, 5, 1, 0.5, and 0.1 Hz. Testing was performed on four CCPR specimens and three CIR specimens. The remaining specimens were held for dynamic modulus testing using the indirect tensile geometry, but testing was not completed during this study.

Figure 10 shows the dynamic modulus master curves of the CCPR and CIR materials using the indirect tensile geometry in the AMPT. The master curves are computed from the average dynamic modulus values for each material type. From Figure 10 it can be seen that the dynamic modulus for the CCPR material was generally stiffer at low and intermediate reduced frequencies whereas the CIR was generally stiffer at high reduced frequencies. This would suggest that the CCPR material might have better performance at higher temperatures. Tables A10 and A11, Appendix A, show the dynamic modulus values for the individual test specimens for the CCPR and CIR materials, respectively. From these tables it can be seen that the between-sample COV ranged from approximately 18% to 45%. The COV was generally found to increase with increasing temperature but not always with decreasing test frequency. The COV range was similar for the CCPR and CIR materials. From the limited number of specimens, it is not clear if the observed variability was typical for these materials or a function of the small sample size. It is recommended that additional cores be collected and tested for dynamic modulus to characterize the performance of these materials better.

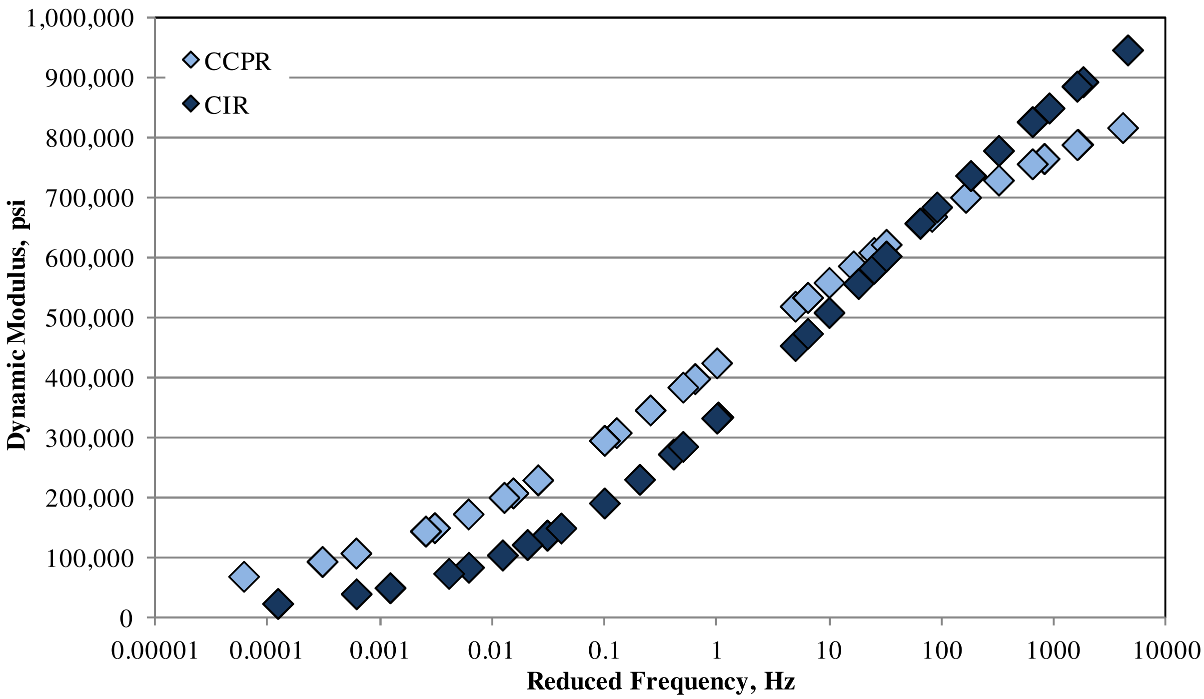

Figure 10. Dynamic Modulus Master Curve for Cold Central-Plant Recycling (CCPR) and Cold In-Place Recycling (CIR) Specimens Produced From Field Cores

Field Evaluation

Rut Depth and Ride Quality

Manual Rut Depth Measurements

Prior to construction, the primary concern regarding the pavement recycling process was the potential for further densification under traffic to the point where rutting at the surface could become a safety concern. This concern was brought about by the relative lack of experience in

Virginia using these materials on roadways with high truck volumes. Chen et al. (2002, 2006) highlighted the potential for these issues in Texas, especially when similar materials were exposed to moisture. To investigate the potential for early rutting, periodic manual rut measurements were conducted in the right wheel path of the right lane over the first approximately 3,200 ft of the project. These measurements were taken following the application of the first of two AC overlay courses but before the second overlay was applied. The rutting measurements were conducted using a 6-ft rut beam as shown in Figure 11. The rut beam used had the capability to measure rut depths to the nearest 0.01 in.

Measurements were made over the first 3 months in the right lane following construction; testing was conducted as lane closures were allowable. The first approximately 2,150 ft of the right lane consisted of the work performed during the first lane closure (see Table 2) using the original thickness design (denoted as Section 1). The remainder of the manual rut testing consisted of work performed during the second lane closure (see Table 2) using the modified thickness design (denoted as Section 2). In both sections, the final 2-in SMA overlay had not yet been placed, and thus the as-tested structure consisted of a 2-in AC layer over 8 in of CCPR in Section 1 and a 4-in AC layer over 6 in of CCPR in Section 2. Both sections were underlain by the 12-in FDR layer.

Figure 12 shows the average measured rut depths on the first two completed sections of the right lane (approximately 3,200 ft) for approximately 2.5 to 3 months after construction. The error bars shown in Figure 12 represent ±1 standard deviation. The average rut depth for Section 1 ranged from 0.12 to 0.14 in, with a standard deviation that ranged from 0.06 to 0.07 in. The average rut depth for Section 2 ranged from 0.06 to 0.07 in, with a standard deviation that ranged from 0.02 to 0.04 in. These values were not considered to be practically significant so long as they did not increase with time. It was estimated that these sections carried more than 350,000 trucks from the time they were constructed to the time the final round of manual rut testing was completed.

Figure 11. Six-Foot Rut Beam and Measurement Wheel Used During Manual Rut Depth Measurements

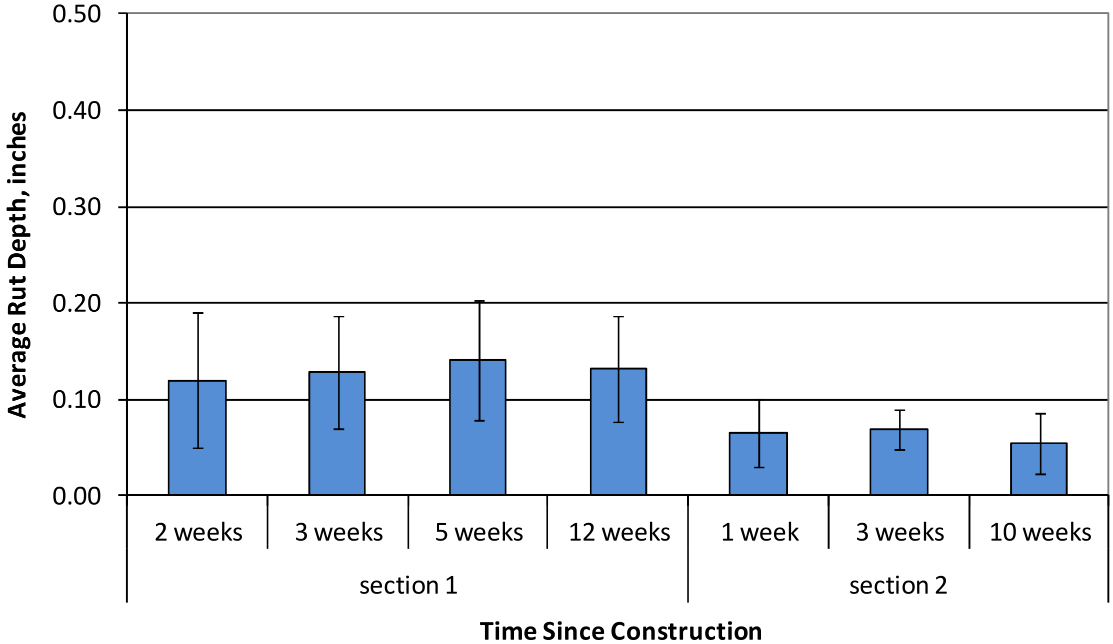

Figure 12. Average Manual Rut Measurement for Right Lane (Showing Standard Deviation As Error Bars) Over First 10 to 12 Weeks After Construction

A *t*-test was performed to determine if differences in the mean values were statistically significant for different test dates for each section or for different sections for each test date. For Section 1, the differences were not statistically significant; *p*-values ranged from 0.36 to 0.69. Further, the differences in average rut depths on the different test dates for Section 2 were not statistically significant; *p*-values ranged from 0.32 to 0.64. However, when the average rut depths for the same test date for Sections 1 and 2 were compared, the differences were statistically significant; *p*-values were less than 0.002. Thus, the measured rut depths did not change with time in either section (over the first 10 to 12 weeks). However, there was a statistically significant difference in measured rut depth between the original and modified thickness design. However, given the magnitude of the rut depths, the difference was not considered practically significant.

Rut Depth and Ride Quality Measurements Using a Traffic-Speed Inertial Profiler

Additional rut measurements were conducted following the second AC overlay course using a traffic-speed inertial profiler. A third-party vendor was contracted to acquire periodic rut depth and ride quality measurements of the right and left lanes after construction was completed. Testing was conducted at approximately 5, 9, 12, 16, 23, 28, and 34 months after construction. The data shown in Figures 13 through 16 were collected by a device having three laser sensors. Ideally, the three sensors travel along the same line within the two wheel paths and the center of the lane during subsequent tests. However, it is known that this is not always the case. The reader should know that some unquantifiable error exists in the data shown in Figures 13 through 16. Rather than the specific values, the trends in each figure are the most important results.

Figure 13 shows the ride quality in terms of the average IRI for both lanes. Figure 13 also includes a horizontal dashed line representing the before construction average IRI of 72 that was measured in the right lane in 2011 prior to the start of construction. Figure 14 shows the average rut depth results. Figures B1 through B4, Appendix B, show details of the ride quality and rut depth measurements averaged every 0.1 mile along the project.

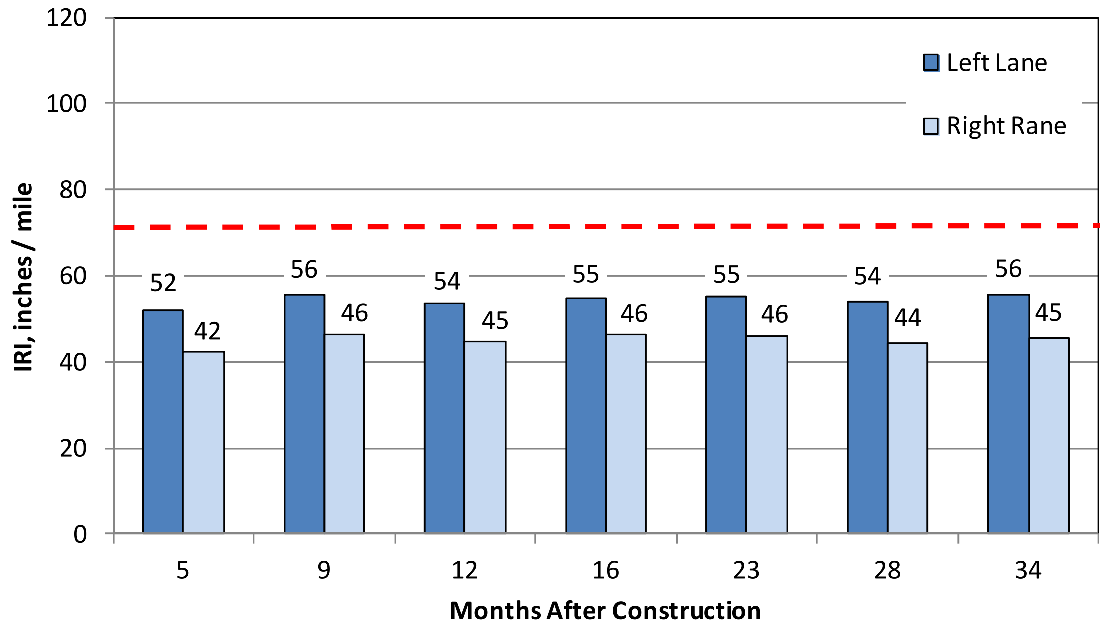

Figure 13. Average Ride Quality for Right and Left Lanes. IRI = International Roughness Index. The horizontal dashed line represents the before construction average IRI of 72 that was measured in the right lane in 2011 prior to the start of construction.

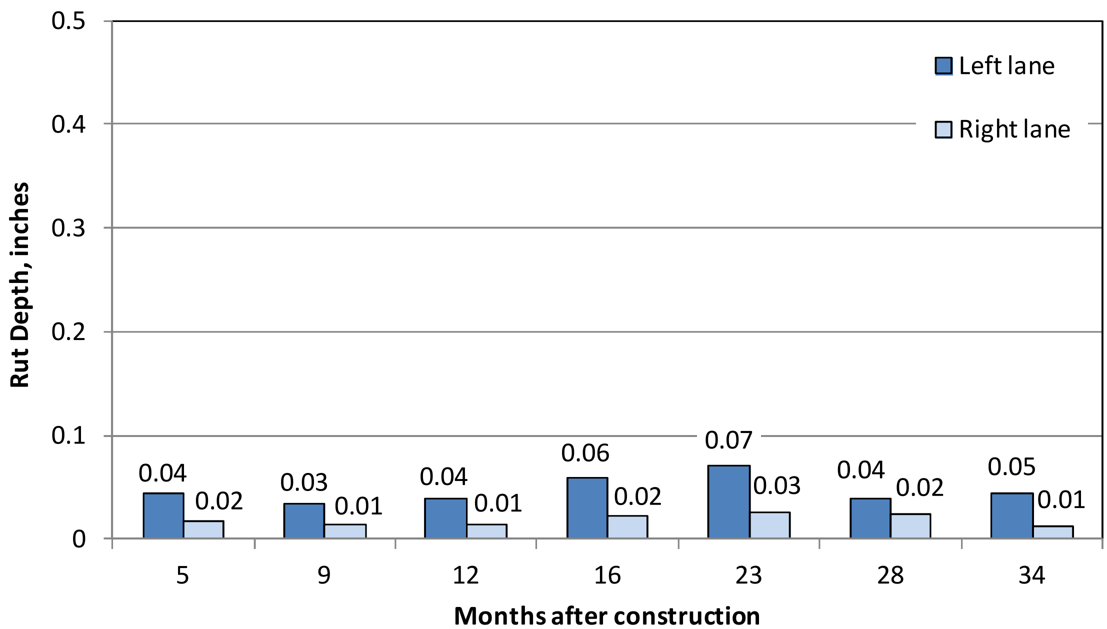

Figure 14. Average Rut Depth for Right and Left Lanes

Figure 13 shows that the left lane had a higher IRI than the right lane. Despite these differences, VDOT would still classify the pavement ride quality for both lanes as excellent (VDOT, 2012). VDOT's standard specification for ride quality (VDOT, 2009) was applied to this project, and the contractor achieved a pay incentive for smoothness. Figure 14 shows that the left lane also had a slightly greater rut depth than the right lane; however, the rut depths measured could still be considered negligible from a practical perspective.

Figures 15 and 16 show the ride quality and rut depth measurements, respectively, for the two segments in the right lane having different AC overlay thicknesses. A rut depth of 0.5 in is commonly cited as a maximum allowable value. These figures show data from the first approximately 2,150 ft, constructed as 4 in of AC over 8 in of CCPR (denoted as *4 inch AC* and consisting of MP 217.66 to 217.25), and the next approximately 2,150 ft, constructed as 6 in of AC over 6 in of CCPR (denoted as *6 inch AC* and consisting of MP 217.25 to 216.84). From these two figures it can be seen that the IRI is greater in the 4-in AC section. However, it is not possible to determine if this was caused by the difference in overlay thickness or if it simply reflects the fact that this was the first section constructed on a project with a unique and difficult construction sequence. Figure 16 shows that the rut depth measurements in the 6-in AC section are slightly greater than in the 4-in AC section; however, these differences are not considered to be practically significant.

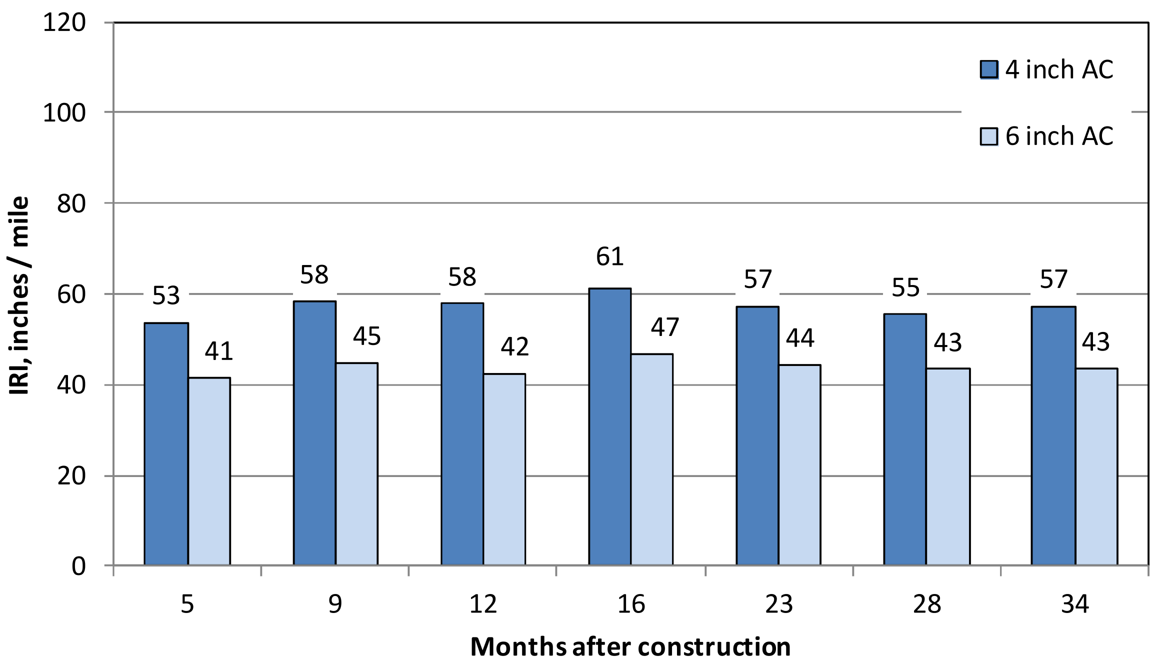

Figure 15. Average Ride Quality for 4-in and 6-in Asphalt Concrete (AC) Sections in Right Lane. IRI = International Roughness Index.

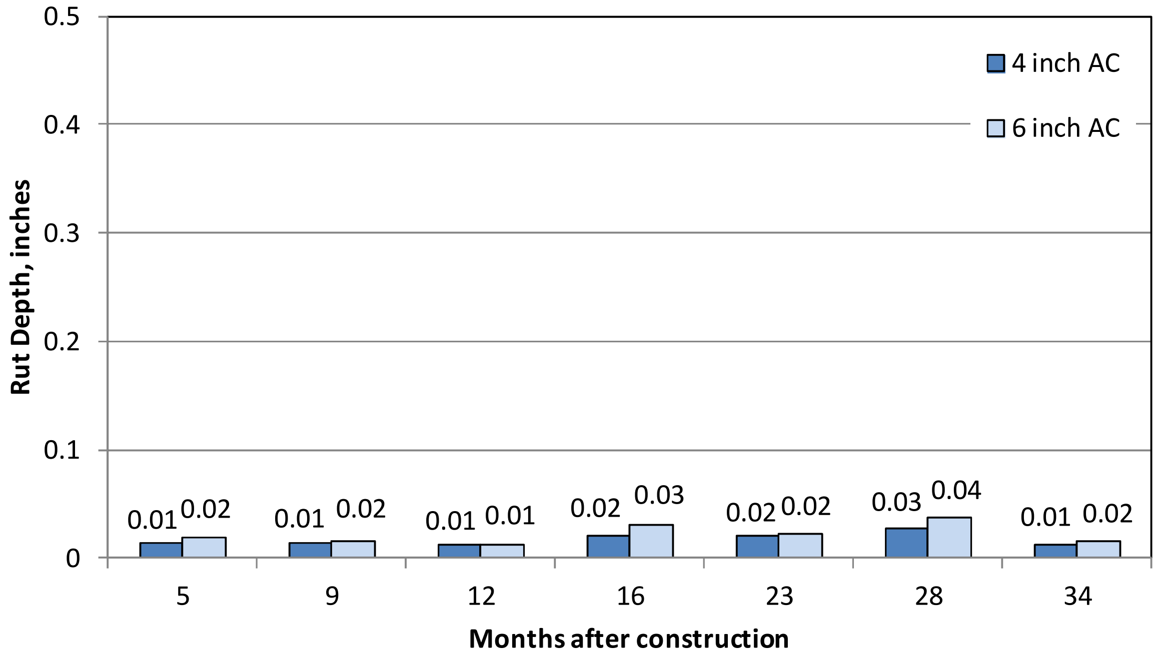

Figure 16. Average Rut Depth for 4-in and 6-in Asphalt Concrete (AC) Sections in Right Lane

Pavement Layer Thickness by Ground Penetrating Radar

GPR testing was conducted using a 2 GHz horn antenna at approximately 7 months after construction. Testing was conducted at traffic speed with the antenna located approximately in the center of each lane. The results of the GPR testing were analyzed to determine the layer thicknesses for use with FWD analysis.

For the right lane, the average depth of all asphalt layers (including AC and CCPR) was approximately 13.4 in. For the left lane, the average depth of all asphalt layers (including new AC, CIR, and old AC layers) was approximately 13.3 in.

Figures 17 and 18 show the average thickness values for the AC and CCPR layers in the right lane and the new AC, CIR, and existing AC layers in the left lane, respectively, at approximately 0.5-mile intervals throughout the project. Figure 17 highlights the change in construction in that the initial portion of the right lane (MP 217.66 to 217.25) was constructed as a 4-in AC layer over an 8-in CCPR layer and the remainder of the right lane was constructed as a 6-in AC layer over a 6-in CCPR layer. The variability in the thickness of the various layers seen in Figures 17 and 18 can be attributed to profile milling and adjustments to match cross slope. Prior to application of the final 2-in SMA overlay, the contractor elected to perform profile milling in the left lane.

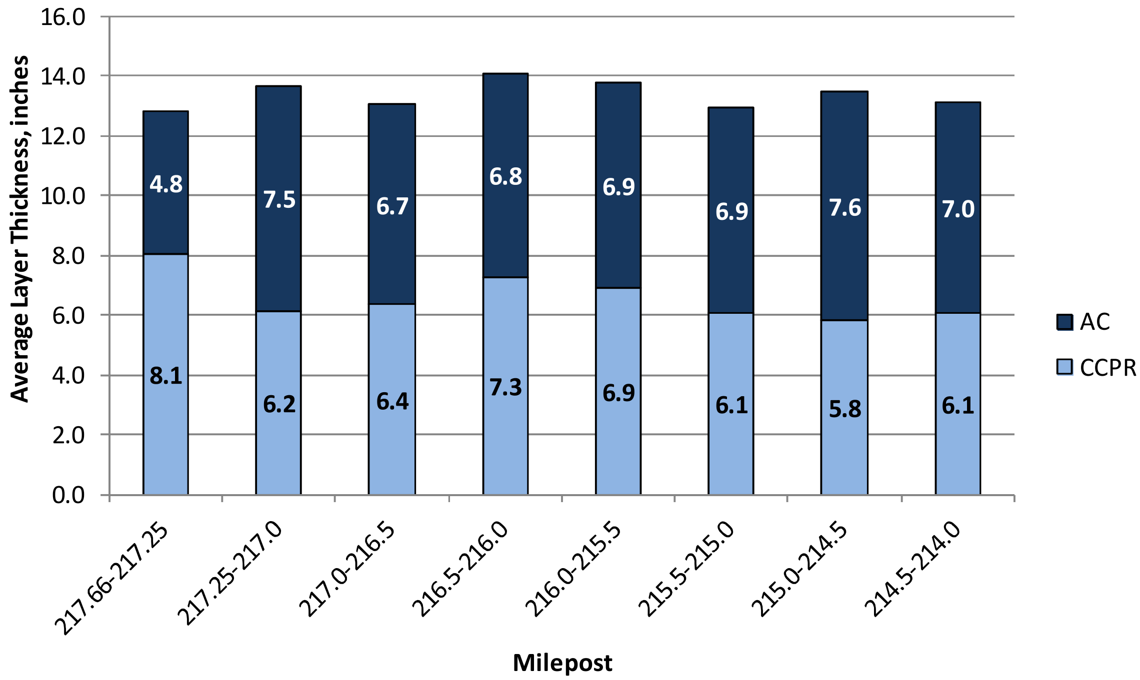

Figure 17. Average Pavement Thickness in Right Lane As Determined by Ground Penetrating Radar Testing (data labels indicate individual layer thicknesses). AC = asphalt concrete; CCPR = cold central-plant recycling.

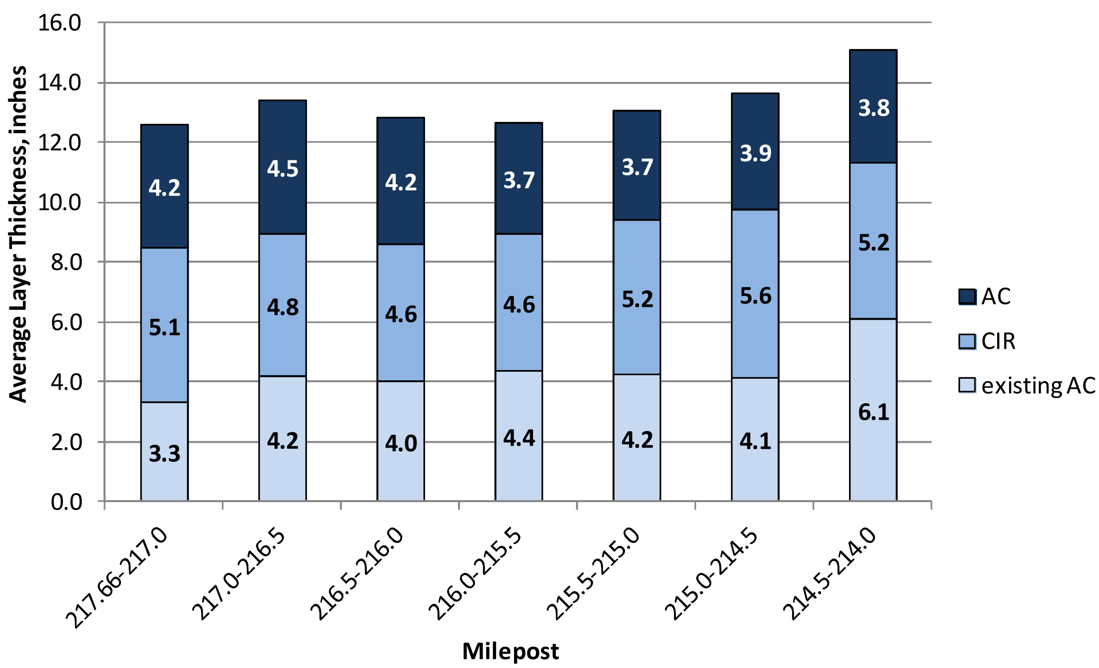

Figure 18. Average Pavement Thickness in Left Lane As Determined by Ground Penetrating Radar Testing (data labels indicate individual layer thicknesses). AC = asphalt concrete; CIR = cold in-place recycling.

Pavement Structural Capacity Measured by the Falling Weight Deflectometer

The results of FWD testing are shown in Table 11. Structural testing using an FWD was performed on December 16, 2011, and January 9, 2012; November 15 and 16, 2012; and October 30 and 31, 2013. These test dates correspond to approximately 6, 15, and 28 months after construction of the recycled layer. During the first round of FWD testing, the right lane was tested in its entirety on December 16, 2011. The first approximately 3 miles of the left lane was tested on December 16, 2011, and the remainder was tested on January 9, 2012, as weather and scheduling constraints caused a delay in testing.

From Table 11 it can be seen that the pavement stiffness increased between the tests at 6 and 15 months for both lanes and again between 15 and 28 months for the right lane. This behavior was expected based on previous experiences with recycled materials as described by Diefenderfer and Apeagyei (2011a). These data can be compared with FWD test results obtained for the right lane during the pre-construction test, conducted on March 3, 2011, approximately 3 months before construction (only the right lane was tested immediately prior to construction). This testing found the SN_{eff} of the right lane to average 3.9 (with a COV of 6.7%). The before-construction subgrade resilient modulus was found to average approximately 22,800 psi (with a COV of 22%). The data in Table 11 show an improvement in structural capacity for the right lane as compared to the pre-construction condition. In addition, it can be seen that the structural capacity for both lanes increased at early ages. Additional research should be conducted to see if similar performance is observed after continued long-term trafficking.

Table 11. Results of Falling Weight Deflectometer Testing

Approximate Months After Construction	Effective Structural Number, SN_{eff}		Deflection at Load Plate[a] (D_0), mils		Subgrade Resilient Modulus (M_R), psi		Pavement Modulus (E_p), psi	
	Average	COV	Average	COV	Average	COV	Average	COV
Right Lane								
6	8.97	5.5%	2.8	18.6%	52,578	34.1%	491,372	16.4%
15	9.86	8.8%	2.2	30.1%	66,588	57.7%	660,521	26.1%
28	9.89	8.5%	2.3	23.7%	59,029	41.3%	660,949	26.2%
Left Lane								
6	5.46	10.6%	6.5	25.6%	28,337	31.6%	247,207	33.7%
15	5.84	9.1%	5.2	23.3%	32,675	31.1%	297,100	28.7%
28	5.68	8.3%	5.2	21.6%	35,663	25.4%	273,112	26.2%

COV = coefficient of variation.
[a] Corrected to 68°F.

Figures C1 and C2, Appendix C, show the effective structural number and temperature-corrected deflection at the load plate averaged over approximately 0.5-mile intervals for the right lane, respectively. The figures also show the results averaged from MP 217.66 to 217.2, which is the segment where the AC was constructed approximately 4 in thick and the CCPR was constructed approximately 8 in thick. The remainder of the project was constructed having an approximately 6-in-thick AC layer and an approximately 6-in-thick CCPR layer. From Figure C1 it can be seen that the effective structural number of the 4-in AC section is comparable to that of the remainder of the project. Figures C3 and C4, Appendix C, show the effective structural number and temperature-corrected deflection at the load plate averaged over approximately 0.5-mile intervals for the left lane, respectively.

Assessing the Structural Layer Coefficient for CCPR, FDR, and CIR

AASHTO (1993) stated that the relative ability of a particular material to function as a structural component of a pavement can be expressed as that material's structural layer coefficient (a_i). In a multi-layered pavement structure, the summation of the layers' structural coefficients (a_i) multiplied by the respective layer thicknesses (D_i) gives the structural number (SN) for that pavement as follows:

$$SN = a_1 \times D_1 + a_2 \times D_2 + a_3 \times D_3 + \ldots \qquad \text{[Eq. 1]}$$

The required SN for a pavement is determined during the design phase and is calculated from the subgrade stiffness, traffic loading, and anticipated service life, among other factors. The measured deflection values from FWD testing can be used to calculate an effective structural number (SN_{eff}) that is indicative of the in-situ structural capacity of an existing pavement structure. From the SN_{eff}, an approach similar to that shown in Equation 1 can be used to calculate unknown layer coefficients.

Taking the most recent FWD test results shown in Table 11, the average effective structural numbers (SN_{eff}) at 28 months after construction for the right and left lanes were calculated as 9.89 and 5.68, respectively. Using the thickness data collected from the GPR survey (as shown in Figure 17), the pavement in the right lane can be assumed to be a three-layer pavement structure consisting of AC, CCPR, and FDR with average thicknesses of 7.1, 6.4, and 12 in, respectively. Since the bottom of the FDR layer was not identifiable in the GPR data, its thickness could not be measured; the design thickness was used for the thickness of this layer. Knowing the SN_{eff} of the right lane and the thickness for each layer, Equation 1 can be rewritten as follows:

$$SN_{eff(right)} = a_1 \times 7.1 + a_2 \times 6.4 + a_3 \times 12 \qquad \text{[Eq. 2]}$$

VDOT (2000) assumes a structural layer coefficient for new surface and intermediate AC layers (including SMA) to be 0.44. Substituting 0.44 for a_1 in Equation 2 still leaves a_2 and a_3 as unknowns. Since there is only one equation, a_i for Layers 2 and 3 can be determined only by assuming a relationship; in this case it was assumed that a_2 equals a_3. From this, Equation 2 can be rewritten as follows:

$$9.89 = 0.44 \times 7.1 + a_2 \times 6.4 + a_2 \times 12 \qquad \text{[Eq. 3]}$$

Solving for a_2, a value of 0.37 is calculated as a combined layer coefficient for the CCPR and FDR layers. No structural number results for CCPR were found in the literature; however, a maximum structural layer coefficient value of 0.33 was found by Diefenderfer and Apeagyei (2011b) for cement stabilized FDR at an age of 24 months. If a value of 0.33 is substituted for a_3 in Equation 1, a_2 is calculated as 0.44. A value of 0.44 is higher than typically reported in the literature; the researchers believe the actual structural layer coefficient for the CCPR layer on this project is between 0.37 and 0.44.

Similarly, an a_i value for the CIR materials in the left lane can be calculated by using the thickness data collected from the GPR survey (as shown in Figure 18). The pavement in the left lane can be assumed as a four-layer pavement structure consisting of AC, CIR, existing AC, and existing aggregate base with average thicknesses of 4.0, 5.0, 4.3, and 6.3 in, respectively. As with the FDR layer in the right lane, the bottom of the existing aggregate in the right lane was not identifiable in the GPR data collected after construction; however, an aggregate layer thickness of 6.3 in was measured using GPR before construction. For a four-layer pavement structure, Equation 1 can be rewritten using the known layer thicknesses as follows:

$$5.68 = 0.44 \times 4.0 + a_2 \times 5.0 + a_3 \times 4.3 + a_4 \times 6.3 \qquad \text{[Eq. 4]}$$

The structural layer coefficient a_2 represents the CIR material and is the variable to be determined. The structural layer coefficients a_3 and a_4 represent the existing AC and aggregate base materials, respectively. These values can be estimated using the results of FWD data from before construction.

As mentioned previously, FWD testing was conducted in the right lane approximately 3 months before construction. The average SN_{eff} was 3.89 with a subgrade resilient modulus of 22,800 psi. Unfortunately, FWD testing before construction was not conducted in the left lane so it was assumed that the right lane values were similar. From these values, Equation 1 can be rewritten as follows:

$$3.89 = a_1 \times 10.6 + a_2 \times 6.6 \qquad \text{[Eq. 5]}$$

where a_1 and a_2 are the structural layer coefficients for the existing AC and aggregate layers, respectively. AASHTO (1993) presented a relationship that can be used to estimate the structural layer coefficient for granular base layers (a_2) from the resilient modulus. This relationship is shown as follows:

$$a_2 = 0.249 \times \log_{10}(E_{BS}) - 0.977 \qquad \text{[Eq. 6]}$$

where E_{BS} is the resilient modulus in psi. Substituting a value of 22,800 psi into Equation 6 yields a value for a_2 of 0.11. Interestingly, this is close to the value of 0.12 assumed by VDOT (2000) for new aggregate base materials. Substituting a value of 0.11 into Equation 5 yields a value for a_1 of 0.29. This is taken as the in-situ layer coefficient for the asphalt materials in both the right and left lanes (from testing in the right lane only).

Finally, Equation 4 can be solved for a_2 by substituting the values of 0.29 and 0.11 for a_3 and a_4, respectively, yielding a result of 0.39 for the structural layer coefficient of the CIR materials. It is possible that the layer coefficient value for the CIR material could be less if the structural capacity of the existing AC materials in the left lane were not as deteriorated as the AC materials in the right lane before construction; although there are no data to prove it, the researchers believe this to be likely. As an example, if a_3 in Equation 4 were assumed to be 0.33, the structural layer coefficient for the CIR materials would be 0.36.

In addition to FWD testing, structural layer coefficient values for the recycled materials can be estimated using relationships with laboratory test results. Since CCPR, CIR, and FDR are a hybrid of AC and aggregate materials, Schwartz and Khosravifar (2013) stated that laboratory-measured stiffness values can be converted to a structural layer coefficient by assuming the recycled material to be an AC, non-stabilized aggregate or a stabilized aggregate material. In addition, Wirtgen (2010) provided a nomograph to correlate ITS with structural layer coefficient.

Schwartz and Khosravifar (2013) reported a relationship between the resilient modulus for an asphalt mixture at 68°F (20°C) and the structural layer coefficient (a_1) from AASHTO (1993) as follows:

$$a_1 = 0.1665 \times \ln(M_R) - 1.7309 \qquad \text{[Eq. 7]}$$

where M_R is the resilient modulus in psi.

Substituting the M_R results shown in Table 8 into Equation 7, layer coefficients ranging from 0.45 to 0.48 were calculated and are shown in Table 12. AASHTO (1993) stated that Equation 7 is valid for dense-graded AC surface courses and recommended caution for M_R values greater than 450,000 psi (AASHTO did not go so far as to state that the relationship between M_R and a_1 at these levels is not valid, but it is extrapolated). Equation 6 can be used to estimate the structural layer coefficient by assuming the recycled materials act as granular base layers. Substituting the “all” average M_R results shown in Table 8 into Equation 6, layer coefficients ranging from 0.44 to 0.46 were calculated and are shown in Table 12.

AASHTO (1993) also presented a nomograph that can be used to estimate the structural layer coefficient for bituminous-treated base materials from various laboratory tests including the resilient modulus. Although M_R values as high as those shown in Table 8 are not included, if the figure is extended vertically, a_2 values can be estimated as ranging from approximately 0.36 to 0.41. In an analysis of various methods to calculate layer coefficients of recycled materials, Schwartz and Khosravifar (2013) placed a lower emphasis on the relationship between M_R and a_2 since the basis for the nomograph was unclear.

Wirtgen (2004) presented a nomograph to describe the relationship between the dry ITS value and the structural layer coefficient. Table 7 shows the dry ITS values to be 77.2 (532.3 kPa) and 71.6 psi (493.7 kPa) for CCPR and CIR, respectively. Using this nomograph, the structural layer coefficient could be estimated as approximately 0.37 and 0.35 for the CCPR and CIR, respectively. In their analysis, Schwartz and Khosravifar (2013) also placed a low emphasis on the relationship between ITS and structural layer coefficient since the basis for the nomograph was unclear.

Table 12. Structural Layer Coefficient Results Based on Relationships With M_R

Process	Location	Laboratory Measured M_R, psi	Structural Layer Coefficient Estimate		
			Equation 7	Equation 6	AASHTO (1993)
CCPR	Core top	574,035	0.48	0.46	0.41
	Core all	486,546	0.45	0.44	0.36
CIR	Core top	554,745	0.47	0.45	0.40
	Core all	577,597	0.48	0.46	0.41

CCPR = cold central-plant recycling; CIR = cold in-place recycling.

SUMMARY OF FINDINGS

Literature Review

- Active fillers have often been combined with asphalt-based recycling agents to improve resistance to the detrimental effects of moisture and to improve early strength.

- On higher volume routes, an AC overlay has typically been placed over recycled materials, but functional treatments (such as chip seals) have been used on lower volume facilities.

- Studies describing testing on in-service construction projects have shown that BSMs can achieve gains in stiffness from early ages to approximately 2 years after construction.

- Structural layer coefficient values for recycled materials have typically been reported as ranging from approximately 0.25 to 0.35 (with FDR and CIR/CCPR tending to be on the lower or upper end of this range, respectively).

- A 10-year performance review of in-place recycling in Ontario has shown that CIR is an effective pavement rehabilitation option with deterioration rates similar to those for conventional pavement rehabilitation practices.

Construction Description for the Study Project

- An in-place pavement recycling project was successfully constructed on a high-volume interstate facility. The unique traffic management plan showed that with planning prior to construction, a pavement rehabilitation strategy can be employed that allows the contractor a much longer construction window than normally offered by typical night-time closures.

  — The recycling and initial AC overlays for the right lane were completed in four 5-day closure periods between April 16 and May 24, 2011. The recycling and initial AC overlay for the left lane was completed in a single 3-day closure between June 4 and June 6, 2011.

  — No tack coat was used between the recycled layers and AC overlay in either lane.

  — Approximately 4 to 6 hours passed between placement of the FDR and placement of the CCPR material at any one location. The 4-in AC overlay was applied to the CCPR after approximately 8 to 16 hours. The decision regarding when to pave the AC overlay was based on nuclear density gauge measurements showing a moisture content that was less than one-half the as-placed moisture content. The pavement was opened to traffic following the AC overlay, and then the 2-in SMA wearing course was added after approximately 8 weeks.

  — The entire work in the left lane was completed within 3 workdays. During the first 2 days, an average of approximately 8,000 ft was completed each day. A 2-in AC overlay was applied to the CIR after approximately 2 days, and the pavement was opened to traffic. The 2-in SMA wearing course was added after approximately 6 weeks.

Construction Quality Control and Acceptance Testing for the Study Project

- The average field density exceeded the desired minimum density, which was 98% of the modified Proctor value established in accordance with AASHTO T 180-10, Standard Method of Test for Moisture-Density Relations of Soils Using a 4.4-kg (10-lb) Rammer and a 457-mm (18-in.) Drop, Method D (AASHTO, 2013).

- In 16 of the 19 sublots for the right lane, the ITS of the CCPR material exceeded the desired minimum. In 12 of the 16 sublots for the left lane, the ITS of the CIR material exceeded the desired minimum.

- The average gradations for the CCPR and CIR processes from cores were similar, but the CCPR materials were slightly finer than the CIR materials. The gradation was on the finer side of the recommended values, especially on the coarser sieve sizes.

Laboratory Evaluation for the Study Project

- The differences in density between the top and bottom of the CCPR and CIR specimens were statistically significant; however, differences in density from the top portions of the cores from the two processes were not statistically significant.

- The differences in ITS from the top portions of the cores from the two processes were not statistically significant.

- The differences in ITS between the CCPR and CIR processes were not statistically significant.

- The M_R values showed that both the CIR and CCPR exhibited a temperature-dependent behavior; i.e., the materials become less stiff at higher temperatures, similar to AC mixtures.

- It is unclear if removing materials greater than ¾ in adversely affected the results of tests conducted on laboratory specimens prepared from field-produced materials.

- The confined FN test results were much higher than the unconfined FN test results for both the CCPR and CIR materials.

- The dynamic modulus for the CCPR materials from cores was generally stiffer than that for the CIR materials from cores at low and intermediate reduced frequencies. The CIR materials from cores were generally stiffer at high reduced frequencies than the CCPR materials from cores.

- The structural layer coefficients for the CCPR and CIR materials were estimated by relationships with M_R to range from 0.36 to 0.48 and from 0.40 to 0.48, respectively.

- The structural layer coefficients for the CCPR and CIR materials were estimated by relationships with ITS as 0.37 and 0.35, respectively.

Field Evaluation for the Study Project

- Manual rut depth measurements collected soon after construction showed the rut depth of the first two sections of the right lane to be less than 0.15 in approximately 3 months after construction. Statistical testing found differences in the manually measured rut depth between the first two sections, but the values were so small as not to be statistically significant.

- Rut depth measurements collected using a traffic-speed inertial profiler showed the rut depths to be less than 0.1 in for the left lane and less than 0.05 in for the right lane after 34 months of service. Given the limitations of the testing equipment, the actual values are not as important as the overall trend suggesting that the rut depth is not increasing after 34 months of service.

- Ride quality measurements by a traffic-speed inertial profiler indicated improvements in IRI after construction. The IRI was about 56 and 45 in per mile for the left and right lanes, respectively, after 34 months of service. These values can be compared to the preconstruction IRI value of 72 in per mile. Given the limitations of the testing equipment, the actual values are not as important as the overall trend suggesting that the ride quality is not decreasing after 34 months of service.

- The right lane, consisting of AC and CCPR over FDR, has a lower IRI than the left lane, consisting of AC over CIR. However, it is not clear if the difference is due to the difference in the depth of AC overlay, the number of layers constructed, or the recycling processes.

- The initial portion of the right lane, constructed as 4 in of AC over 8 in of CCPR, had a higher IRI when compared to that of the remainder of the right lane, constructed as 6 in of AC over 6 in of CCPR. It is unclear if this difference is due to the difference in layer thicknesses or a result of the contractor gaining experience as the project progressed.

- The average thickness of all asphalt layers in the right lane (including AC and CCPR) and the left lane (including new AC, CIR, and old AC layers) was approximately 13.4 and 13.3 in, respectively, as measured by GPR.

- FWD testing showed the structural capacity of the right lane to be greatly improved by the recycling project when compared to the pre-construction effective structural number of 3.9 measured in the right lane. FWD testing showed the SN to be approximately 9.9 and 5.7 in the right and left lanes, respectively. FWD testing showed that the effective structural number of the 4-in AC section (with 8-in CCPR) in the right lane was comparable to that of the remainder of the right lane having 6-in AC (with 6-in CCPR).

- The combined structural layer coefficient for the CCPR and FDR materials was calculated from the results of FWD testing as 0.37. The structural layer coefficient for the CIR material was calculated from the results of FWD testing as 0.39. The structural layer coefficient for the CCPR material was calculated to have a likely range of 0.37 to 0.44.

CONCLUSIONS

- *Active fillers (such as cement) can be used to improve resistance to the detrimental effects of moisture and improve the early strength of recycled materials.*

- *Functional treatments (such as chip seals) can be used to surface recycled materials on lower volume roadways.*

- *Structural layer coefficient values from this study suggest that the layer coefficients of the CCPR and CIR placed on I-81 are within the range of 0.36 to 0.44 and 0.35 to 0.39, respectively.*

- *Structural layer coefficient values from this study suggest that the combined layer coefficient of CCPR and FDR placed on I-81 is 0.37.*

- *The combination of foamed asphalt as a recycling agent, cement as an active filler, and construction at less than optimum moisture content are advantageous in that the AC overlay can be placed approximately 8 to 16 hours after construction of the CCPR layer.*

- *During construction, CCPR and CIR layers are generally able to meet or exceed 98% of the modified Proctor density requirements based on AASHTO T 180, Standard Method of Test for Moisture-Density Relations of Soils Using a 4.4-kg (10-lb) Rammer and a 457-mm (18-in.) Drop, Method D.*

- *The combination of public awareness, traffic control, and traffic operations and the unique traffic management plan were key components to the successful and safe completion of the construction project.*

- *It is unclear if removing materials greater than ¾ in adversely affected the results of laboratory tests conducted on specimens prepared from field-produced materials.*

- *Based on ITS and M_R laboratory testing, the performance of CCPR and CIR is expected to be similar. Dynamic modulus testing indicated, however, that the CCPR material might have a better performance at higher temperatures.*

- *From the results of FWD testing and relationships with laboratory tests, the structural layer coefficients for the recycled materials placed on I-81 were higher than those values typically assumed by VDOT.*

- *The field performance tests demonstrated that the section of pavement rehabilitated by the three in-place recycling methods continues to perform well after nearly 3 years of high-volume interstate traffic.*

RECOMMENDATIONS

1. *VDOT's Materials Division should encourage the VDOT districts to pursue in-place recycling as a pavement rehabilitation technique on those asphalt pavement sections where it is most suitable.* Other agencies have shown considerable per-project cost savings when these techniques were employed.

2. *VDOT's Materials Division and VCTIR should work to help the VDOT districts scope and evaluate potential in-place recycling projects as VDOT builds experience with in-place recycling techniques.* This effort would keep the process from becoming overly burdensome for district staff.

3. *VDOT's Materials Division should consider increasing the structural layer coefficients used in the design for recycled materials based on the findings of this study.*

4. *VCTIR should continue to assess the performance of the I-81 project and other projects in an effort to develop long-term performance data.* This work could be done by FWD testing and laboratory testing of collected cores.

5. *VCTIR and VDOT's Materials Division should assess the as-constructed properties of future pavement recycling projects for the purpose of compiling a materials characterization database.*

6. *VDOT's Traffic Engineering Division should consider long-term lane closure strategies similar to those employed in this project on other major pavement rehabilitation projects.* The safety of both the traveling public and workers was likely increased by minimizing the construction time required to complete the work.

COSTS AND BENEFITS ASSESSMENT

Direct cost savings to VDOT from using in-place pavement recycling on I-81 as compared with using conventional pavement rehabilitation methods are difficult to estimate. Determination of the costs of alternatives to conventional pavement rehabilitation would likely be highly subjective and greatly influenced by the traffic control options selected. The need to ensure the safety of the traveling public and construction personnel, as well as minimize construction-related traffic congestion, weighs heavily in the determination of feasible options for interstate rehabilitation. Such alternative rehabilitation methods could range from conventional paving with AC within the original footprint of the pavement section rehabilitated in this project—including FDR or deep undercutting and replacement with an aggregate base—to constructing an additional lane throughout the project limits to ensure at least two full through lanes of traffic during construction. Depending on the alternative that could have been chosen instead of the pavement recycling used on this project, the estimated cost savings for this project ranged from a few percent of the contract price (approximately $7.9 million as bid) to more than $70 million.

Not considered were the savings gained from avoiding delays by using pavement recycling to reduce construction time. Again, depending on whatever other alternative might have been selected, it is estimated that VDOT reduced construction time for the I-81 pavement recycling project by several weeks to nearly 1 year.

Although not the primary objective of this study, the traffic management plan this project employed should serve as an example for other complex pavement reconstruction projects in Virginia—whether recycling or conventional paving projects—of what can be accomplished when alternative traffic management strategies are considered.

More definitive, but still not quantifiable, were the extensive knowledge and experience gained by VDOT and the local, national, and international construction industry in conducting a major pavement recycling project on the U.S. interstate system.

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APPENDIX A

DETAILS OF LABORATORY TESTING

The tables in this appendix show details of the laboratory testing conducted on field-produced and laboratory-produced specimens.

Table A1. Bulk Density Testing Results

CCPR (right lane)				CIR (left lane)			
Top of Core		Bottom of Core		Top of Core		Bottom of Core	
VCTIR Lab ID	Bulk Density, lb/ft^3	VCTIR Lab ID	Bulk Density, lb/ft^3	VCTIR Lab ID	Bulk Density, lb/ft^3	VCTIR Lab ID	Bulk Density, lb/ft^3
11-1009 8T	138.2	11-1009 3B	129.3	11-1026 5T	144.9	11-1026 5B	135.9
11-1009 12T	137.5	11-1009 8B	126.4	11-1026 6T	141.9	11-1026 6B	139.0
11-1025 3T	139.3	11-1009 12B	126.7	11-1026 7T	142.9	11-1026 7B	135.3
11-1025 5T	139.9	11-1025 3B	128.4	11-1026 15T	136.8	11-1026 15B	127.5
11-1025 11T	141.1	11-1025 5B	130.7	11-1037 5T	144.2	11-1026 17B	121.2
		11-1025 11B	130.4	11-1037 16T	140.3	11-1037 3B	132.4
						11-1037 9B	132.6
						11-1037 16B	135.5

Table A2. Indirect Tensile Strength Test Results

CCPR (right lane)				CIR (left lane)			
Top of Core		Bottom of Core		Top of Core		Bottom of Core	
VCTIR Lab ID	ITS, psi	VCTIR Lab ID	ITS, psi	VCTIR Lab ID	ITS, psi	VCTIR Lab ID	ITS, psi
11-1009 8T	106.5	11-1009 3B	59.5	11-1026 5T	96.49	11-1026 5B	72.72
11-1009 12T	92.5	11-1009 8B	45.1	11-1026 6T	65.04	11-1026 6B	57.4
11-1025 3T	69.51	11-1009 12B	55.1	11-1026 7T	75.02	11-1026 7B	67.21
11-1025 5T	61.91	11-1025 3B	30.97	11-1026 15T	72.59	11-1026 15B	67.82
11-1025 11T	73.52	11-1025 5B	51.27	11-1037 5T	73.1	11-1026 17B	54.62
		11-1025 11B	50.3	11-1037 16T	71.55	11-1037 3B	77.61
						11-1037 9B	45.86
						11-1037 16B	76.45

Table A3. Indirect Tensile Strength Test Results for Moisture Susceptibility

CCPR (right lane)				CIR (left lane)			
Dry specimens		Wet specimens		Dry specimens		Wet specimens	
VCTIR Lab ID	ITS, psi	VCTIR Lab ID	ITS, psi	VCTIR Lab ID	ITS, psi	VCTIR Lab ID	ITS, psi
11-1009 RC6	59.0	11-1009 RC4	36.0	11-1026 13	60.0	11-1026 10	40.0
11-1009 RC18	70.1	11-1009 RC17	42.1	11-1037 2	64.0	11-1026 12	43.9
11-1009 RC1	71.1	11-1009 RC10	54.0	11-1037 4	65.0	11-1026 3	52.9
11-1009 RC16	80.9	11-1025 6	67.0	11-1026 11	65.0	11-1037 13	54.0
11-1025 4	84.0	11-1025 12	71.9	11-1026 16	66.0	11-1037 15	54.0
11-1025 1	87.0			11-1026 2	71.1	11-1026 8	55.0
11-1025 2	88.0			11-1037 6	73.0	11-1026 14	56.0
				11-1037 8	73.0	11-1037 12	56.0
				11-1037 1	78.0	11-1026 1	61.9
				11-1026 4	79.0	11-1037 11	62.9
				11-1026 9	79.0	11-1037 14	65.0
				11-1037 7	86.0	11-1037 10	78.0

Table A4. Resilient Modulus Testing on CCPR (Right Lane) Materials

Location	VCTIR Lab ID	Resilient Modulus, psi		
		4°C	20°C	38°C
Top	11-1009 8T	2,433,588	794,081	219,732
	11-1009 12T	1,167,263	715,471	294,572
	11-1025 3T	1,113,164	357,663	190,289
	11-1025 5T	1,169,584	484,281	-
	11-1025 9T	954,493	550,273	216,831
	11-1025 11T	1,148,409	542,441	256,862
Bottom	11-1009 3B	785,089	579,861	268,755
	11-1009 8B	634,540	315,747	261,938
	11-1009 12B	596,685	329,671	340,113
	11-1025 3B	886,760	380,579	139,236
	11-1025 5B	533,739	429,457	390,006
	11-1025 9B	892,562	403,205	198,557
	11-1025 11B	883,135	442,365	176,366

Table A5. Resilient Modulus Testing on CIR (Left Lane) Materials

Location	VCTIR Lab ID	Resilient Modulus, psi		
		4°C	20°C	38°C
Top	11-1026 5T	1,605,422	635,120	336,197
	11-1026 6T	1,130,134	694,150	189,129
	11-1026 7T	1,027,447	424,235	231,770
	11-1026 15T	1,010,623	526,487	290,656
	11-1037 5T	1,197,141	600,021	267,304
	11-1037 17T	1,046,302	448,457	185,068
Bottom	11-1026 5B	1,065,157	530,693	341,709
	11-1026 6B	1,490,697	567,533	248,595
	11-1026 7B	1,007,867	414,953	152,725
	11-1026 15B	863,264	492,113	278,907
	11-1026 17B	968,417	749,700	320,098
	11-1037 3B	793,066	537,510	189,999
	11-1037 9B	943,615	423,510	125,893
	11-1037 16B	1,398,744	1,041,806	930,852

Table A6. Flow Number Testing on CCPR (Right Lane) Materials; Field-Produced, Lab-Compacted

Test date	5/2/2011						5/11/2011				
Sieved?	No						Yes				
Confining Stress, psi	0			10			0		10		
Deviator Stress, psi	30			70			30		70		
Sample #	1	2	3	1	2	3	1	2	1	2	3
FN	60	30	56	693	949	829	1745	1087	10000*	10000*	10000*

Test date	5/21/2011					
Sieved?	Yes					
Confining Stress, psi	0			10		
Deviator Stress, psi	30			70		
Sample #	1	2	3	1	2	3
FN	68	83	46	2935	1329	1481

*Tests were stopped at 10,000 cycles.

Table A7. Flow Number Testing on CIR (Left Lane) Materials; Field-Produced, Lab-Compacted

Test date	6/4/2011					
Sieved?	No					
Confining Stress, psi	0			10		
Deviator Stress, psi	30			70		
Sample #	1	2	3	1	2	3
FN	52	35	30	747	732	838

Table A8. Flow Number Testing on CCPR (Right Lane) Materials; Field Cores

VCTIR Lab ID	11-1009						13-1004			
Confining Stress, psi	0			10			0			
Deviator Stress, psi	30			70			70			
Sample #	1	2	3	1	2	3	1	2	3	4
FN	3806	1242	3234	8303	9170	10000*	950	200	72	15

*Test was stopped at 10,000 cycles.

Table A9. Flow Number Testing on CIR (Left Lane) Materials; Field Cores

VCTIR Lab ID	13-1005		
Confining Stress, psi	0		
Deviator Stress, psi	70		
Sample #	1	2	3
FN	53	77	55

Table A10. Dynamic Modulus Testing on CCPR (Right Lane) Materials

Temperature, °C	Specimen #	25 Hz	10 Hz	5 Hz	1 Hz	0.5 Hz	0.1 Hz
4.4	1	1,011,783	1,036,584	1,125,783	1,002,211	953,188	821,929
	2	773,921	717,356	692,845	604,372	581,311	499,365
	3	650,929	584,502	573,044	507,777	495,884	439,754
	4	845,425	774,211	753,471	660,792	653,395	572,754
	Average	820,515	778,164	786,286	693,788	670,944	583,450
	COV, %	18%	24%	30%	31%	30%	29%
21.1	1	828,310	752,456	700,967	570,143	520,540	410,602
	2	538,525	481,525	447,296	354,907	315,167	238,152
	3	457,449	431,777	418,724	337,793	328,946	263,824
	4	596,830	549,403	514,159	404,510	377,678	292,106
	Average	605,279	553,790	520,286	416,838	385,583	301,171
	COV, %	26%	25%	24%	25%	24%	25%
37.8	1	546,212	473,548	426,701	323,579	290,946	222,198
	2	330,251	277,892	248,450	172,595	155,190	110,243
	3	365,640	309,075	278,037	197,251	180,282	134,406
	4	374,197	315,167	279,053	196,381	176,511	126,792
	Average	404,075	343,921	308,060	222,452	200,732	148,410
	COV, %	24%	26%	26%	31%	30%	34%
54.4	1	320,968	264,549	233,221	167,228	151,854	116,726
	2	169,549	130,969	111,592	74,723	66,717	48,269
	3	189,999	146,488	126,908	88,604	81,482	63,585
	4	192,465	145,908	123,877	82,700	73,404	53,504
	Average	218,245	171,978	148,899	103,314	93,364	70,521
	COV, %	32%	36%	38%	42%	42%	45%

Table A11. Dynamic Modulus Testing on CIR (Left Lane) Materials

Temperature, °C	Specimen #	25 Hz	10 Hz	5 Hz	1 Hz	0.5 Hz	0.1 Hz
4.4	1	685,158	641,067	614,525	502,991	483,846	385,655
	2	1,307,515	1,227,309	1,165,088	985,386	920,844	742,013
	3	918,669	852,532	817,287	683,998	647,013	520,395
	Average	970,447	906,969	865,633	724,125	683,901	549,354
	COV, %	32%	33%	32%	34%	32%	33%
21.1	1	408,861	359,693	328,800	233,221	208,854	139,860
	2	734,616	660,067	604,807	439,899	391,892	268,465
	3	509,953	454,983	421,915	305,739	278,182	192,320
	Average	551,143	491,581	451,841	326,286	292,976	200,215
	COV, %	30%	31%	31%	32%	32%	32%
37.8	1	272,816	203,488	169,114	102,063	83,730	49,501
	2	454,403	363,029	306,465	187,969	158,091	98,045
	3	309,655	246,709	216,541	138,250	123,775	80,626
	Average	345,625	271,075	230,707	142,761	121,866	76,058
	COV, %	28%	30%	30%	30%	31%	32%
54.4	1	101,004	63,396	47,891	26,658	21,480	13,459
	2	202,473	138,758	109,997	60,307	51,503	33,431
	3	159,106	109,358	87,458	51,677	43,265	28,863
	Average	154,194	103,837	81,782	46,214	38,749	25,251
	COV, %	33%	37%	38%	38%	40%	41%

APPENDIX B

DETAILS OF FUNCTIONAL PAVEMENT TESTING

The figures in this appendix show details of the ride quality and rut depth measurements.

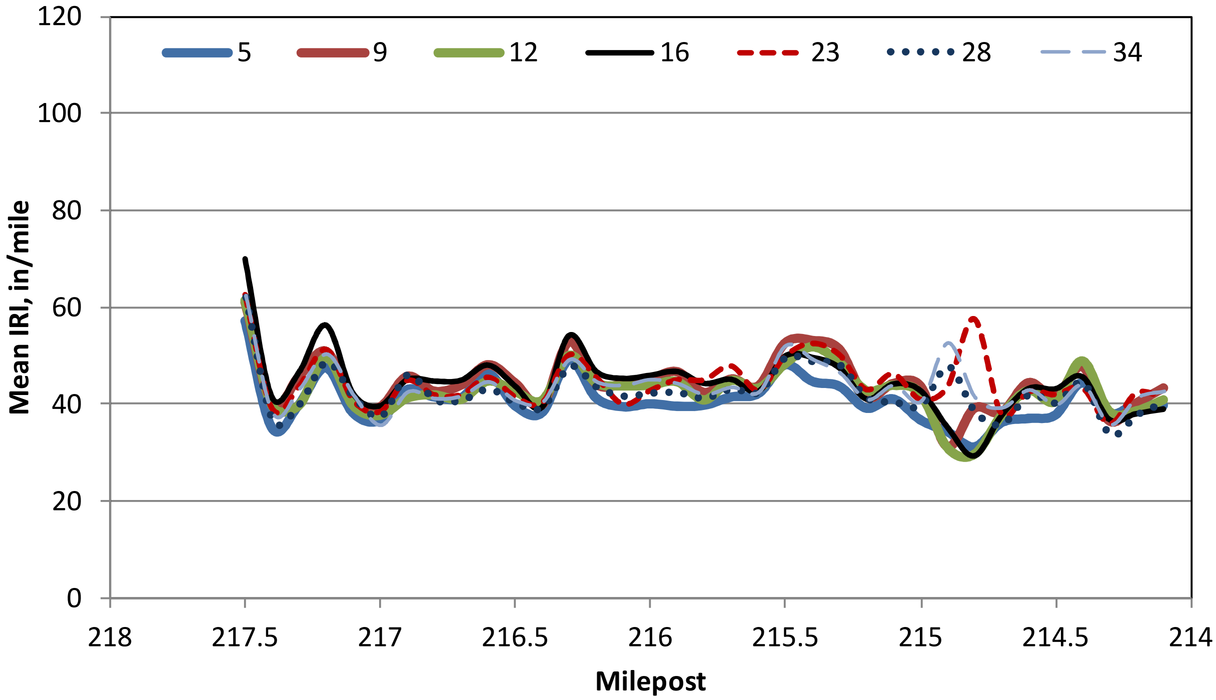

Figure B1. Ride Quality of Right Lane Between 5 and 34 Months After Construction Averaged Every 0.1 Mile

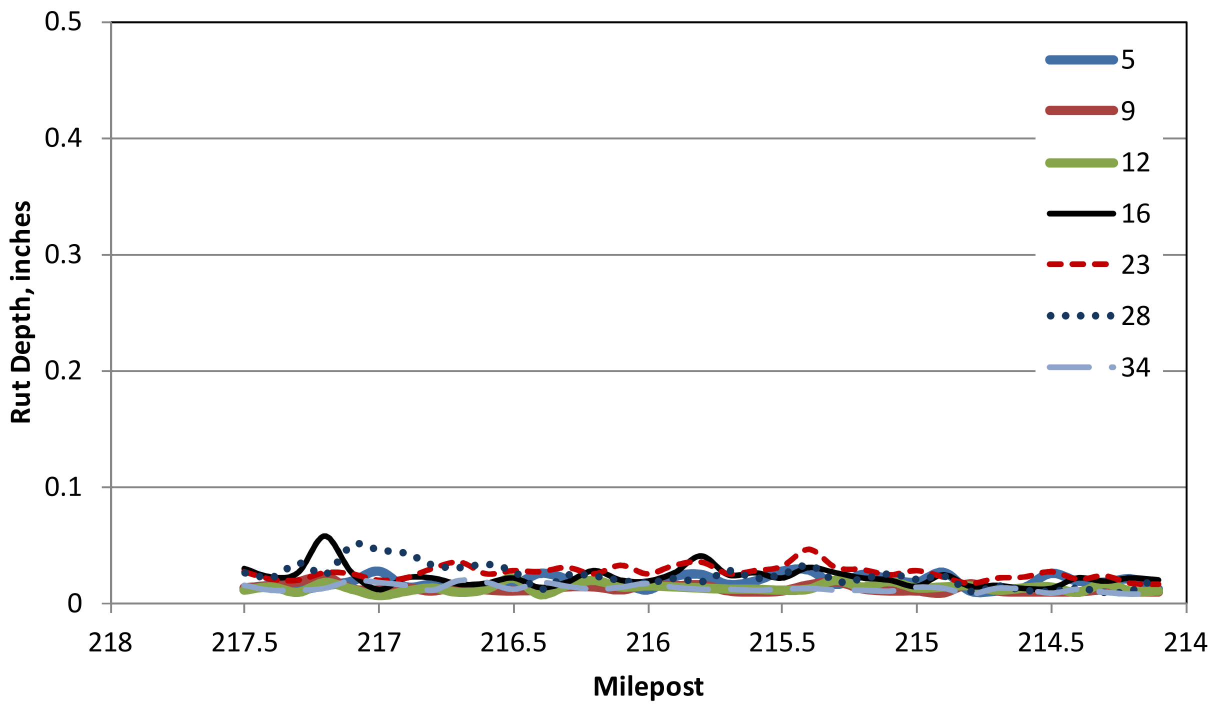

Figure B2. Rut Depth of Right Lane Between 5 and 34 Months After Construction Averaged Every 0.1 Mile

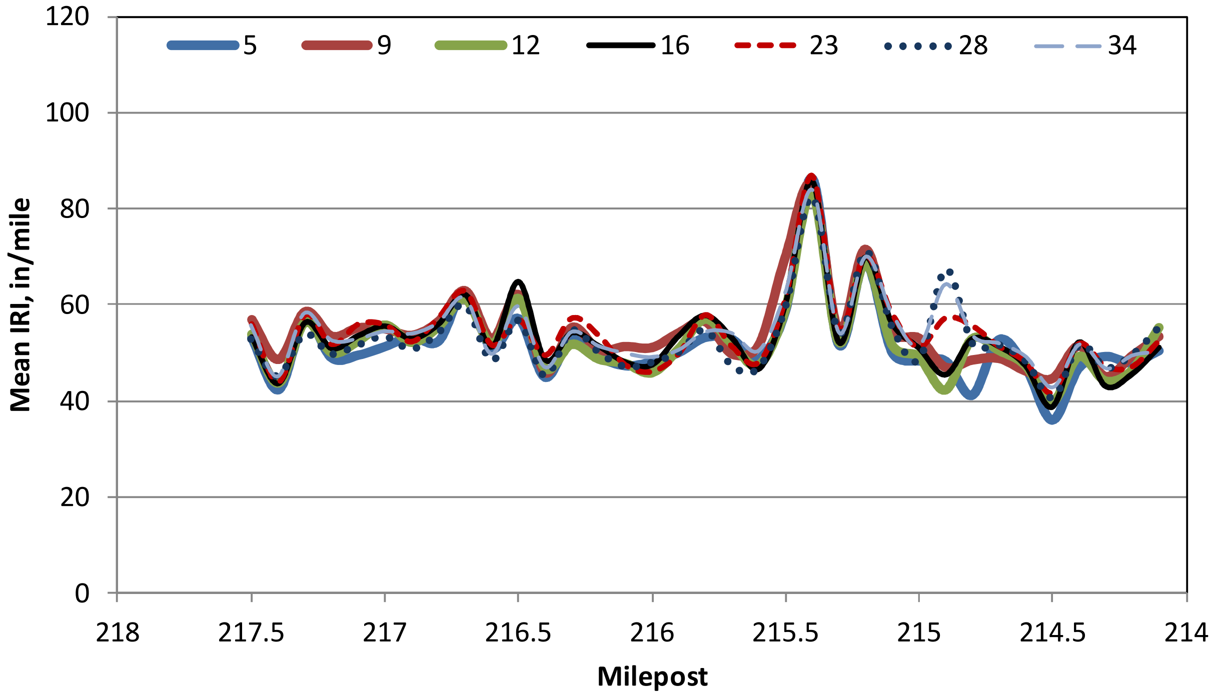

Figure B3. Ride Quality of Left Lane Between 5 and 34 Months After Construction Averaged Every 0.1 Mile

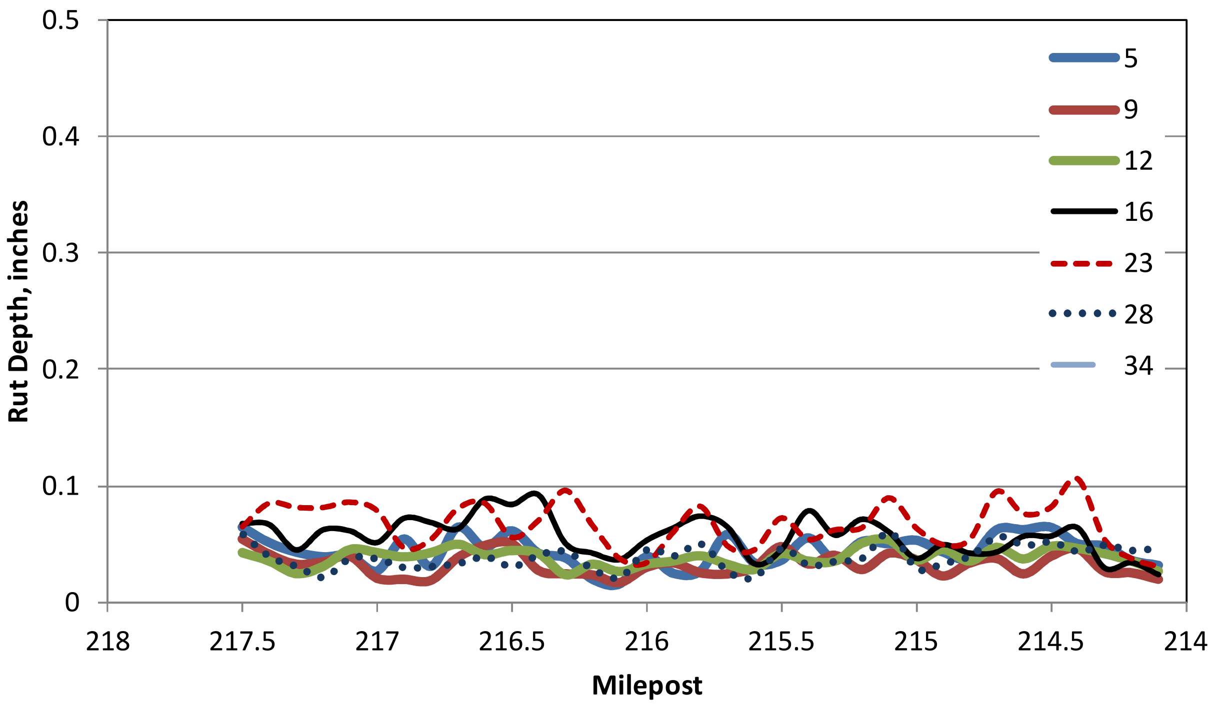

Figure B4. Rut Depth of Left Lane Between 5 and 34 Months After Construction Averaged Every 0.1 Mile

APPENDIX C

DETAILS OF FALLING WEIGHT DEFLECTOMETER TESTING

The figures in this appendix show details of the falling weight deflectometer testing conducted after construction in the right and left lanes.

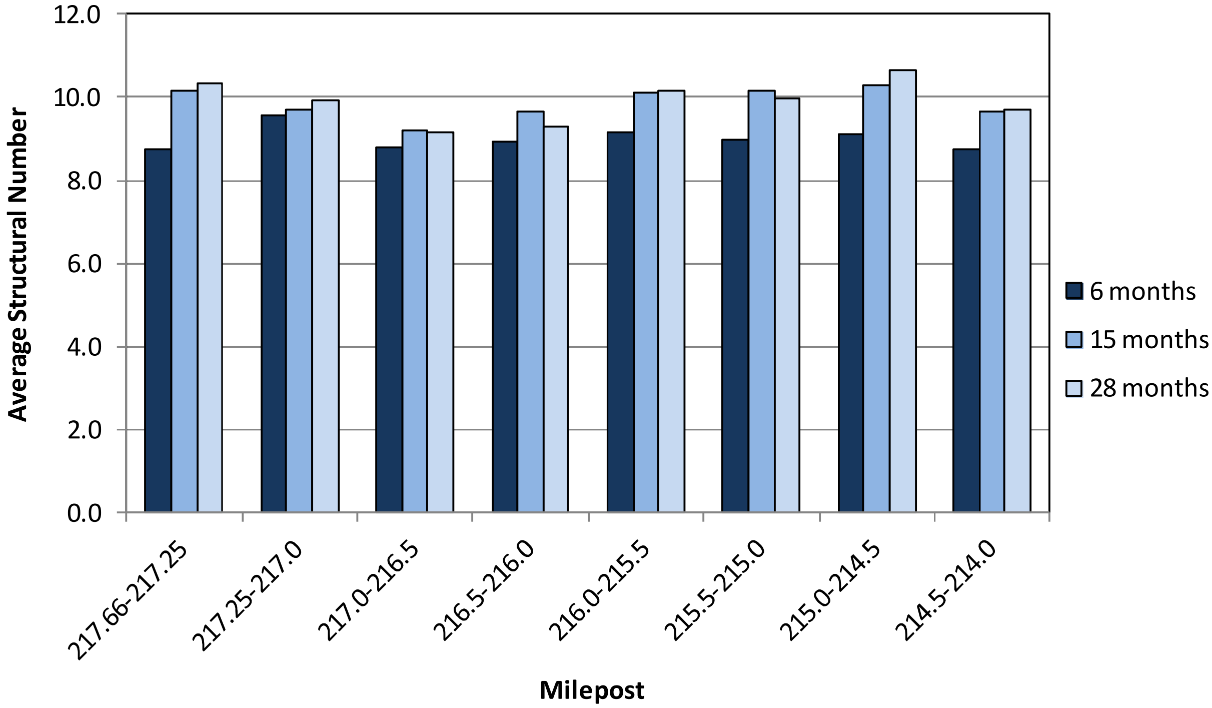

Figure C1. Average Structural Number (SN_{eff}) at 6, 15, and 28 Months After Construction, Right Lane

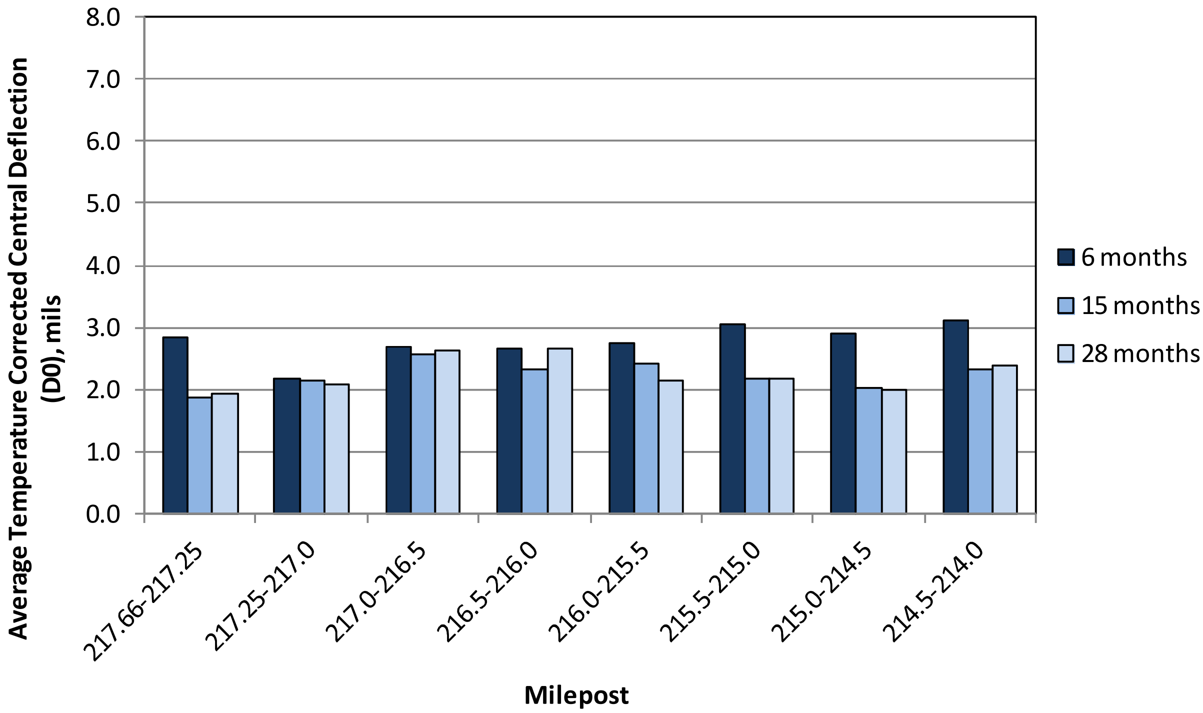

Figure C2. Average Temperature-Corrected Deflection at Load Plate (D0) at 6, 15, and 28 Months After Construction at 9,000 lbf Load Level, Right Lane

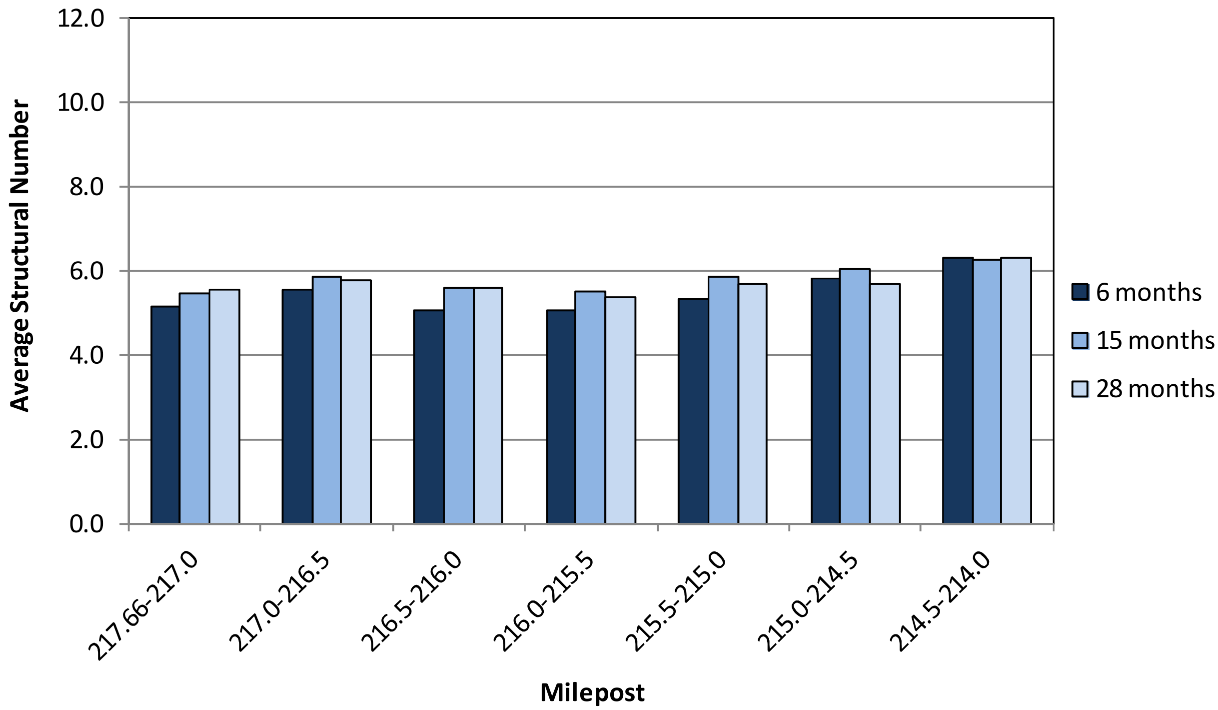

Figure C3. Average Structural Number (SN_{eff}) at 6, 15, and 28 Months After Construction, Left Lane

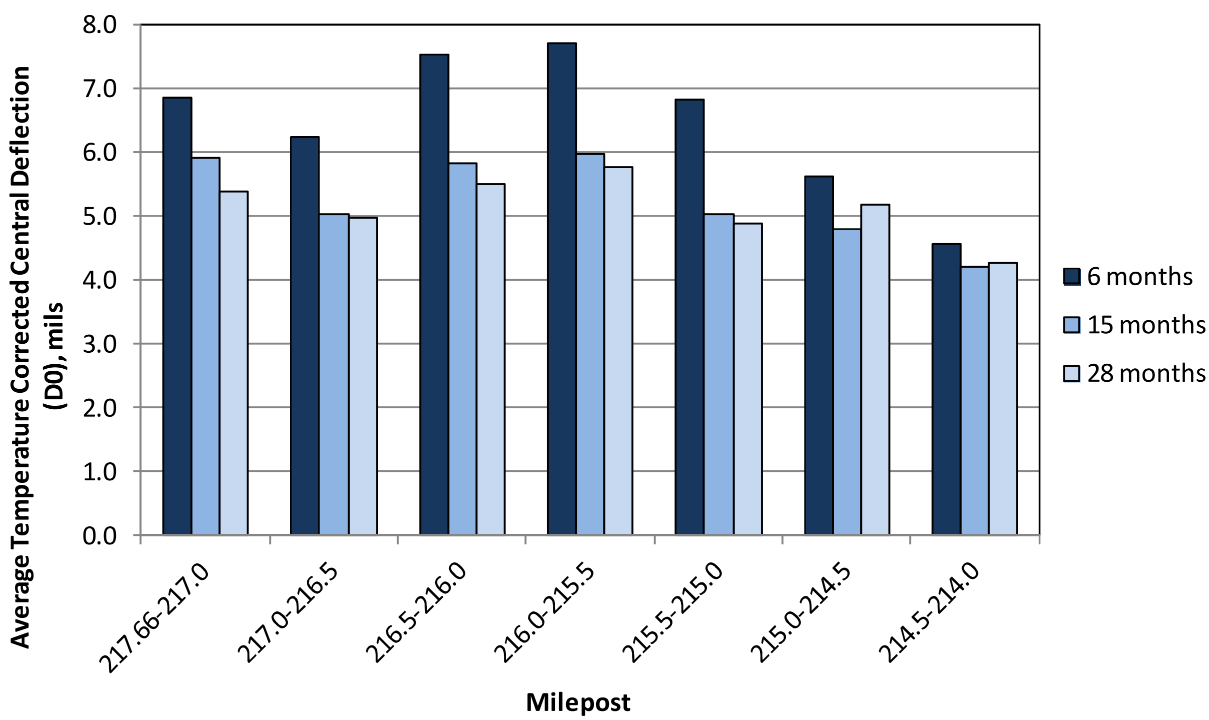

Figure C4. Average Temperature-Corrected Deflection at Load Plate (D0) at 6, 15, and 28 Months After Construction at 9,000 lbf Load Level, Left Lane