Initial Performance of Virginia's Interstate 81 In-Place Pavement Recycling Project

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Rehabilitating or reconstructing existing asphalt pavements by applying in-place pavement recycling techniques has been shown to be more cost-effective and more environmentally friendly than with traditional techniques. However, the scanty information in the literature documenting the structural and functional performance of these processes for high-volume roadways, leads to a perception by pavement engineers that the techniques are suitable only for lower-volume roads. This paper presents performance data for the 3.66-mi section of Interstate 81 in Virginia that was rehabilitated through the use of full-depth reclamation, cold central-plant recycling, and cold in-place recycling during the 2011 construction season. The performance data used to document the pavement condition included the structural capacity obtained by a falling-weight deflectometer and rut depth and ride quality measurements collected by an inertial profiler. The structural layer coefficients for the recycled materials were at the upper range of values reported in the literature. The pavement sections continued to perform well in regard to their functional and structural condition after the application of approximately 6 million equivalent single-axle loads and nearly 3 years of service. Two sections with different overlay thicknesses in the right lane also performed well after nearly 3 years of service.

In-place pavement recycling includes those technologies that can be used to rehabilitate or reconstruct a distressed asphalt pavement while using little to no virgin materials. In general, in-place pavement recycling remixes the in situ pavement material and reuses it in the final pavement structure. In-place recycling techniques have been successfully used by many highway agencies (*1–9*). The benefits of using in-place recycling rather than traditional techniques include reduced use of virgin materials, reduced fuel consumption, reduced lane closures, reduced emissions related to construction (*10, 11*), and large cost savings (*3*), which allow highway agencies to stretch available funding for pavement rehabilitation.

Despite the experience of many agencies, in-place pavement recycling techniques have been viewed primarily as suitable only for lower volume roadways. In addition, only a few studies have documented their performance beyond the first year of construction. The lack of reliable performance data has contributed to making in-place recycling techniques less likely to be specified by pavement engineers as the long-term performance on higher volume facilities is not widely known.

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During the 2011 construction season, the Virginia Department of Transportation (DOT) completed an in-place pavement recycling project to rehabilitate a 3.66-mi section of pavement on southbound Interstate 81 (I-81) near Staunton, Virginia. A construction contract was awarded in December 2010 with a value of $7.64 million and a time frame of approximately 8 months. The project used full-depth reclamation (FDR), cold central-plant recycling (CCPR), cold in-place recycling (CIR), and a unique traffic management plan to accomplish the work. The project marked the first time in the United States that these three recycling techniques were combined on one project on the Interstate system.

The CCPR and CIR materials were produced with 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 completed cross section is shown in Figure 1. The right lane was constructed with a combination of FDR, CCPR, and asphalt concrete (AC) overlay. The left lane was constructed with a combination of CIR and AC overlay.

Of note are the two pavement structures constructed in the right lane. The first 2,150 ft of the right lane was constructed as 4 in. of AC over 8 in. of CCPR (called a 4-over-8 structure). The rest of the right lane was constructed as 6 in. of AC over 6 in. of CCPR (called a 6-over-6 structure). The 4-over-8 structure was the original design for the right lane; the 6-over-6 structure was a design modification made just before construction. Additional details are provided by Diefenderfer et al. (*12*) and Diefenderfer and Apeagyei (*13*).

In 2008, Virginia DOT records indicated a directional traffic volume of approximately 23,000 vehicles per day with 28% trucks (*14*). Recent data from a continuous count station located approximately 5 mi downstream showed that approximately 84% of the trucks traveled in the right lane (F. Hamlin Williams, unpublished data). From these data and the average equivalent single-axle load (ESAL) values obtained from the two closest weigh in motion stations on I-81 (approximately 80 mi north and approximately 65 mi south of the project), the road segment was calculated to accumulate approximately 1.7 million and 0.3 million ESALs per year in the right and left lanes, respectively (*15*).

IN-PLACE PAVEMENT RECYCLING METHODS

In-place pavement recycling methods include FDR, cold recycling, hot in-place recycling, and cold planing. Cold recycling includes CCPR and CIR (*17*). FDR, CCPR, and CIR will be discussed in more detail here.

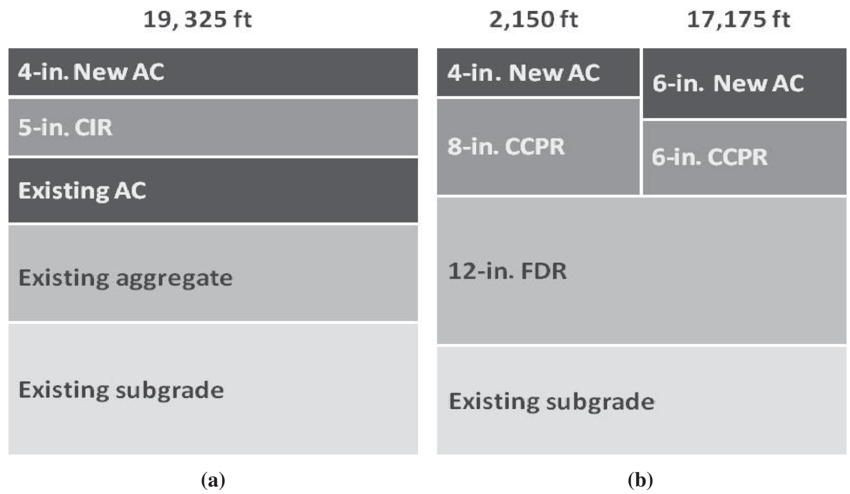

FIGURE 1 Completed cross section: (*a*) left lane and (*b*) right lane.

FDR is a process 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. (*16*). FDR is performed on the bound layers and a portion of the underlying unbound materials. FDR may involve simply pulverizing and remixing the roadway foundation, called "mechanical stabilization," but it most often includes introducing one or several stabilizing agents, called "chemical stabilization." Typical FDR stabilizing agents include active fillers (e.g., lime, fly ash, cement, and cement and lime kiln dust) and asphalt-based stabilizing agents (e.g., emulsified asphalt and foamed asphalt) (*16*). The most commonly used stabilizing agents are foamed or emulsified asphalt, lime, or cement (*17*). Active fillers may often be combined with asphalt-based stabilizers to improve resistance to the detrimental effects of moisture and to improve early strength. For higher volume routes, an AC overlay is usually applied after the FDR layer has been allowed to cure.

CIR is 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. (*16*). The CIR process is most commonly performed with a train of equipment that often includes a tanker, a cold recycler, a paver, and rollers. Typical asphalt-based recycling agents for CIR include emulsified asphalt and foamed asphalt (*16*). Active fillers, such as lime, fly ash, cement, and cement and lime kiln dust, may often be combined with asphalt-based stabilizers to improve resistance to the detrimental effects of moisture and to improve early strength (*17*). On higher volume routes, an AC overlay is typically applied, but nonstructural treatments (such as chip seals) have been used on lower volume facilities (*2, 3*).

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 twofold. First, material can be removed from the roadway and stockpiled to be used as a recycled layer, and at the same time the underlying foundation can be either stabilized or replaced if needed. Second, existing stockpiles of reclaimed asphalt pavement can be treated and used in the construction of new pavement or in the rehabilitation of existing pavement.

FUNCTIONAL PERFORMANCE AFTER CONSTRUCTION

Relatively few studies describe the long-term functional performance of pavements constructed with in-place recycled materials, which is an impediment to the increased use of these processes, as discussed by Stroup-Gardiner (*10*). These few studies pointed predominantly to the use of recycled materials on roadways carrying fewer than approximately 15,000 vehicles per day (*10*). Thus, it is possible to gain the impression that in-place pavement recycling techniques are suitable only for roadways with low to medium volumes and thereby not suitable for higher volume roadways.

In a summary of more than 10 years of performance in Ontario, Canada, hot in-place recycling and CIR were found to be effective pavement rehabilitation options with deterioration rates similar to those for conventional pavement rehabilitation practices (*18*). In a review of projects in Pennsylvania on roadways with annual average daily traffic volumes up to approximately 13,000 vehicles per day, CIR projects were found to greatly exceed their anticipated 10-year design life and provide two to three times the reflective crack resistance of conventionally resurfaced control sections (*19*). In addition, the cost of the CIR sections was one-third to two-thirds less than that of conventionally resurfaced control sections.

On the basis of 10 years of experience with CIR mixtures in Nevada, CIR was found to be an effective rehabilitation process for low- and medium-volume facilities (defined as 30 to 300 ESALs per day) (*20*). In addition, CIR was effective in reducing the development of reflective and thermal cracking and rutting. A study of 24 projects in Iowa described sites constructed between 1986 and 2004 with traffic volumes of approximately 130 to more than 5,800 vehicles per day (*21*). Stiffness [as measured by the falling weight deflectometer (FWD)] was one of the most influential parameters in the performance of CIR for the sections with higher traffic volumes.

STRUCTURAL PERFORMANCE AFTER CONSTRUCTION

As reported in the literature, the primary method used to characterize the structural performance of constructed recycled layers was the FWD. Analysis of deflection data has been used to determine the layer moduli and structural layer coefficient of the recycled layers and to monitor their strength gain with time (*1, 6, 20, 22–24*). The results from a particular project are influenced by many parameters; as such, the structural contribution measured by the FWD can vary widely. Structural layer coefficient values used in empirical-based design procedures were typically reported as ranging from approximately 0.25 to 0.35 per inch (with FDR and CIR-CCPR tending to be on the lower and upper end of this range, respectively). Layer moduli were not calculated in this study, but these data were measured from field cores and are reported elsewhere (*13, 25*).

PURPOSE AND SCOPE

The purpose of this study is to report the initial 3-year performance of the 3.66-mi. section of I-81 in Virginia that was rehabilitated in 2011 with the use of three in-place pavement recycling methods. These performance data are needed to further the use of in-place pavement recycling since there is little information in the literature documenting the structural and functional performance of pavement recycling projects on higher volume facilities. The performance was assessed in regard to rut depth, ride quality, and structural capacity as indicated by the effective structural number, pavement modulus, and structural layer coefficient.

METHODOLOGIES

Determining Rut Depth and Ride Quality

A third-party vendor simultaneously collected rutting and ride quality data 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; AASHTO R 43-07, Standard Practice for Determination of International Roughness Index (IRI) to Quantify Roughness of Pavements; and AASHTO R 48-10, Standard Practice for Determining Maximum Rut Depth in Asphalt Pavements. The data were reported from the vendor at 0.01-mi intervals. These data were collected approximately 5, 9, 12, 16, 23, 28, and 34 months after construction.

Assessing Structural Capacity

The structural capacity was assessed with deflection testing performed with an FWD in accordance with ASTM D4694-09, Standard Test Method for Deflections with a Falling-Weight-Type Impulse Load Device. Testing was conducted in the right and left lanes approximately 6, 15, and 28 months after construction. The FWD load plate was located in the right wheelpath 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 the 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* (*26*). Deflection data were analyzed with MODTAG, Version 4.1.9 (*27*). The pavement sections were characterized by evaluating the effective structural number and the pavement modulus. The in situ layer thicknesses were assessed by ground-penetrating radar (GPR) testing performed in accordance with ASTM D4748-10, Standard Test Method for Determining the Thickness of Bound Pavement Layers Using Short-Pulse Radar. In addition, the structural layer coefficients were determined for layers constructed by the three recycling processes.

RESULTS AND DISCUSSION

Rut Depth and Ride Quality

As discussed previously, 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. The data, shown in Figures 2 through 5, were collected by a device with

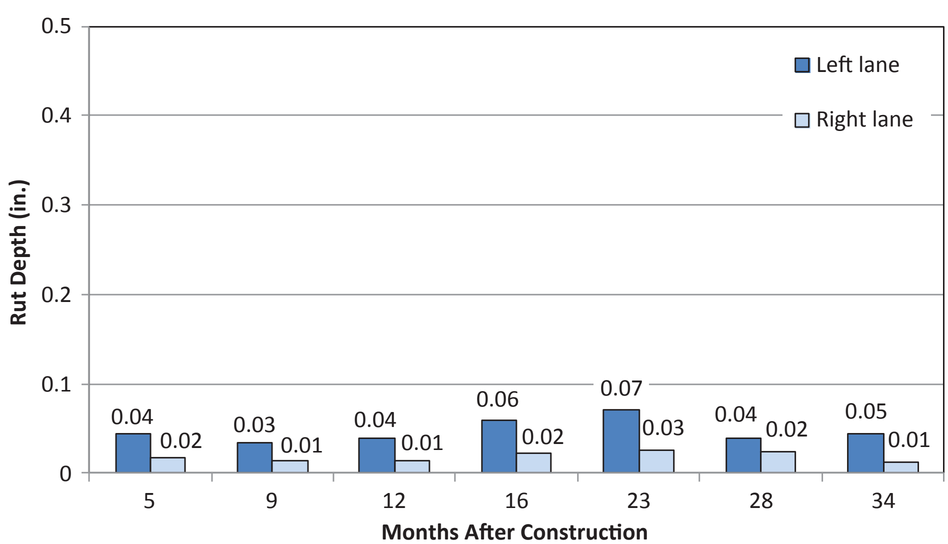

FIGURE 2 Average rut depth for left and right lanes.

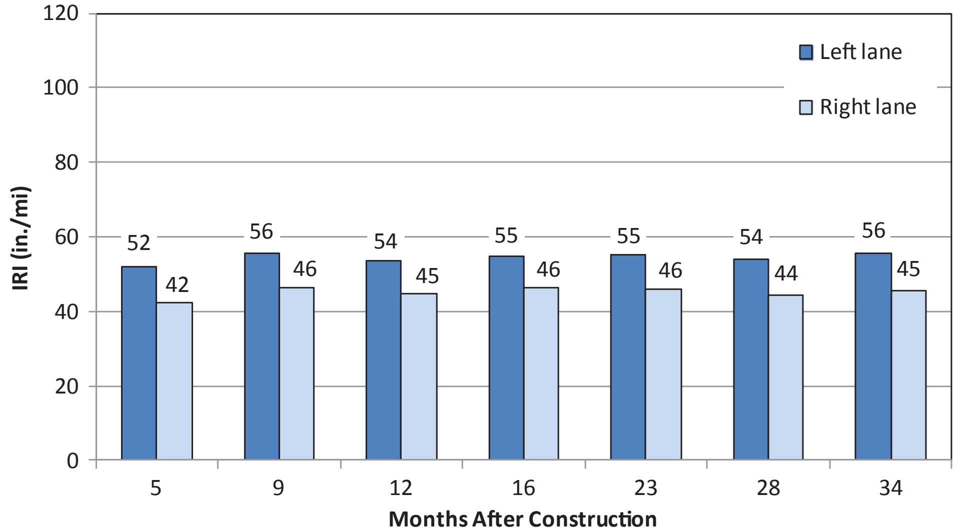

FIGURE 3 Average ride quality for left and right lanes.

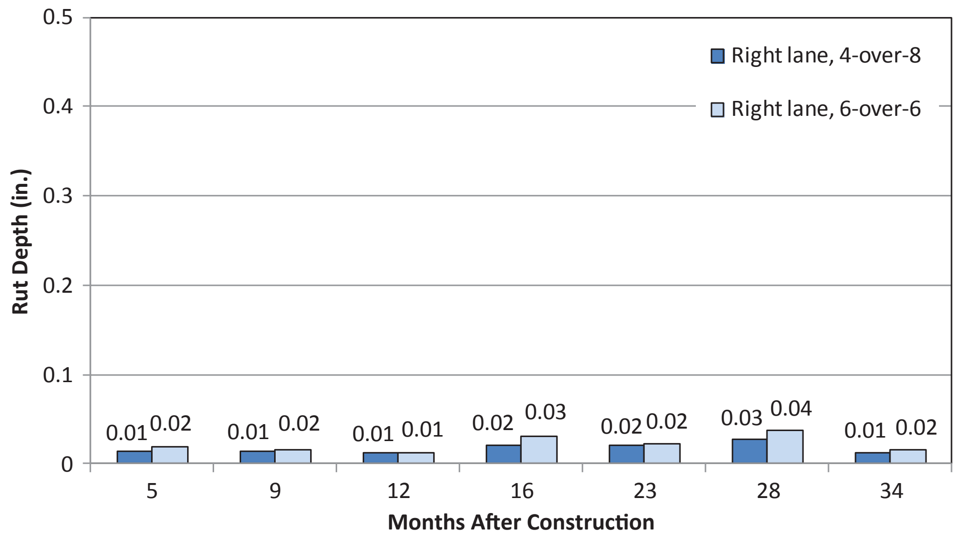

FIGURE 4 Average rut depth for 4- and 6-in. AC sections in right lane (4-over-8 = 4 in. of AC over 8 in. of CCPR and 6-over-6 = 6 in. of AC over 6 in. of CCPR).

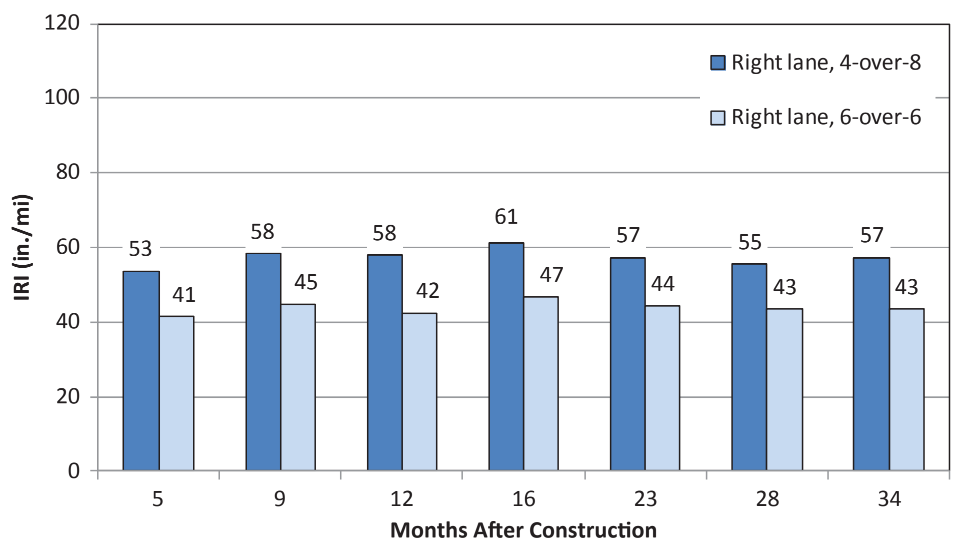

FIGURE 5 Average ride quality for 4- and 6-in. AC sections in right lane (4-over-8 = 4 in. of AC over 8 in. of CCPR and 6-over-6 = 6 in. of AC over 6 in. of CCPR).

three laser sensors. Ideally, each of the three sensors travels along the same line within the two wheelpaths and the center of the lane during successive tests. However, it is known that this is not always the case; therefore, some unquantifiable error exists in the data shown in Figures 2 through 5. The Virginia DOT considers a rut depth greater than 0.5 in. and an IRI greater than or equal to 140 in./mi to be deficient relative to rut depth and ride quality, respectively (*28*).

The average rut depth results, as seen in Figure 2, showed that the left lane had a slightly greater rut depth than the right lane; however, the rut depths were still considered negligible from a practical perspective. From Figure 2, there was little practical change in the rut depth during the first 34 months of service. Since the rut depth values were considered negligible, no statistical testing to assess differences with respect to time or lane was performed.

An examination of the ride quality relative to the average IRI for both lanes, as seen in Figure 3, showed that the left lane had a higher IRI than the right lane. Despite those differences, the Virginia DOT would still classify the pavement ride quality for both lanes as excellent (*28*). The Virginia DOT's standard specification for ride quality was applied to this project, and the contractor achieved a pay incentive for smoothness (*29*). A series of *t*-tests was performed to determine whether differences in the ride quality in each lane at 5 months and 34 months after construction were statistically significant. The differences in ride quality at 5 months and 34 months were statistically significant for the right and left lanes ($p = .006$ and .007, respectively). From Figure 3, visual interpretation showed that the IRI increased by approximately 3 to 4 in./mile for the left and right lanes from 5 to 34 months after construction.

Figures 4 and 5 show the rut depth and ride quality measurements, respectively, for the two segments in the right lane with different AC overlay thicknesses. These figures show data from the 4-over-8 section (the first approximately 2,150 ft of the right lane, constructed as 4 in. of AC over 8 in. of CCPR) and the initial portion of the 6-over-6 section (the next approximately 2,150 ft of the right lane, constructed as 6 in. of AC over 6 in. of CCPR).

Figure 4 shows that the rut depth values in the initial portion of the 6-over-6 section were slightly greater than in the 4-over-8 section; however, these differences were not considered to be practically significant. Since the rut depth values were considered negligible, no statistical testing to assess differences with respect to time or section was performed.

Figure 5 shows the IRI, a measure of pavement ride quality, with respect to age for the 4-over-8 and 6-over-6 sections. Figure 5 shows that the IRI was greater in the 4-over-8 section. However, it is not possible to determine whether this result was caused by the difference in overlay thickness or if it simply reflects that this section was the first constructed on a project with a unique and difficult construction sequence [additional details are provided by Diefenderfer et al. (*12*)]. A series of *t*-tests was performed to determine the statistical significance of differences in the ride quality in each section. The differences in ride quality of the 4-over-8 versus the initial portion of the 6-over-6 sections at 5 months and 34 months were statistically significant ($p = .002$ and .0009, respectively); that is, the difference in ride quality between the two sections was significantly different soon after construction and at nearly 3 years. However, the differences in ride quality at 5 months versus 34 months for the 4-over-8 and the initial portion of the 6-over-6 sections were not statistically significant ($p = .34$ and .37, respectively); that is, the change in ride quality soon after construction and at nearly 3 years was not statistically significant for either section.

Structural Capacity

As discussed previously, structural capacity testing with an FWD was performed approximately 6, 15, and 28 months after construction. GPR testing was used to assess the layer thickness of the various layers in each lane. For the 4-over-8 section in the right lane, the AC and CCPR layers averaged 4.8 and 8.1 in., respectively. For the 6-over-6 section in the right lane, the AC and CCPR layers averaged 7.1 and 6.4 in., respectively. The thickness of the FDR layer was not assessed with the GPR (which used a 2-GHz air horn antenna) because the bottom of the FDR layer was not identifiable in the data (the bottom of the FDR layer was approximately 24 in. from the pavement surface). Since the bottom of the FDR layer was unidentifiable, the thickness of the FDR layer was assumed to be equal to the design thickness of 12 in. For the left lane, the AC, CIR, existing AC, and aggregate layers were 4.0, 5.0, 4.3, and 6.3 in., respectively (the bottom of the existing aggregate layer was not identifiable by GPR after construction but was evident in data collected before construction). The variability in the thickness of the various layers from their design values can be attributed to profile milling and adjustments to match cross slope. The FWD and GPR testing data were used to estimate the overall structural capacity of the pavement (effective structural number and pavement modulus) and the individual pavement layer load-carrying ability (layer coefficient). As previously noted, these important pavement design parameters are often lacking for high-volume roads rehabilitated with in-place recycling techniques.

Effective Structural Number and Pavement Modulus

The results of FWD testing are shown in Figures 6 and 7. Figure 6 shows the average effective structural number, and Figure 7 shows the average pavement modulus. The data in Figures 6 and 7 show the results from the first two sections of the right lane (each 2,150 ft in length), the results for the entire right lane, and the results for the entire left lane. Figure 6 shows that the effective structural number increased from 6 to 15 months for all sections. This behavior was expected on the basis of previous experience with recycled materials (*1*). Figure 6 also shows that the effective structural number was either steady or decreased between 15 and 28 months. A similar trend for the pavement modulus is shown in Figure 7.

Structural Layer Coefficients

In addition to the data presented in Figures 6 and 7, the structural layer coefficients for the FDR, CCPR, and CIR materials were assessed from FWD data and the results of laboratory testing published elsewhere (*13*). Although the authors understand that many agencies are transferring to mechanistic-based pavement designs, they still believe there is benefit in reporting values for empirical-based designs. The topic of mechanistic design values for asphalt-based recycled materials is the subject of an ongoing national study (*30*).

AASHTO 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) (*26*). In a multilayered 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:

$$\mathrm{SN} = a_1 \times D_1 + a_2 \times D_2 + a_3 \times D_3 + \ldots \quad (1)$$

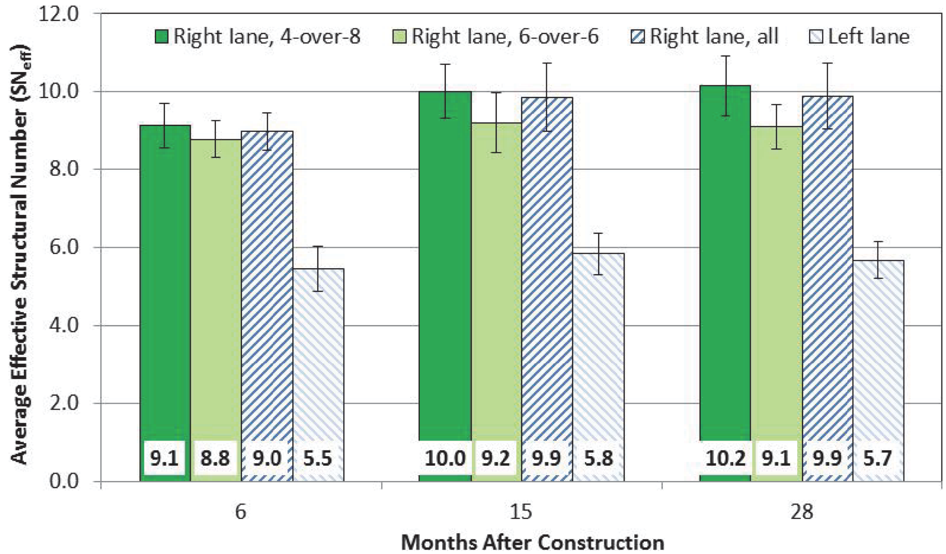

FIGURE 6 Average effective structural number for right lane (4-over-8 section, first 2,150 ft of 6-over-6 section, all) and left lane [error bars indicating ± 1 standard deviation (4-over-8 = 4 in. of AC over 8 in. of CCPR and 6-over-6 = 6 in. of AC over 6 in. of CCPR)].

The required structural number 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 that is indicative of the in situ structural capacity of an existing pavement structure.

With the most recent FWD test results shown in Figure 6, the effective structural numbers 28 months after construction for the right and left lanes were calculated as 9.89 and 5.68, respectively. With the use of the thickness data collected from the GPR survey, the pavement in the right lane was assumed to be a three-layer pavement structure consisting of AC, CCPR, and FDR, with an average thickness of 7.1, 6.4, and 12 in., respectively. With the effective structural number (SN_{eff}) of the right lane and the thickness for each layer, Equation 1 can be rewritten as follows:

$$SN_{eff(right)} = a_1 \times 7.1 + a_2 \times 6.4 + a_3 \times 12 \quad (2)$$

The Virginia DOT assumes a structural layer coefficient for new surface and intermediate AC layers (including stone-matrix asphalt) of 0.44 (*31*). 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 that a_2 equals a_3. From this, Equation 2 can be rewritten as follows:

$$9.89 = 0.44 \times 7.1 + a_2 \times 6.4 + a_2 \times 12 \quad (3)$$

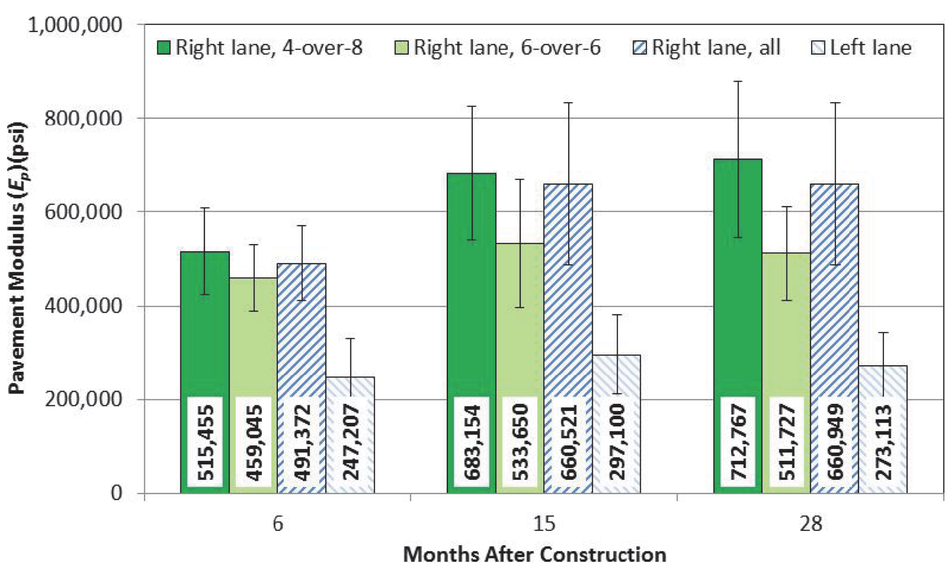

FIGURE 7 Pavement modulus of right lane (4-over-8 section, first 2,150 ft of 6-over-6 section, all) and left lane [error bars indicating ± 1 standard deviation (4-over-8 = 4 in. of AC over 8 in. of CCPR and 6-over-6 = 6 in. of AC over 6 in. of CCPR) psi = pounds per square inch].

Solving for a_2, a value of 0.37 was calculated as a combined layer coefficient for the CCPR and FDR layers. No structural number results for CCPR were found in the literature. However, by estimating a structural layer coefficient value of 0.33 for a cured cement-stabilized FDR material, a value for the CCPR material can be calculated (*24*). Substituting a value of 0.33 for a_3 in Equation 2, a_2 is calculated as 0.44. A value of 0.44 is higher than typically reported in the literature for other recycled materials. The authors believe the actual structural layer coefficient for the CCPR layer from this project was between 0.37 and 0.44.

An a_i value for the CIR materials in the left lane can also be calculated. The pavement in the left lane can be assumed to be a four-layer pavement structure consisting of AC, CIR, existing AC, and existing aggregate base. With the data collected from the GPR survey, the average thickness of the AC, CIR, existing AC, and aggregate base was calculated as 4.0, 5.0, 4.3, and 6.3 in., respectively. From those values, Equation 1 can be rewritten as follows:

$$5.68 = 0.44 \times 4.0 + a_2 \times 5.0 + a_3 \times 4.3 + a_4 \times 6.3 \tag{4}$$

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 with the FWD data from before construction. FWD testing was conducted in the right lane approximately 3 months before construction. The average effective structural number was 3.89, with a subgrade resilient modulus of 22,800 psi (*14*). Unfortunately, FWD testing before construction was not conducted in the left lane, so it was assumed that the right lane values were similar. From those values, Equation 1 can be rewritten as follows:

$$3.89 = a_1 \times 10.6 + a_2 \times 6.6 \tag{5}$$

where a_1 and a_2 are the structural layer coefficients for the existing AC and aggregate layers, respectively. AASHTO presented a relationship that can be used to estimate the structural layer coefficient for granular base layers (a_2) from the resilient modulus (*26*). That relationship is shown as follows:

$$a_2 = 0.249 \times \log_{10}(E_{BS}) - 0.977 \tag{6}$$

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 result is close to the value of 0.12 assumed by the Virginia DOT for new aggregate base materials (*31*). Substituting a value of 0.11 into Equation 5 yields a value for a_1 of 0.30. This value is taken as the in situ layer coefficient for the asphalt materials in the right and left lanes (on the basis of testing in the right lane only).

Finally, Equation 4 can be solved for a_2 by substituting the values of 0.30 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 materials could be less if the structural capacity of the existing AC materials in the left lane was not as deteriorated as the structural capacity of the AC materials in the right lane before construction. Although there are no data to prove this possibility, the authors believe it to be likely on the basis of the lane distribution of traffic loading as discussed previously.

From the FWD testing, it is seen that the combined structural layer coefficient for the FDR and CCPR layers was calculated to be 0.37. The structural layer coefficient for the CCPR layer was likely in the range of 0.37 to 0.44 depending on the method used for the calculation. The structural layer coefficient for the CIR layer was calculated to be 0.39 on the basis of the assumption that the structural capacities of the existing AC materials in the left lane are equal to those of the right lane. If the structural capacities of the AC materials in the left lane are higher, the structural layer coefficient of the CIR material could be slightly less than 0.39. These structural layer coefficient values agree with the ranges recently determined by correlations from laboratory properties for these same materials (*13*) and FWD- and laboratory-based values from other projects (*32*).

SUMMARY OF RESULTS

- The section of pavement rehabilitated by the three in-place recycling methods continues to perform well after nearly 3 years of heavy Interstate traffic. To date, approximately 6 million ESALs have been applied.
- The 4-over-8 section and the 6-over-6 section in the right lane are performing well after nearly 3 years of heavy Interstate traffic.
- Rut depth measurements showed the average rut depth to be negligible in both lanes.
- In regard to ride quality, the performance of the 4-over-8 and 6-over-6 sections in the right lane was the same and the performance of the left and right lanes was excellent (*28*).
- The effective structural number for all sections considered increased from 6 to 15 months but either remained steady or decreased slightly from 15 to 28 months.
- The structural layer coefficients for the recycled materials calculated from the FWD testing were at the upper range of values reported in the literature.

CONCLUSIONS AND RECOMMENDATIONS

- The initial performance of the I-81 pavement section rehabilitated with in-place recycling techniques can be considered excellent.
- Functional and structural monitoring of these sections should be continued to establish their long-term performance.
- Additional studies are necessary to define the required AC overlay thickness for pavements on high-volume facilities that are rehabilitated with in-place recycling techniques.

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