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# Analysis of Full-Depth Reclamation Trial Sections in Virginia

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16. Abstract:

Full-depth reclamation (FDR) has become an increasingly common technology to restore the service life of pavement structures requiring deep rehabilitation and to stretch available funding for pavement rehabilitation. FDR consists of pulverizing the existing bound flexible pavement layers along with a portion of the unbound layers (or all the unbound layers and a portion of the subgrade); adding a stabilizing agent; compacting the mixture; and surfacing with a new bound material layer(s) or surface treatment. FDR has been successfully demonstrated by many highway agencies. However, the results of structural characterization and life-cycle cost analysis (LCCA) vary greatly, making it difficult to implement the results of previous studies directly as typical values for future pavement designs.

This study assessed the condition of three FDR trial sections constructed by the Virginia Department of Transportation (VDOT) during the 2008 construction season. The sites were on SR 40 (Franklin County), SR 13 (Powhatan County), and SR 6 (Goochland County). The test site on SR 40 used asphalt emulsion and foamed asphalt binder as the respective stabilizing agents on two sections within the project. The test sites on SR 13 and SR 6 used portland cement as the stabilizing agent. Following reclamation, all three sites received a hot-mix asphalt overlay. The FDR assessment was conducted using a variety of methods that included gradation analysis; determination of resilient modulus, dynamic modulus, and indirect tensile strength; ground-penetrating radar; falling-weight deflectometer testing; and LCCA. A hypothetical LCCA was also conducted to document the potential cost savings between a pavement rehabilitation schedule that included FDR with one that was based solely on traditionally used rehabilitation techniques.

The study showed that pavements could be successfully reconstructed using FDR and that the structural capacity of FDR sections was dependent on both the stabilizing agent and time. The study recommends that VDOT pursue additional FDR projects where appropriate and work on refining a list of criteria to select future projects for those pavements where it is most suitable. Further, a project-level investigation should be performed on any potential FDR site to verify that it is an appropriate candidate. The LCCA showed that if a pavement rehabilitation strategy that includes FDR is applied to a preliminary candidate list of projects on VDOT's primary and secondary networks, a 50-year life-cycle cost savings of approximately $10 million and $30.5 million, respectively, is possible when compared to a traditionally used pavement rehabilitation approach. If the potential cost savings were annualized, the savings to VDOT would be approximately $463,000 and $1.42 million per year for the primary and secondary networks, respectively.

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**FINAL REPORT**

# ANALYSIS OF FULL-DEPTH RECLAMATION TRIAL SECTIONS IN VIRGINIA

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(A partnership of the Virginia Department of Transportation
and the University of Virginia since 1948)

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## DISCLAIMER

## ABSTRACT

Full-depth reclamation (FDR) has become an increasingly common technology to restore the service life of pavement structures requiring deep rehabilitation and to stretch available funding for pavement rehabilitation. FDR consists of pulverizing the existing bound flexible pavement layers along with a portion of the unbound layers (or all the unbound layers and a portion of the subgrade); adding a stabilizing agent; compacting the mixture; and surfacing with a new bound material layer(s) or surface treatment. FDR has been successfully demonstrated by many highway agencies. However, the results of structural characterization and life-cycle cost analysis (LCCA) vary greatly, making it difficult to implement the results of previous studies directly as typical values for future pavement designs.

This study assessed the condition of three FDR trial sections constructed by the Virginia Department of Transportation (VDOT) during the 2008 construction season. The sites were on SR 40 (Franklin County), SR 13 (Powhatan County), and SR 6 (Goochland County). The test site on SR 40 used asphalt emulsion and foamed asphalt binder as the respective stabilizing agents on two sections within the project. The test sites on SR 13 and SR 6 used portland cement as the stabilizing agent. Following reclamation, all three sites received a hot-mix asphalt overlay. The FDR assessment was conducted using a variety of methods that included gradation analysis; determination of resilient modulus, dynamic modulus, and indirect tensile strength; ground-penetrating radar; falling-weight deflectometer testing; and LCCA. A hypothetical LCCA was also conducted to document the potential cost savings between a pavement rehabilitation schedule that included FDR with one that was based solely on traditionally used rehabilitation techniques.

The study showed that pavements could be successfully reconstructed using FDR and that the structural capacity of FDR sections was dependent on both the stabilizing agent and time. The study recommends that VDOT pursue additional FDR projects where appropriate and work on refining a list of criteria to select future projects for those pavements where it is most suitable. Further, a project-level investigation should be performed on any potential FDR site to verify that it is an appropriate candidate. The LCCA showed that if a pavement rehabilitation strategy that includes FDR is applied to a preliminary candidate list of projects on VDOT's primary and secondary networks, a 50-year life-cycle cost savings of approximately $10 million and $30.5 million, respectively, is possible when compared to a traditionally used pavement rehabilitation approach. If the potential cost savings were annualized, the savings to VDOT would be approximately $463,000 and $1.42 million per year for the primary and secondary networks, respectively.

# FINAL REPORT

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## INTRODUCTION

Pavement recycling has become an increasingly common technology to restore the service life of pavement structures and to stretch available funding for pavement rehabilitation. In general, in-place pavement recycling remixes the in-situ pavement material in some form and reuses it in the final pavement. This may be performed as cold in-place recycling (CIR), hot in-place recycling (HIR), or full-depth reclamation (FDR). CIR and HIR consist of using a portion of the existing asphalt pavement as a new base or surface layer. For CIR, the existing asphalt pavement is pulverized to a specified depth (typically 3 to 6 in), may be processed in an on-board crushing unit and sized through screens, and is mixed with a cold asphalt-based stabilizing agent. The resultant material is then repaved in an unheated process. With HIR, the pavement is heated and scarified to a minimal depth (typically 2 in or less), mixed with an asphalt-based rejuvenating agent, and paved to provide a final surface or a surface upon which another layer is placed. FDR, a technique also performed in place, consists of pulverizing the existing bound layers along with a portion of the unbound layers (or all the unbound layers and a portion of the subgrade); adding a stabilizing agent (such as foamed or emulsified asphalt binder, lime, or portland cement); compacting the mixture; and surfacing with a new riding surface layer(s).

FDR has been successfully demonstrated by numerous highway agencies. In particular, the Nevada Department of Transportation has placed an emphasis on the use of both CIR and FDR for their pavement network. Bemanian et al. (2006) stated that Nevada has completed nearly 900 centerline miles of FDR since 1985 and that the use of this process has increased the load-carrying capacity and structural uniformity of their pavement system. They further stated that the use of FDR has resulted in cost savings in the hundreds of millions of dollars when compared to traditional reconstruction processes.

Others have reported their processes for determining layer coefficients and/or layer moduli of FDR materials to use FDR in empirical-based pavement design methodologies more effectively. The layer coefficient is a very important design parameter as it indicates the structural capacity of the paving materials and therefore directly affects the selection of the overlying pavement layer thickness. An accurate determination of the layer coefficient is therefore essential for an economical pavement design. However, reported results of the layer coefficient vary widely, with values ranging from 0.17 to 0.42. If an insufficient layer coefficient is used, the overlay design could run the risk of being more substantial than needed.

On the other hand, if the design layer coefficient is greater and overstates the stiffness of the recycled materials, the overlying pavement could be under-designed; leading to the potential for premature failures.

## PURPOSE AND SCOPE

The purpose of this study was to document the processes used and assess three FDR trial sections constructed by the Virginia Department of Transportation (VDOT) during its 2008 construction season and to summarize the experiences of other agencies as documented in the literature. These efforts were performed with the ultimate goal of helping VDOT determine if pursuing future FDR projects is beneficial.

## METHODS

### Overview

In 2008, VDOT completed three trial sections where flexible pavements on three primary routes were rehabilitated using FDR incorporating three stabilizing agents. The three test sites were at the following locations: State Route (SR) 40 (near Ferrum College, Franklin County), SR 13 (South of US 60, Powhatan County), and SR 6 (West of US 522, Goochland County). The test site on SR 40 used asphalt emulsion and foamed asphalt binder as the stabilizing agents on two separate sections within the project. The test sites on SR 13 and SR 6 both used portland cement as the stabilizing agent. Figure 1 shows the location of each demonstration project.

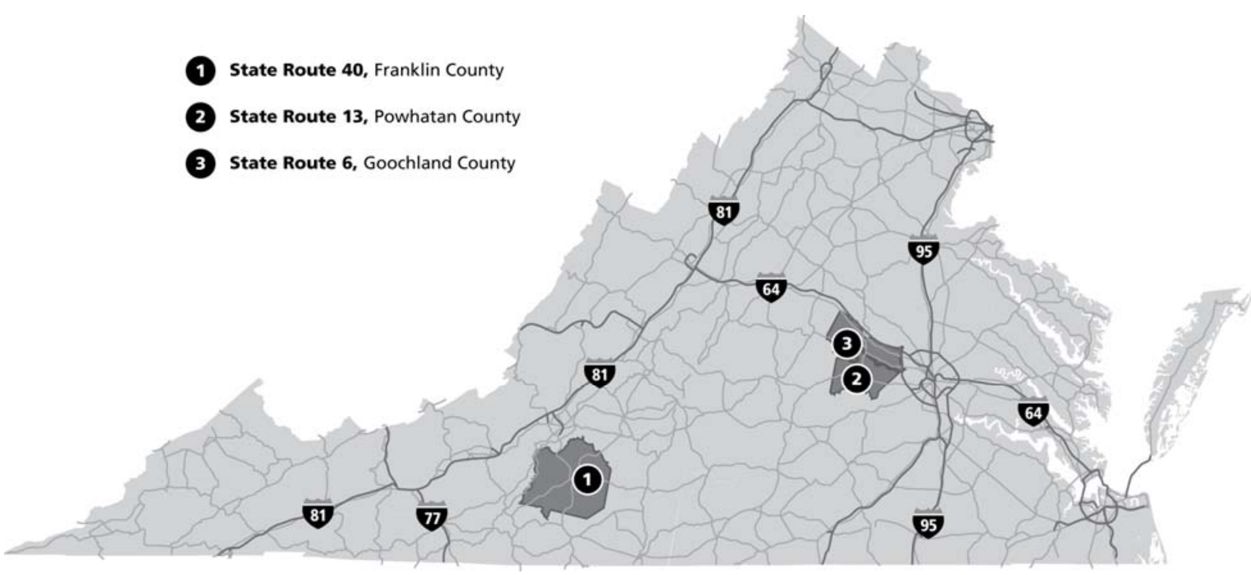



**Figure 1. Location of Full-Depth Reclamation Demonstration Projects**

Three tasks were conducted to achieve the study objectives:

1. literature review
2. laboratory evaluation
3. field evaluation.

## Literature Review

The literature review was conducted by searching various databases related to transportation engineering such as the Transportation 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.

## Laboratory Evaluation

Core samples were collected at the SR 40 project on October 28, 2008, approximately 5 months after reclamation, and again on February 7, 2011, approximately 33 months after reclamation. Eight cores were collected on each date. Cores collected in October 2007 were tested for gradation analysis, resilient modulus, and indirect tensile strength with limited dynamic modulus testing. Cores collected in February 2011 were subjected only to resilient modulus and indirect tensile strength testing. For both rounds of core collection, cores were denoted as E1 through E4 for the eastbound direction and W1 through W4 for the westbound direction. The core locations were spaced approximately evenly, with E1 collected at the western end of the project and E4 at the eastern end of the project. Conversely, W1 was collected at the eastern end of the project and W4 was collected at the western end of the project. Cores E1, E2, W3, and W4 were collected from the asphalt emulsion section. Cores E3, E4, W1, and W2 were collected from the foamed asphalt section. The same specimens with a nominal diameter of 6 in and a nominal thickness of 2 in were used for resilient modulus and indirect tensile strength testing. For the dynamic modulus testing, specimens with a diameter of 4 in and a height of 6 in were used. No cores for laboratory evaluation were collected from the SR 6 or the SR 13 projects.

## Field Evaluation

### Ground-Penetrating Radar

Ground-penetrating radar (GPR) was used to assess the layer thickness of the reclamation projects. This technique has been shown to be an effective nondestructive means for determining pavement layer thickness (Maser, 2002; Maser and Scullion, 1992).

The GPR system used in this study consisted of a 2.0 GHz air-launched horn antenna and a SIR-20 controller unit, both manufactured by Geophysical Survey Systems, Inc. (GSSI). The antenna was mounted on a survey vehicle, as shown in Figure 2. The pulse rate of the antenna

**Figure 2. VDOT's Air-Launched Ground-Penetrating Radar System**

was maintained at a rate of 1 scan per foot, 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 of each layer boundary is automatically calculated. Information from GPR testing was used in the FWD analysis.

## Falling-Weight Deflectometer

Deflection testing was performed using a Dynatest Model 8000 falling-weight deflectometer (FWD) in both directions. The FWD load plate was located in approximately the center 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 at the SR 40 site was conducted at 100-ft intervals and at four load levels (6,000; 9,000; 12,000; and 16,000 lbf). Deflection testing at SR 6 and SR 13 was conducted at 250-ft intervals and at three load levels (6,000; 9,000; and 12,000 lbf). At each load level, two deflection basins were recorded.

Deflection data analyses were conducted in accordance with the American Association of State Highway and Transportation Officials' (AASHTO) 1993 *Guide for Design of Pavement Structures* (AASHTO, 1993). Deflection data were analyzed using ModTag, Version 4.1.9 (VDOT, 2007b). The trial pavement sections were analyzed by evaluating the deflection under the load plate ($D_0$) and effective structural number ($SN_{eff}$) values and then using the $SN_{eff}$ to calculate the layer coefficient ($a_i$) for the reclaimed material. The $D_0$ value was temperature corrected using the previous day's average air temperature (average of high and low) that was obtained from a nearby weather station from the Weather Underground website (www.wunderground.com).

The pavement $SN_{eff}$ describes the structural capacity of a flexible pavement and is calculated during the pavement design process as the sum of the individual layer thicknesses multiplied by their respective empirically based layer coefficient (in accordance with the AASHTO design methodology). Based on FWD data, the effective structural number may be calculated as follows:

$$SN_{eff} = 0.0045 * D * \sqrt[3]{E_p} \qquad \text{(Eq. 1)}$$

where

$SN_{eff}$ = effective structural number
$D$ = total pavement thickness above the subgrade (in)
$E_p$ = effective pavement modulus of all layers above the subgrade (psi).
$E_p$ was calculated using the following equation:

$$d_0 = 1.5 * p * a \left[ \frac{1}{M_R \sqrt{1 + \left( \frac{D}{a} \sqrt[3]{\frac{E_p}{M_R}} \right)^2}} + \frac{\left( 1 - \frac{1}{\sqrt{1 + \left( \frac{D}{a} \right)^2}} \right)}{E_p} \right] \qquad \text{(Eq. 2)}$$

where

$d_o$ = deflection at the center of the load plate (mils)
$p$ = contact pressure (psi)
$M_R$ = subgrade resilient modulus (psi)
$D$ = total pavement thickness above the subgrade (in)
a = radius of load plate (in).

$M_R$ is a fundamental engineering material property that describes the subgrade strength and ability to resist deformation under repeated traffic loadings. $M_R$ is computed from the following:

$$M_R = C \left( \frac{P * \left( 1 - \mu^2 \right)}{\pi * r * d_r} \right) \qquad \text{(Eq. 3)}$$

where

$M_R$ = subgrade resilient modulus (psi)

$P$ = applied load (lb)
$\mu$ = Poisson's ratio
$r$ = radial distance at which the deflection is measured (in)
$d_r$ = measured deflection at a radial distance, r (mils)
$C$ = correction factor, 0.33.

Deflection data for the SR 40 site were analyzed as two separate pavement structures composed of the foamed asphalt and asphalt emulsion sections. Deflection data for all pavement sites were analyzed with respect to direction in case there was any variation. GPR testing and core sampling were used to estimate the thickness of the pavement layers for the calculations. Coring, performed on the SR 40 site in October 2008 and on the SR 13 site during construction, confirmed the pavement thickness. No cores were collected on the SR 6 site.

## Condition Assessment

Through a contract with a third-party vendor, VDOT collects condition data on the pavement network annually by using continuous digital imaging and automated crack detection technology (VDOT, 2010). In addition, roughness and rutting data are simultaneously collected with vehicle-mounted sensors. The data are analyzed to quantify the pavement network condition and the condition assessment is performed on the entire interstate and primary networks and approximately 20% of the secondary network each year. Through this process, the condition of the three trial sections was assessed between January and May 2009 and between March and April 2010, approximately 8 and 22 months after construction, respectively.

An index calculation methodology is employed to quantify the distresses observed in terms of a critical condition index (CCI). The CCI 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. The condition of each homogeneous pavement section is described in accordance with the rating scheme shown in Table 1. The condition distribution of pavements on the primary network is also shown in Table 1. VDOT's primary network consists of 21,642 lane-miles of pavement, of which 20,344 lane-miles (94.0%) is flexible pavement. The flexible pavement condition distribution for the primary network is similar to the pavement condition distribution for all pavement types.

**Table 1. Pavement Condition Category and Distribution Based on Critical Condition Index**

| Pavement Condition | Index Scale | % Primary Network | Lane-Miles |
|---|---|---|---|
| Excellent | >90 | 33.3 | 7,206 |
| Good | 70-89 | 33.6 | 7,271 |
| Fair | 60-69 | 8.8 | 1,904 |
| Poor | 50-59 | 10.7 | 2,315 |
| Very Poor | <50 | 13.6 | 2,943 |

*Source:* Virginia Department of Transportation. (2010). *State of the Pavement Report: 2009.* Maintenance Division, Richmond.

### Life-Cycle Cost Analysis

A life-cycle cost analysis (LCCA) was performed to explore the potential for cost savings if VDOT were to implement an FDR program for flexible pavements within the primary and secondary network. The LCCA, based on a present cost methodology, compared a traditional pavement rehabilitation program (based on partial- and full-depth mill and replacement) with one that incorporated FDR. The treatments for the traditional approach were based on a combination of VDOT's LCCA documentation (VDOT, 2008), which considers a 50-year analysis period, historical information regarding typical service lives, and information from the literature.

## RESULTS AND DISCUSSION

### Literature Review

Recycling pavement materials in place has been ongoing to varying degrees since the early 1900s. However, technological advances in the 1970s made these techniques more practical. Typical roadway geometry in areas having long straight stretches of pavement led to in-place recycling becoming more prevalent in certain areas of the United States (Asphalt Recycling and Reclaiming Association [ARRA], 2001). More recently, the Federal Highway Administration (FHWA) established a policy that supports the use of recycling nationwide and has the following directives (FHWA, 2002):

- Recycling and reuse can offer engineering, economic, and environmental benefits.
- Recycled materials should get first consideration in materials selection.
- Determination of the use of recycled materials should include an initial review of engineering and environmental suitability. An assessment of economic benefits should follow in the selection process. Restrictions that prohibit the use of recycled materials without a technical basis should be removed from specifications.

As discussed previously, FDR is a commonly used method of in-place recycling. FDR consists of pulverizing the existing bound layers along with all or a portion of the unbound layers and/or subgrade (one primary difference between FDR and HIR and CIR recycling is that HIR and CIR are performed only within the existing bound layers). After pulverization, a stabilizing agent (such as foamed or emulsified asphalt, lime, or portland cement) is added, the mixture is compacted, and a surface is added. For higher volume routes, a new HMA layer(s) may be applied in lieu of a chip seal (or similar) that may be more common on lower volume routes. FDR is used to correct severe structural deficiencies and defects that are deep within the bound pavement structure. The depth of pulverization depends upon the thickness of the bound layers of the existing pavement but is typically 4 to 12 in (ARRA, 2001). A self-propelled reclaimer is typical of modern equipment and offers the ability to meter dosages of the stabilizing agent precisely based on the forward speed of the machine. The reclaimer features a milling drum that rotates against the direction of travel (in an up-cut direction) and is housed within a mixing chamber where the stabilizing agent is introduced. The particle size of the reclaimed material is controlled by the forward speed of the reclaimer and the rotational speed of the milling drum among other factors (ARRA, 2001).

A stabilizing agent is used during the FDR process to bind the pulverized particles together following compaction. In this way the structural properties of the materials are enhanced, and, in addition, the resistance to the detrimental effects of moisture is improved. The most commonly used stabilizing agents are foamed or emulsified asphalt binder, lime, or portland cement. Wirtgen GmbH (2004) stated that the selection of stabilizing agent is driven by the following factors: price, availability, effectiveness, and policy. Price and availability may be somewhat interrelated in that if a stabilizing agent is not readily available locally, it may be very expensive to use. The effectiveness of the various stabilizing agents should be considered as certain agents are more effective in certain applications. Thomas (2010) provided a set of guidelines as to when each stabilizing agent is most effective, as shown in Table A1, Appendix A. In general, bituminous stabilizing agents are recommended when the pulverized materials are classified as consisting of larger particle sizes, generally ranging from gravel to sand-grain-size classifications. Cementitious stabilizing agents are recommended for the entire range of the AASHTO Classification System (Thomas, 2010) with cement recommended when the pulverized materials are classified as consisting of sand grain sizes and larger and lime when the pulverized materials are classified as consisting of silt and clay grain sizes. Because of interactions with clay particles, lime is preferred in those cases where the plasticity index is greater than 10 (Wirtgen GmbH, 2004).

The dosage rate for each stabilizing agent is determined by a process of mixing samples having various dosage rates in the laboratory and selecting the dosage rate that gives the optimum performance (usually in terms of an unconfined compressive strength, retained indirect tensile strength following moisture conditioning, and/or stability value). In part because of its familiarity in other areas of civil engineering, cementitious materials (cement, lime, fly ash, etc.) are perhaps the most popular stabilizing agents. These materials tend to be somewhat rigid, with the potential for shrinkage cracking being a concern at higher dosage rates. (Wirtgen GmbH [2004] suggested a maximum unconfined compressive strength to minimize the potential for shrinkage cracking.) The result of asphalt-based (foamed asphalt and asphalt emulsion) FDR can be described as a granular material having a binder-rich mastic with void contents often greater than 10% (Wirtgen GmbH, 2004). These materials tend to be less rigid but appear to offer a benefit in terms of repeated loading performance. The overall performance of asphalt-based FDR tends to combine the performance of a granular material (high inter-particle friction showing stress dependency) with that of a visco-elastic material (capable of withstanding repeated loading). FDR projects where foamed asphalt is used as the stabilizing agent often also employ small amounts of cement or hydrated lime (0.5% to 1.5% by mass) to improve early-term strength, increase the fines content, and reduce moisture sensitivity (Fu et al., 2010; Jones et al., 2008; Wirtgen GmbH, 2004). Jones et al. (2008) and Fu et al. (2010) provided additional details about the interparticle bonding with various stabilizing agents.

Curing is an important factor that occurs in all FDR mixtures. Curing is essential for maximum strength gain. Fu et al. (2010) defined *curing* as the process in which a material develops strength and stiffness with time. For FDR using foamed asphalt plus portland cement, Jones et al. (2008) reported that the curing mechanism is related to evaporation of water at the mastic aggregate interface. The study noted that curing in FDR using foamed asphalt plus portland cement is similar to curing in regular cementitious mixtures. For asphalt emulsions, curing occurs when asphalt breaks from solution and bonds to the coarse aggregates. Kroge et al. (2009) stated that a cured FDR layer is indicated by a moisture content of 3% or less. For laboratory testing, many modified curing procedures have been used to assess the structural properties of FDR mixtures (Fu et al., 2010; Guthrie et al., 2007; Jones et al., 2008; Lewis et al., 2006; Loizos, 2007; Mallick et al., 2002a–c; Marquis et al., 2002; Miller et al., 2006; Nataatmadja, 2001; Thompson et al., 2009).

FDR has been successfully demonstrated by numerous highway agencies in several states and countries including Georgia (Kroge et al., 2009; Lewis et al., 2006); Kansas (Romanoschi et al., 2004); Louisiana (Mohammad et al., 2003); Maine (Mallick et al., 2002b,c); Nevada (Bemanian et al., 2006; Maurer et al., 2007); Texas (Hilbrich and Scullion, 2008); Utah (Guthrie et al., 2007); Wisconsin (Wen et al., 2004); Saskatchewan (Berthelot et al., 2007); and New Zealand (Saleh, 2004). Despite the adoption by numerous agencies, there is no widely accepted approach used to describe the structural capacity of FDR materials for design purposes. As the majority of highway agencies are still using empirical-based pavement design methodologies (ARA Inc., 2004), much of the work related to quantifying the structural capacity of FDR has been performed with respect to determining an appropriate layer coefficient of the material. Ranges of structural layer coefficient are shown in Table 2. The ranges of layer coefficients are likely due to differences in materials, proportions of bound versus unbound materials in the reclaimed mixture, and stabilizing agent type and dosage rate.

**Table 2. Summary of Published Structural Layer Coefficients for Full-Depth Reclamation**

| Reference | Stabilizing Agent | Layer Coefficient |
|---|---|---|
| Romanoschi et al. (2004) | Foamed asphalt | 0.18 |
| Bemanian et al. (2006) | Foamed asphalt | 0.18 |
| Marquis et al. (2002) | Foamed asphalt | 0.22-0.35 |
| Dai et al. (2008) | Foamed asphalt | 0.20-0.42 |
| | Asphalt emulsion | 0.17-0.41 |
| Wen et al. (2004) | Fly ash | 0.23 |

## Description of Project Test Sites

As described previously, in 2008, VDOT completed three trial sections where flexible pavements on three primary routes were rehabilitated using FDR incorporating three stabilizing agents. The test sites were on SR 40 (near Ferrum College, Franklin County), SR 13 (south of Route 60, Powhatan County), and SR 6 (west of SR 522, Goochland County). For the test site on SR 40, asphalt emulsion and foamed asphalt were used as the stabilizing agents on two sections within the project. For the test sites on SR 13 and SR 6, portland cement was used as the stabilizing agent. A summary is provided in Table 3.

**Table 3. Summary of Virginia's Full-Depth Reclamation Demonstration Projects**

| Project | County | AADT[a] (% trucks) | Approximate Project Length, Lane-Miles | Stabilizing Agent | Stabilizing Agent Content |
|---|---|---|---|---|---|
| State Route 40 | Franklin | 4,400 (4%) | 0.5 | Foamed asphalt (PG 64-22) | 2.7% (+1.0% portland cement) |
| | | | 0.6 | Asphalt emulsion (PG 58-22) | 2.28%[b] |
| State Route 13 | Powhatan | 2,300 (5%) | 7.3 | Portland cement | 5.0% |
| State Route 6 | Goochland | 3,900 (7%) | 7.2 | Portland cement | 5.0% |

AADT = annual average daily traffic; PG = performance grade.
[a] Source: Virginia Department of Transportation. (2007). *Average Daily Traffic Volumes with Vehicle Classification Data on Interstate, Arterial and Primary Routes.* Richmond. http://www.virginiadot.org/info/ct-TrafficCounts.asp. Accessed August 5, 2008.
[b] The emulsion content (including newly added asphalt binder and water) was 3.5%.

## State Route 40, Franklin County

SR 40 is an east-west facility that runs through southern central Virginia approximately 35 miles south of Roanoke. This section has a combined annual average daily traffic (AADT) of approximately 4,400 (4% trucks) (VDOT, 2007a). The original pavement consisted of 5 to 6 in of HMA and surface treatments over 6 to 10 in of aggregate materials (consisting of crushed aggregate and uncrushed gravel). Prior to reclamation, the original pavement showed numerous structural distresses including longitudinal cracking and fatigue cracking within the wheel paths. Cores collected during the pre-reclamation site investigation showed multiple thin layers with some layers being completely debonded.

The pavement was reclaimed to a depth of approximately 8 to 10 in, and a 1.5-in HMA overlay (following a leveling course) was placed as a wearing surface. Two treatments were applied at this site. The western half of the project (both lanes) was reclaimed using an asphalt emulsion, and the eastern half of the project (both lanes) was reclaimed using foamed asphalt as the stabilizing agent. The binder used during the foaming process was a performance grade (PG) 64-22 binder, and the binder used in the emulsion process was a PG 58-22.

The reclaimer made two passes: the first pulverized the existing pavement, and the second added the stabilizing agent. Figure 3 shows an example of the reclaimer adding the asphalt emulsion during the second pass. Once the stabilizing agent was incorporated, the resulting material was shaped by a motor grader and compacted by pad foot, steel wheeled, and rubber tire rollers. The original pavement lanes were approximately 10 ft wide, and the reclamation process also allowed for an increase in width of 2 ft/lane. The stabilizing agent content was 2.7% and 2.28% for the foamed asphalt and asphalt emulsion, respectively. In addition, approximately 1.0% portland cement was added within the foamed asphalt section. The reclamation process on the foamed asphalt portion occurred May 13 through 15, 2008, and on the asphalt emulsion portion from May 19 through 22, 2008. The HMA overlay was placed approximately 3 weeks later.

**Figure 3. Reclamation Process, State Route 40**

### State Route 13, Powhatan County

SR 13 is an east-west facility located approximately 25 miles west of Richmond. This section has a combined AADT of approximately 2,300 (5% trucks) (VDOT, 2007a). The original pavement consisted of 4.5 to 6 in of HMA over an aggregate base. Prior to reclamation, widespread longitudinal cracking and minor fatigue cracking were evident in the wheel paths. Cores collected during the pre-reclamation site investigation showed debonded and stripped layers.

The pavement was reclaimed to a depth of 8 to 10 in, and a 3.75-in HMA overlay was placed as a wearing surface. The reclamation was performed using portland cement as the stabilizing agent at a content of approximately 5.0%. The reclaimer made two passes: the first pulverized the existing pavement and the second added the stabilizing agent. The resulting material was shaped by a motor grader and compacted by pad foot, steel wheeled, and rubber tire rollers. The reclamation process occurred between June 18 and July 21, 2008; the HMA overlay was added approximately 1 week after the reclamation was completed.

### State Route 6, Goochland County

SR 6 is an east-west facility located approximately 25 miles west of Richmond. This section has a combined AADT of approximately 3,900 (7% trucks) (VDOT, 2007a). The original pavement consisted of 8 to 9 in of HMA over an aggregate base. The pavement surface exhibited widespread longitudinal cracking and periodic fatigue cracking in the wheel paths. Cores collected during the pre-reclamation site investigation showed debonded and stripped layers.

The top 2 in of the pavement was milled, and the remaining pavement was reclaimed to a depth of 8 to 10 in. A 3.5-in HMA overlay was placed as a wearing surface. The reclamation was performed using portland cement as the stabilizing agent at a content of approximately 5.0%. The reclaimer made two passes: the first pulverized the existing pavement and the second added the stabilizing agent. The resulting material was shaped by a motor grader and compacted by pad foot, steel wheeled, and rubber tire rollers. The reclamation process occurred between July 21 and August 7, 2008; the HMA overlay was added approximately 1 week after the reclamation was completed.

## Laboratory Evaluation

### State Route 40, 5-Month Evaluation (October 2007)

Eight cores (four each from the asphalt emulsion section and foamed asphalt section) were collected from the Route 40 site in October 2007. Results of the gradation and asphalt content analysis are provided in Table B1, Appendix B. Three of the four cores tested were thick enough to be cut in half; the results are reported as *upper* and *lower*. In general, the percentage passing the No. 200 sieve ranged from 8.0% to 11.5%, and that passing the 1-in sieve ranged from 93% to 100%. The asphalt content ranged from 7.2% to 9.3% and reflected the original plus added binder in the reclaimed mixture.

The average gradations of the specimens from the foam section and the emulsion section are shown in Figure 4. These two gradation curves are also presented with two control lines (shown as dashed lines in Figure 4) that bound an area of “suitable” gradation for foamed asphalt reclamation as recommended by Wirtgen GmbH (2004). The materials from the SR 40 project had a higher percentage passing the larger sieve sizes than that suggested by the control lines; however, the most critical portion of the gradation lies among the smallest particle sizes. Insufficient fines will not allow for a proper dispersion of the foamed asphalt particles and results in poor adhesion of the recycled particles.

The results of the resilient modulus testing at 20°C are shown in Table B2, Appendix B. The average resilient modulus of the asphalt emulsion was approximately 499,000 psi with a range of approximately 413,000 to 661,000 psi. The average resilient modulus of the foamed asphalt was approximately 527,000 psi with a range of approximately 398,000 to 632,000 psi. A $t$-test, assuming unequal variances, indicated that the differences between stabilizing agents were not statistically significant ($p = 0.78$).

The results of the indirect tensile strength tests (conducted at 25°C) are also shown in Table B2, Appendix B. The average indirect tensile strength of the asphalt emulsion was approximately 109 psi with a range of 90 to 130 psi. The average indirect tensile strength of the foamed asphalt was approximately 104 psi with a range of 73 to 133 psi. A $t$-test, assuming unequal variances, indicated that the differences between stabilizing agents were not statistically significant ($p = 0.80$).

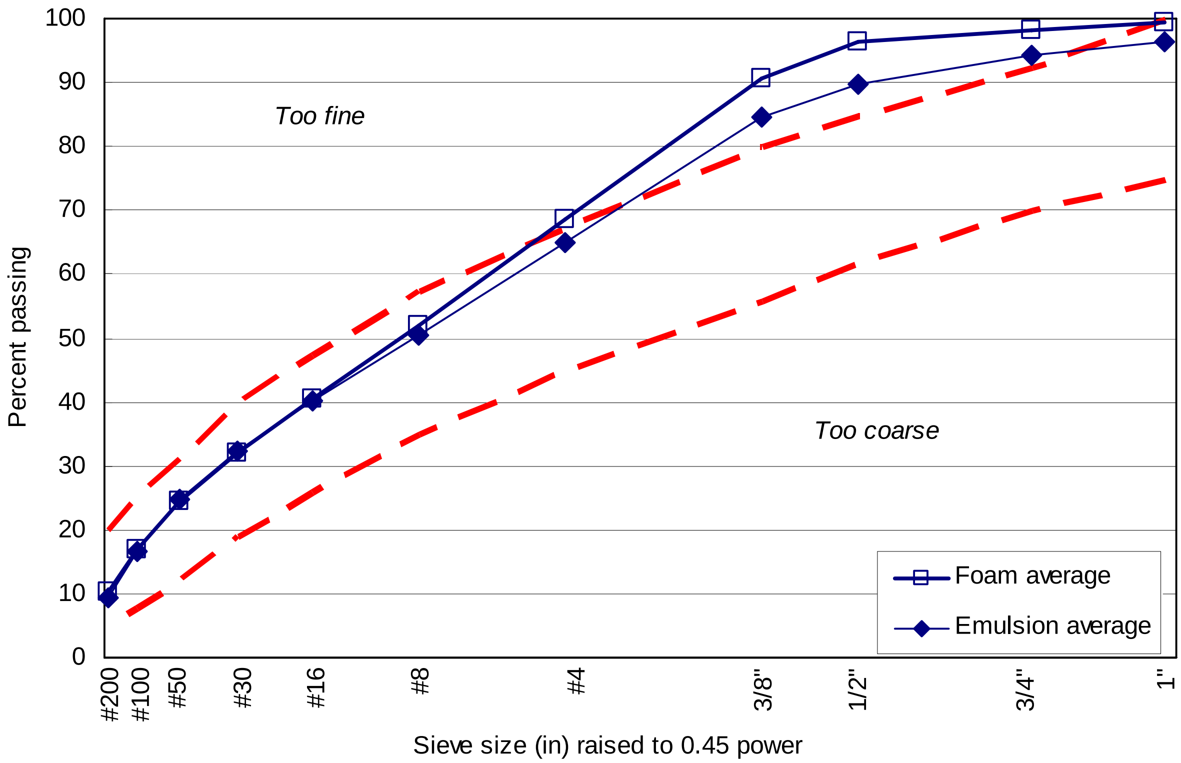



**Figure 4. Average Gradation, State Route 40**

Table B3, Appendix B, shows the results of limited dynamic modulus testing. The testing was performed on one specimen each for the asphalt emulsion and the foamed asphalt stabilizing agents. Testing was performed over eight frequencies, ranging from 0.1 to 20 Hz, at each of two temperatures (4°C and 20°C). In general, the sample having foamed asphalt as the stabilizing agent had a higher dynamic modulus value at all temperatures and frequencies tested. The differences ranged from 0% to 11% at the 4°C test and from 35% to 54% at the 20°C test. It is interesting to note that at 4°C the differences increased with increasing frequency but the opposite trend occurred at 20°C. The higher dynamic modulus at the higher temperatures could be expected as Table 3 shows that the asphalt binder used in the foamed asphalt section was a PG 64-22 as opposed to the PG 58-22 used in the asphalt emulsion section.

## State Route 40, 33-Month Evaluation (February 2011)

Eight cores (four each from the asphalt emulsion section and foamed asphalt section) were collected from the Route 40 site in February 2011. The cores were collected near the same locations where the 5-month cores were collected. All cores were to be collected as 6-in-diameter cores, but unexpectedly, cores W1, W2, and W3 were collected as 4-in-diameter cores. Because of the smaller diameter, cores W1, W2, and W3 were not tested. The remaining cores were cut into approximately 2-in-tall slices with one slice coming from the upper half (denoted *upper*) and one slice coming from the bottom half (denoted *lower*) for resilient modulus and indirect tensile strength testing.

The results of the resilient modulus testing at 4°C, 20°C, and 38°C are shown in Table B4, Appendix B. The average resilient modulus of the asphalt emulsion was approximately

1,005,800; 619,100; and 399,100 psi at test temperatures of 4°C, 20°C, and 38°C, respectively. The average resilient modulus of the foamed asphalt was approximately 1,123,700; 673,400; and 331,600 psi at test temperatures of 4°C, 20°C, and 38°C, respectively. Table B4, Appendix B, shows a wide range of resilient modulus values. A *t*-test, assuming unequal variances, indicated that the differences in the two stabilizing agents at each test temperature were not statistically significant ($p$ = 0.59, 0.82, and 0.61, respectively). By comparison with the cores collected at 5 months, the average resilient modulus (at a test temperature of 20°C) increased slightly; however, a *t*-test, assuming unequal variances, indicated that the differences across both stabilizing agents were not statistically significant ($p$ = 0.36 and 0.55 for asphalt emulsion and foamed asphalt, respectively).

The results of the indirect tensile strength tests (conducted at 25°C) are also shown in Table B4, Appendix B. The average indirect tensile strength of the asphalt emulsion was approximately 70 psi with a range of 41 to 95 psi. The average indirect tensile strength of the foamed asphalt was approximately 81 psi with a range of 70 to 87 psi. A *t*-test, assuming unequal variances, indicated that the differences in the two samples were not statistically significant ($p$ = 0.37). By comparison with the cores collected at 5 months, the average indirect tensile strength decreased slightly; however, the range of individual test results was similar. A *t*-test, assuming unequal variances, indicated that the differences between the indirect tensile strengths at 5 months versus 33 months were statistically significant for the asphalt emulsion specimens but not for the foamed asphalt specimens ($p$ = 0.05 and 0.25, respectively). Indirect tensile strength would be expected to decrease over time if moisture damage is occurring. This may be the case for the asphalt emulsion specimens.

## Field Evaluation

### Ground-Penetrating Radar

GPR testing was conducted approximately 6 weeks after construction at all three project sites. Figure 5 shows an example of the GPR data output where the layer interfaces can be seen as a series of horizontal bands on the grayscale image. The figure also shows the difference in structural uniformity between the original pavement (left) and the reclaimed pavement structure (right). The results of the GPR testing were analyzed to determine the layer thicknesses for use with FWD analysis. Table 4 shows the results of the average layer thickness for each project in terms of an idealized three-layer pavement structure.

**Table 4. Average Layer Thickness (in) for Full-Depth Reclamation Demonstration Projects**

| | Route 40 | | | | Route 13 | | Route 6 | |
|---|---|---|---|---|---|---|---|---|
| | EB | WB | EB | WB | EB | WB | EB | WB |
| Material | Foamed asphalt | | Asphalt emulsion | | Portland cement | | Portland cement | |
| HMA | 2.5 | 2.2 | 2.2 | 2.2 | 3.3 | 3.9 | 3.4 | 3.8 |
| Reclaimed layer | 10.4 | 8.3 | 9.1 | 10.5 | 9.1 | 9.3 | 10.1 | 8.1 |
| Aggregate layer | 3.8 | 3.6 | 3.0 | 2.5 | 2.5 | 2.4 | 1.8 | 2.0 |

EB = eastbound; WB = westbound.

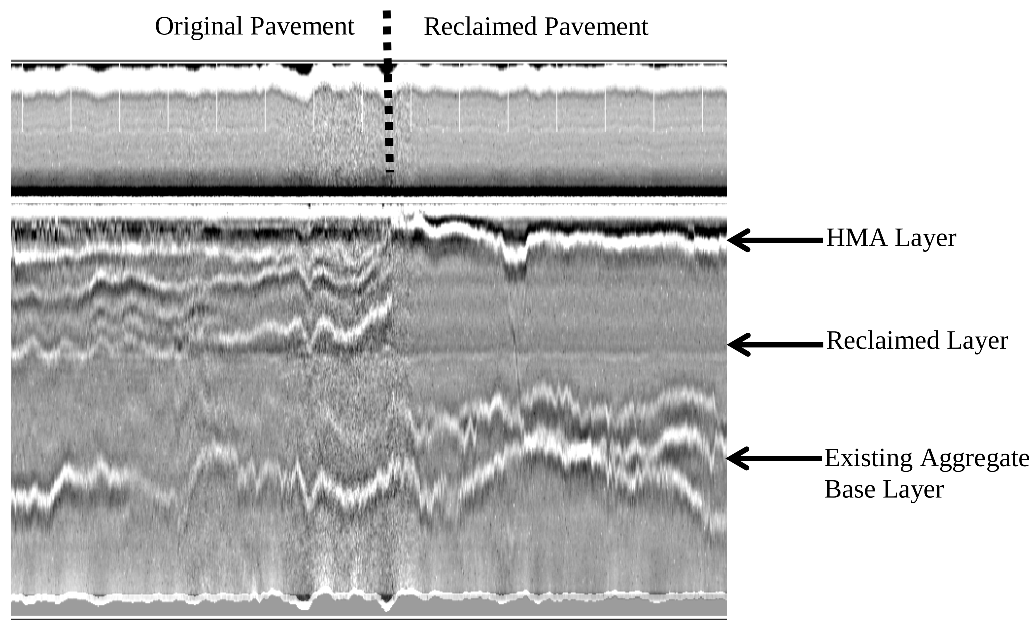



**Figure 5. State Route 40 Ground-Penetrating Radar Results. HMA = hot-mix asphalt.**

## Falling-Weight Deflectometer

### *State Route 40*

Figure 6 shows the results of the FWD testing in terms of the $M_R$ for SR 40. The $M_R$ was generally higher within the foamed asphalt section in both directions and ranged from approximately 16,000 to more than 33,000 psi. Over the first 6 months, the $M_R$ generally increased with time. In addition, seasonal trends were observed from approximately month 8 to

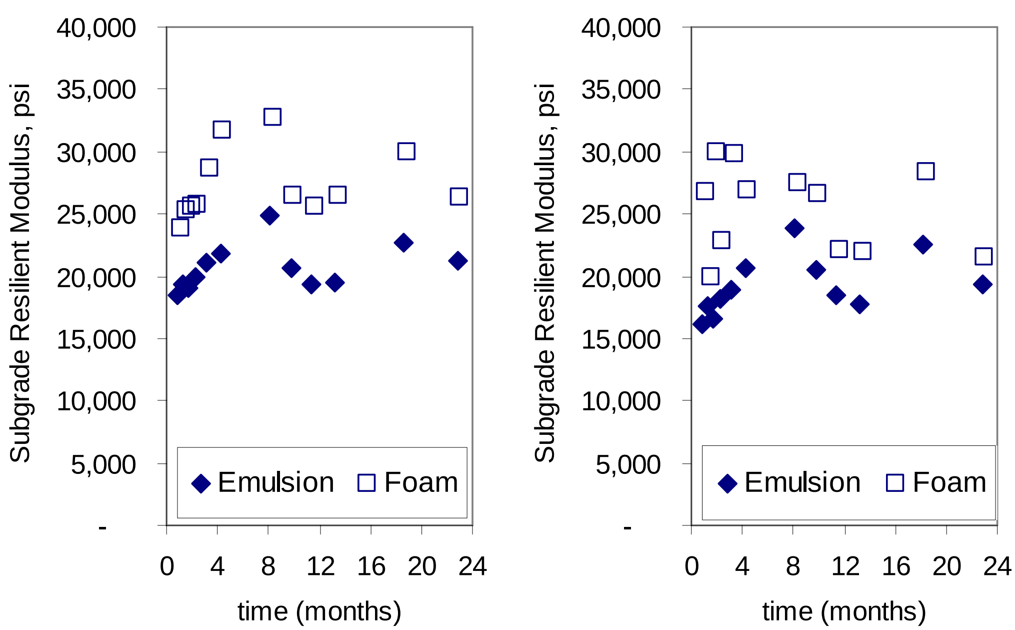
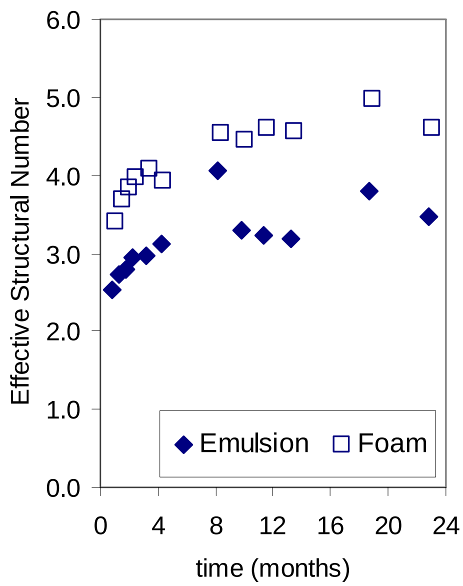



**Figure 6. Subgrade Resilient Modulus, State Route 40: eastbound *(left)*, westbound *(right)***

13 (January 2009 to June 2009) and again from month 19 to 24 (January 2010 to May 2010). These trends showed a reduction in the $M_R$ over the time period when local precipitation is expected to be highest.

Figure 7 shows the results of the FWD testing in terms of the $SN_{eff}$ for SR 40. The figure shows data collected from approximately 3 weeks until approximately 24 months after reclamation. The $SN_{eff}$ for the eastbound direction ranged from approximately 2.5 to 3.8 within the asphalt emulsion section and from approximately 3.4 to 5.0 within the foamed asphalt section. The $SN_{eff}$ for the westbound direction ranged from approximately 2.5 to 3.7 within the asphalt emulsion section and from approximately 3.1 to 4.0 within the foamed asphalt section. A general increasing trend was seen for both directions and with both stabilizing agents, but a seasonal component similar to that discussed for the $M_R$ was seen (especially for the asphalt emulsion sections). A similar seasonal trend was described by Jones et al. (2008) when FWD testing was conducted on FDR sections using foamed asphalt in California.



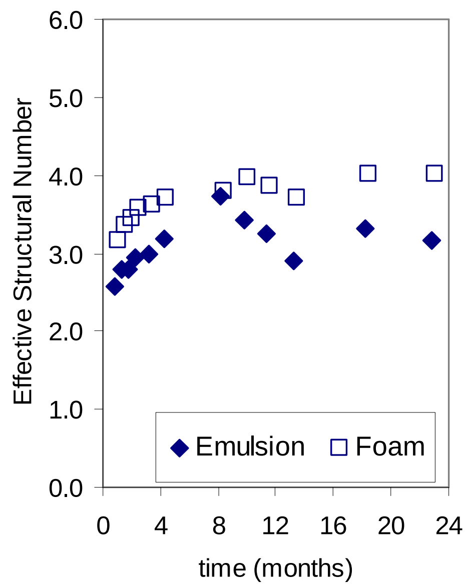




**Figure 7. Effective Structural Number, State Route 40: eastbound *(left)*, westbound *(right)***

*State Route 6 and State Route 13*

Figure 8 shows the results of the FWD testing in terms of the $M_R$ for SR 13 and SR 6. The $M_R$ was generally higher for the SR 13 project and ranged from approximately 19,000 to 27,000 psi for the SR 13 project and from 13,000 to 22,000 psi for the SR 6 project. The $M_R$ fluctuated with time, but a seasonal trend was more evident for the SR 6 project from month 6 to 11 (February 2009 to June 1009) and again from month 16 to 21 (December 2009 to April 2010).

Figure 9 shows the results of the FWD testing in terms of the $SN_{eff}$ for SR 13 and SR 6. The figure shows data collected from approximately 4 weeks until approximately 21 months after reclamation. A general increasing trend was seen for both sites with less of the seasonal variability observed for the $M_R$ results or the SR 40 results where asphalt-based stabilizing agents were used. The $SN_{eff}$ ranged from approximately 4.0 to 5.0 for both projects.

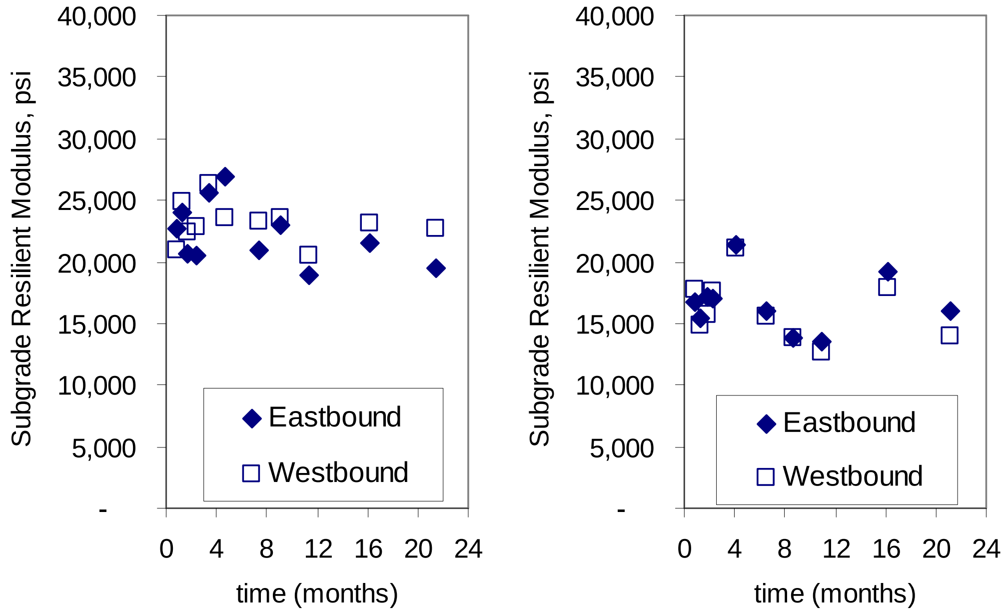



**Figure 8. Subgrade Resilient Modulus for State Route 13 *(left)* and State Route 6 *(right)***

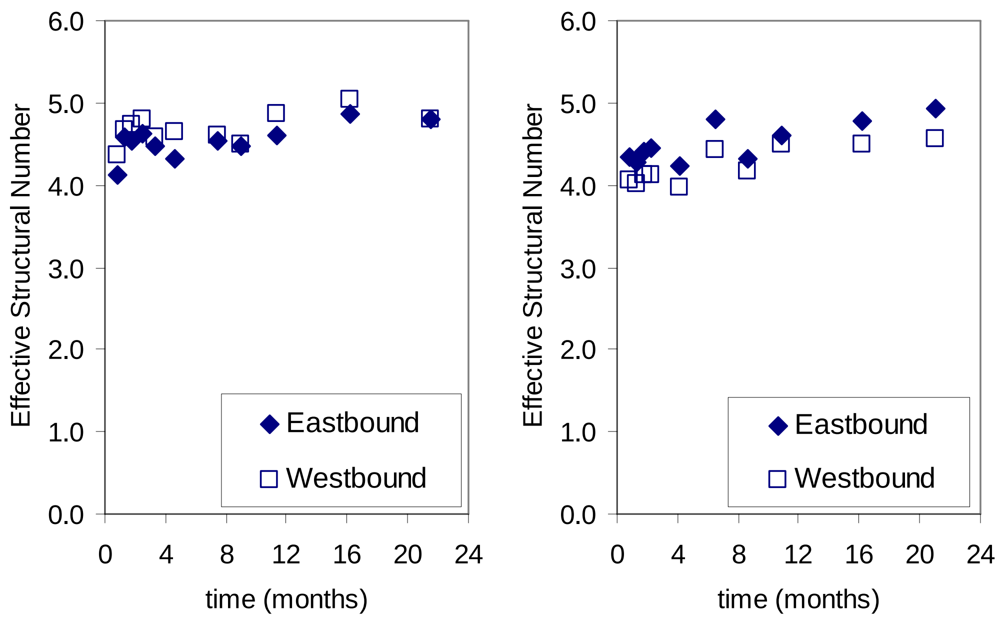



**Figure 9. Effective Structural Number for State Route 13 *(left)* and State Route 6 *(right)***

## Layer Coefficient Calculations

Given the wide range of structural layer coefficients for FDR materials reported elsewhere (shown in Table 2), this study sought to develop representative values for future FDR pavement designs in Virginia in accordance with the procedure employed by Romanoschi et al. (2004) and Kroge et al. (2009). VDOT currently follows the AASHTO 1993 pavement design methodology (AASHTO, 1993), and as such, the structural capacity of a pavement is defined by the effective structural number ($SN_{eff}$). The $SN_{eff}$ for an idealized three-layer pavement structure is given as the following (assuming the drainage coefficient is equal to 1):

$$SNe_{ff} = a_1 D_1 + a_2 D_2 + a_3 D_3 \quad \text{(Eq. 4)}$$

where

$a_i$ = layer coefficient for layer i
$D_i$ = thickness of layer i.

Modeling the reclaimed pavement as an idealized three-layer structure (e.g., HMA, reclaimed layer, existing aggregate base), Equation 4 can be rewritten to solve for the layer coefficient of the reclaimed layer ($a_2$) by determining the layer thicknesses and the in-place structural number and assuming a value for $a_1$ and $a_3$. VDOT's current design procedures call for typical HMA and aggregate base layer coefficients of 0.44 and 0.12, respectively (VDOT, 2000). By rearranging Equation 4, the reclaimed layer coefficient ($a_2$) can be calculated as follows:

$$a_2 = \left[ \frac{SN_{eff} - 0.44 * D_1 - 0.12 * D_3}{D_2} \right] \quad \text{(Eq. 5)}$$

The pavement layer thicknesses used to calculate the reclaimed layer coefficients were shown in Table 4.

Figures 10 and 11 show the calculated layer coefficient using Equation 5 for SR 40, SR 13, and SR 6, respectively. The layer coefficient for the eastbound direction of SR 40 ranged from approximately 0.12 to 0.29 within the asphalt emulsion section and from approximately 0.18 to 0.33 within the foamed asphalt section. The effective structural number for the westbound direction ranged from approximately 0.12 to 0.24 within the asphalt emulsion section and from approximately 0.22 to 0.33 within the foamed asphalt section. The layer coefficients of the SR 13 and SR 6 projects ranged from approximately 0.25 to 0.34 and 0.24 to 0.33, respectively.

A logarithmic trend line was applied to highlight the basic trend of increasing structural capacity as time progresses. Equations 6 and 7 show the regression equation and coefficient of determination ($R^2$) for the asphalt emulsion and foamed asphalt sections, respectively, in the eastbound direction for the SR 40 site. Equations 8 and 9 show the regression equation and $R^2$ for the asphalt emulsion and foamed asphalt sections, respectively, in the westbound direction for the SR 40 site. Equations 10 and 11 show the regression equation and $R^2$ for the eastbound and westbound directions, respectively, for the SR 13 site. Equations 12 and 13 show the regression equation and $R^2$ for the eastbound and westbound directions, respectively, for the SR 6 site.

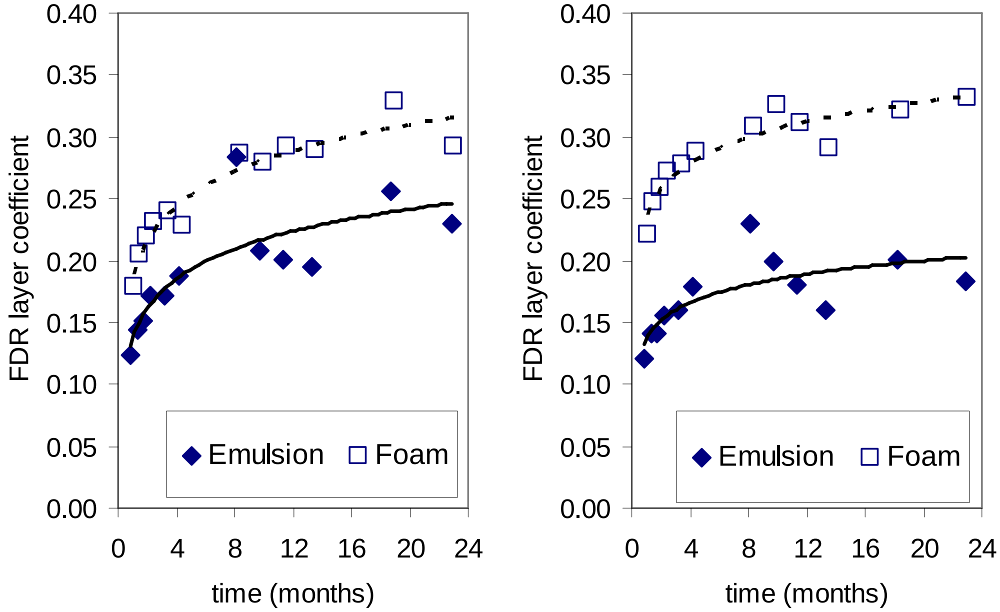



**Figure 10. Layer Coefficient for State Route 40: eastbound *(left)*, westbound *(right)***

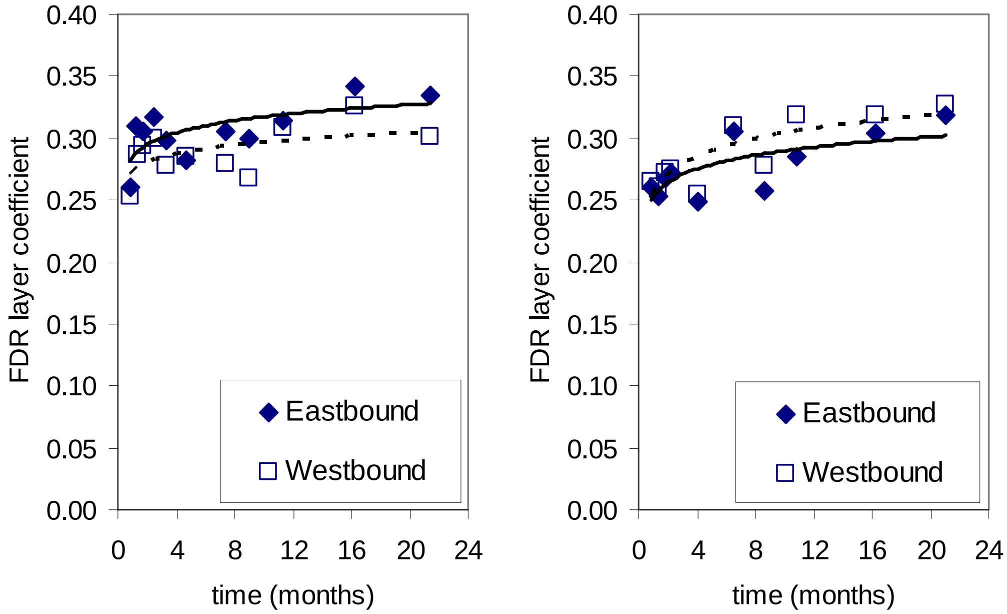



**Figure 11. Layer Coefficient for State Route 13 *(left)* and State Route 6 *(right)***

*State Route 40*

| | | | |
|---|---|---|---|
| Eastbound Emulsion | $y = 0.0349Ln(x) + 0.1371$ | $(R^2 = 0.68)$ | (Eq. 6) |
| Eastbound Foam | $y = 0.0414Ln(x) + 0.1857$ | $(R^2 = 0.92)$ | (Eq. 7) |
| Westbound Emulsion | $y = 0.0211Ln(x) + 0.1368$ | $(R^2 = 0.58)$ | (Eq. 8) |
| Westbound Foam | $y = 0.0309Ln(x) + 0.2355$ | $(R^2 = 0.88)$ | (Eq. 9) |

*State Route 13*

| | | | |
|---|---|---|---|
| Eastbound | $y = 0.0104Ln(x) + 0.2732$ | ($R^2 = 0.31$) | (Eq. 10) |
| Westbound | $y = 0.014Ln(x) + 0.2851$ | ($R^2 = 0.46$) | (Eq. 11) |

*State Route 6*

| | | | |
|---|---|---|---|
| Eastbound | $y = 0.016Ln(x) + 0.253$ | ($R^2 = 0.52$) | (Eq. 12) |
| Westbound | $y = 0.0211Ln(x) + 0.2559$ | ($R^2 = 0.70$) | (Eq. 13) |

The logarithmic regression equation proved to be a fair-to-good fit for the data points and was also thought to be a good model form, intuitively, based on the expected strength-gain behavior through continued curing. The model fit, shown in terms of the coefficient of determination ($R^2$), was better for the asphalt-based projects (SR 40) than for the cement-based projects (SR 13 and 6). This indicates that the increase in structural capacity at early ages is not as great for the cement-based projects as for the asphalt-based projects. The $R^2$ values also show that the model fit was not as good for the emulsion sections as for the foam sections. If the data point from the 8-month test is considered an outlier and removed, the $R^2$ value approaches 0.90 and 0.75 for the eastbound and westbound emulsion sections, respectively. Given that this test was conducted at the end of January 2009, it is possible that some part of the pavement system was frozen and thus the section appears stiffer. From the developed regression equation, a layer coefficient value was calculated for each project to represent the structural capacity of the sections at ages ranging from 2 to 24 months. The results are shown in Table 5.

**Table 5. Calculated Layer Coefficient for Full-Depth Reclamation Demonstration Projects From Regression Analysis**

| | State Route 40 | | | | State Route 13 | | State Route 6 | |
|---|---|---|---|---|---|---|---|---|
| | Asphalt Emulsion | | Foamed Asphalt | | Cement | | Cement | |
| Months | EB | WB | EB | WB | EB | WB | EB | WB |
| 2 | 0.16 | 0.15 | 0.21 | 0.26 | 0.28 | 0.29 | 0.26 | 0.27 |
| 4 | 0.19 | 0.17 | 0.24 | 0.28 | 0.29 | 0.30 | 0.28 | 0.29 |
| 6 | 0.20 | 0.17 | 0.26 | 0.29 | 0.29 | 0.31 | 0.28 | 0.29 |
| 12 | 0.22 | 0.19 | 0.29 | 0.31 | 0.30 | 0.32 | 0.29 | 0.31 |
| 18 | 0.24 | 0.20 | 0.31 | 0.32 | 0.30 | 0.33 | 0.30 | 0.32 |
| 24 | 0.25 | 0.20 | 0.32 | 0.33 | 0.31 | 0.33 | 0.30 | 0.32 |

EB = eastbound; WB = westbound.

## Condition Assessment

Table 6 shows the condition of the three test sections in terms of the average measured rut depth and the average CCI for 2009 and 2010. The values are from the eastbound direction (principal direction) at all three test sites. The rutting values at the three sites can be compared with a typical rutting limit of 0.25 in. The SR 13 project site appears to be approaching this limit at a faster pace than the other sites, but the reason is not clear. Table 3 shows the number of trucks per day was not as high as for the other sites, and the FWD data shown in Appendix C indicated that neither the subgrade resilient modulus nor pavement modulus appeared deficient. When the CCI value is compared with Table 1, the three sections could all be classified as being in excellent condition. Figures D1 through D6, Appendix D, show the individual measurement values from which Table 6 is derived.

**Table 6. Average Measured Rut Depth and Average Critical Condition Index for State Route 40, State Route 13, and State Route 6, Eastbound Direction (2009 and 2010)**

| State Route | Rut depth, in | | Critical Condition Index | |
|---|---|---|---|---|
| | 2009 | 2010 | 2009 | 2010 |
| 40[a] | 0.07 | 0.11 | 100.0 | 99.7 |
| 13 | 0.06 | 0.16 | 99.9 | 93.1 |
| 6 | 0.06 | 0.07 | 100.0 | 99.7 |

[a] Values for SR 40 are averaged across asphalt emulsion and foamed asphalt sections.

## Life-Cycle Cost Analysis

An LCCA was performed to explore the potential for cost savings if VDOT were to implement an FDR program on its flexible pavement network. The LCCA, based on the present cost of rehabilitation alternatives, compared a traditional pavement rehabilitation program (including partial-depth mill and replacement) with one that includes FDR in addition to periodic partial-depth mill and replacement. The LCCA is intended to be a hypothetical example to illustrate the potential benefits of FDR; specific details for a particular project location could alter the assumptions. Table 7 shows the treatments and treatment schedules considered for this analysis. The treatment schedules are based on VDOT experience and information gathered from the literature. The analysis period was chosen as 50 years in accordance with VDOT's LCCA documentation (VDOT, 2008). Many other maintenance schedules could be assumed depending on the maintenance interval and choice of pavement rehabilitation applications.

The treatments listed in Table 7 were applied to a hypothetical pavement structure that was typical for VDOT's primary network. The pavement structure was assumed to have a bound layer (HMA) thickness of 8 in. The pavement structure was assumed to be an existing structure where the first year of needed major rehabilitation was to occur at year 0. This represents the case where a current pavement is in need of rehabilitation and the two options discussed in Table 7 could be considered as potential rehabilitation treatment options. The projected milling and overlay treatments all included full-depth patching of 1% of the pavement surface area. The FDR treatment was assumed to be performed to a depth of 8 in. The salvage value at year 50 was calculated as the cost of the previous rehabilitation treatment multiplied by the proportion of the life still remaining. For example, if the cost of the most recent treatment was $\$10/yd^2$ and the pavement had an expected life of 12 years, a salvage value could be calculated after 8 years as $\$10(4/12) = \$3.33/yd^2$.

**Table 7. Pavement Rehabilitation Schedule**

| Year | Traditional Approach | Approach Including FDR |
|---|---|---|
| 0 | 1% patch, 4-in mill and overlay | 2-in mill, FDR and 3-in overlay |
| 8 | 1% patch, 2-in mill and overlay | --- |
| 12 | --- | 1% patch, 2-in mill and overlay |
| 16 | 1% patch, 2-in mill and overlay | --- |
| 22 | --- | 1% patch, 2-in mill and overlay |
| 24 | 1% patch, 2-in mill and overlay | --- |
| 32 | 1% patch, 4-in mill and overlay | 2-in mill, FDR and 3-in overlay |
| 40 | 1% patch, 2-in mill and overlay | --- |
| 44 | --- | 1% patch, 2-in mill and overlay |
| 48 | 1% patch, 2-in mill and overlay | --- |
| 50 | Salvage | Salvage |

FDR = full-depth reclamation.

The LCCA was performed by calculating the cost for each future rehabilitation activity shown in Table 7 and then discounting those costs to the present. A discount rate of 4% was applied. The cost of each rehabilitation treatment was calculated per square yard based on the assumptions indicated in Table 8. The materials cost data used were average prices for materials and labor and could be expected to vary slightly depending on specific materials used and region. The cost of each rehabilitation treatment used in the LCCA is shown in Table 9.

The results of the present cost analysis are shown in Table 10. The results show that the pavement rehabilitation approach incorporating in-place recycling offers a potential present-cost savings of 16.3% or \$6.99/yd$^2$ based on a 50-year analysis period. The potential savings estimates are thought to be very conservative and are less than the ranges (approximately 40%-50%) found in the literature (Alkins et al., 2008; Collings and Jenkins, 2010; Matthews, 2008).

**Table 8. Materials and Labor Cost Assumptions**

| Item | Unit | Cost per Unit, $ |
|---|---|---|
| HMA milling | Square yards at 2-in depth | 1.50 |
| HMA milling | Square yards at 4-in depth | 3.00 |
| HMA (mainline paving) | Ton[a] | 70.00 |
| HMA (patching) | Ton[a] | 150.00 |
| FDR | Square yards at 8-in depth | 6.00 |

HMA = hot-mix asphalt; FDR = full-depth reclamation.
[a] Applied at 110 lb/yd$^2$/in.

**Table 9. Rehabilitation Treatment Costs**

| Treatment | Cost, \$/yd$^2$ |
|---|---|
| 1% patch, 2-in mill and overlay | 9.70 |
| 1% patch, 4-in mill and overlay | 18.90 |
| 2-in mill, FDR and 3-in overlay | 19.05 |

FDR = full-depth reclamation.

**Table 10. Present Costs of Pavement Rehabilitation**

| | Traditional Approach | | Approach Including FDR | |
|---|---|---|---|---|
| Year | Action | Present Cost, \$/yd$^2$ | Action | Present Cost, \$/yd$^2$ |
| 0 | 1% patch, 4-in mill and overlay | 18.90 | 2-in mill, FDR and 3-in overlay | 19.05 |
| 8 | 1% patch, 2-in mill and overlay | 7.08 | --- | --- |
| 12 | --- | --- | 1% patch, 2-in mill and overlay | 6.06 |
| 16 | 1% patch, 2-in mill and overlay | 5.18 | --- | --- |
| 22 | --- | --- | 1% patch, 2-in mill and overlay | 4.09 |
| 24 | 1% patch, 2-in mill and overlay | 3.78 | --- | --- |
| 32 | 1% patch, 4-in mill and overlay | 5.39 | 2-in mill, FDR and 3-in overlay | 5.43 |
| 40 | 1% patch, 2-in mill and overlay | 2.02 | --- | --- |
| 44 | --- | --- | 1% patch, 2-in mill and overlay | 1.73 |
| 48 | 1% patch, 2-in mill and overlay | 1.48 | --- | --- |
| 50 | Salvage | (1.02) | Salvage | (0.55) |
| Total | | 42.80 | | 35.81 |

FDR = full-depth reclamation.

## Extension of Results to VDOT Pavement Network

The results of the LCCA were considered in the context of VDOT's pavement network to determine the potential for cost savings from VDOT's prospective. The results of the 2009 pavement condition survey (VDOT Maintenance Division, unpublished data, 2010) were reviewed to determine potential projects that might be the most suitable locations for FDR. Pavement condition data were used to determine the most appropriate project locations. In general, the sections selected comprised the most severely deteriorated pavement sections. Potential project sites were selected from a pool of all flexible pavements on the primary and secondary network; this list was further modified using the criteria described here.

The list of potential FDR sites on the primary network that were flexible pavement was refined in accordance with three criteria: (1) condition in terms of LDR, (2) length of homogeneous section, and (3) existing percent patched area. To satisfy the first criterion, those sections rated as having an LDR less than 50 were selected. The LDR criterion was selected to identify only those locations where the primary mode of failure was load-related deterioration and to select the most severely deteriorated sites. This reduced the potential pool of sites to approximately 2,383 lane-miles. The second criterion of project length was included to select sites having a sufficient length that could be economically feasible to construct as a single project. Sites greater than 1.0 directional miles were chosen and resulted in a potential pool of sites of approximately 1,968 lane-miles. The third criterion identified those sites having an existing patched area greater than 15%. Guthrie et al. (2007) identified this as a representative cut-off point where further patching was no longer cost-effective and FDR became an economically viable option. Inclusion of the third criterion resulted in a final potential pool of sites on the primary network of 250.92 lane-miles.

The final potential pool of sites considered as good candidates for FDR on the primary network consisted of 47 individual sites. The average site had a length of 5.3 lane-miles, with the shortest and longest sites being 2.2 and 13.9 lane-miles, respectively. The collected pavement condition data also measures the average lane width, and this information was used to determine the area (square yards) within the 47 sites. By multiplying the individual project length (in lane-miles) by the average project lane width, a total area of 1,424,215 $yd^2$ of potential FDR sites on the primary network was calculated. Table 11 shows the breakdown by VDOT district. Not all districts contained sites that were identified following the procedure described herein. Thus, the sites in Table 11 should not be considered an absolute list. Additional sites could be included or those sites shown in Table 11 could be removed depending on the results of the project-level investigation that should be conducted for any project.

If the present costs of the traditional pavement rehabilitation approach shown in Table 10 are multiplied by the total area of the potential FDR sites, the cost over a 50-year life cycle is calculated as \$60.95 million (\$42.80/$yd^2$). If the present costs of the pavement rehabilitation approach incorporating FDR shown in Table 10 are multiplied by the total area of the potential FDR sites, the cost over a 50-year life cycle is calculated as \$51.00 million (\$35.81/$yd^2$). Thus, it is feasible that VDOT could save approximately \$10 million (approximately \$40,000/lane-mile) over a 50-year period by implementing an FDR program for those flexible pavements identified on the primary network. If these savings are annualized, the potential savings are

**Table 11. Breakdown of Potential Full-Depth Reclamation Sites by VDOT District, Primary Network**

| District | No. of Sites | Lane-miles | Area, $yd^2$ |
|---|---|---|---|
| 1—Bristol | 1 | 2.43 | 16,550 |
| 2—Salem | 17 | 89.55 | 483,788 |
| 3—Lynchburg | 5 | 25.68 | 145,683 |
| 4—Richmond | 3 | 9.13 | 52,792 |
| 5—Hampton Roads | - | | |
| 6—Fredericksburg | 4 | 19.07 | 110,142 |
| 7—Culpeper | - | | |
| 8—Staunton | 9 | 66.92 | 385,521 |
| 9—Northern Virginia | 8 | 38.14 | 229,739 |
| Total | | | 1,424,215 |

approximately $463,000/year (approximately $1,850/lane-mile/year). The potential annual savings will likely not be realized each year as the anticipated maintenance expenditures shown in Table 10 occur periodically.

By further extension, if the same criteria are applied to the 2009 pavement condition data collected for the secondary network, an additional 114 sites having a total length of 230 lane-miles and an area of 1,469,623 $yd^2$ are identified. Pavements on the secondary network vary greatly in scope from high-volume urban collector routes to local streets. In an effort to include only those sites likely to carry the highest traffic volumes, the researchers chose to include those projects that had a lane width greater than 11.0 ft as an additional selection criterion. VDOT does not maintain a coordinated condition and traffic database to allow for a direct determination of traffic volumes. When lane width was included as a criterion, the potential FDR projects on the secondary network comprised 125.53 lane-miles with an area of 873,625 $yd^2$.

Although 100% of the interstate and primary routes are rated annually, only 20% of the secondary system is rated each year. If the remaining 80% of the secondary network is assumed to be similar to the 20% sample, the total potential project list for the secondary network becomes 627.65 lane-miles (4,368,125 $yd^2$). According to the pavement rehabilitation costs shown in Table 10, it is feasible that VDOT could save an additional approximately $30.5 million over a 50-year period by implementing an FDR program for those flexible pavements identified on the secondary network. If these savings are annualized, the potential savings are approximately $1.42 million/year. The potential annual savings will likely not be realized each year as the anticipated maintenance expenditures shown in Table 10 occur periodically.

## CONCLUSIONS

- *Accurate estimation of the final structural adequacy of the reclamation process for sections using asphalt-based stabilizing agents may not be feasible immediately after a project is completed.* Over the first 2 years, the calculated layer coefficients for the FDR material using asphalt emulsion, foamed asphalt, and portland cement as stabilizing agents ranged from 0.12 to 0.29, 0.18 to 0.33, and 0.24 to 0.34, respectively.

- *Regression modeling may be a means for calculating the ultimate structural capacity of asphalt-based reclaimed materials before the complete gain in strength is achieved.*

- *The lower layer coefficient for the emulsion section may be due in part to the softer PG 58-22 asphalt binder employed.* It is possible that this softer binder will result in enhancements in other areas (such as fatigue-cracking resistance). Conversely, the softer binder used for the emulsion could lead to higher rutting susceptibility. Neither fatigue nor rutting susceptibility was evaluated during this study.

- *Estimation of the structural adequacy of FDR sections using portland cement may be feasible much sooner than for the other stabilizing agents investigated in this study.*

- *Laboratory resilient modulus and indirect tensile strength testing showed no statistical difference between foamed asphalt and asphalt emulsion stabilizing agents.*

- *Laboratory resilient modulus strength testing showed no statistical difference between results for cores collected at 5 months versus cores collected at 33 months for foamed asphalt and asphalt emulsion stabilizing agents.*

- *Laboratory indirect tensile strength testing showed no statistical difference between results for cores collected at 5 months versus cores collected at 33 months for foamed asphalt specimens, but a statistically different decrease was noted for the asphalt emulsion specimens.*

- *If VDOT were to implement a pavement rehabilitation strategy that included FDR, the potential savings for the primary network and secondary network would be approximately $10 million and $30.5 million, respectively, over a 50-year life cycle. Annualized savings of approximately $463,000 and $1.42 million are estimated for the primary and secondary networks, respectively.*

## RECOMMENDATIONS

1. *VDOT's Materials Division and Maintenance Division should encourage the districts to pursue FDR as a pavement rehabilitation technique on those flexible pavement sections where it is most suitable.* The selection criteria used during the LCCA analysis could serve as an initial set of guidelines to determine appropriate pavement sections, but the guidelines should be revised as VDOT gains experience with the techniques.

2. *VDOT's Materials Division and the Virginia Center for Transportation Innovation and Research (VCTIR) should develop a formal list of criteria to select future FDR projects with the assistance of other states that use FDR more often.*

3. *VDOT's Richmond and Salem districts should continue to asses the rutting performance of the three trial sections to compare with typical limits of rutting distress.*

4. *VCTIR should pursue laboratory testing of in-place pavement recycling technologies to characterize further materials for use in mechanistic-based pavement design and analysis.*

## COSTS AND BENEFITS ASSESSMENT

Over a 50-year life cycle, it is feasible that VDOT could save approximately $10 million and $30.5 million by implementing an FDR program for those flexible pavements identified on the primary and secondary networks, respectively. If these potential savings are annualized, they are estimated at approximately $463,000 and $1.42 million for the primary and secondary networks, respectively.

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# APPENDIX A

# RECOMMENDED STABILIZING AGENTS

**Table A1. Recommended Stabilizing Agents**

| Material Type | Well-graded Gravel | Poorly graded Gravel | Silty Gravel | Clayey Gravel | Well-graded Sand | Poorly graded Sand | Silty Sand | Clayey Sand | Silt, Silt with Sand | Lean Clay | Organic Silt/ Organic Lean Clay | Elastic Silt | Fat Clay, Fat Clay with Sand |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **USCS** | **GW** | **GP** | **GM** | **GC** | **SW** | **SP** | **SM** | **SC** | **ML** | **CL** | **OL** | **MH** | **CH** |
| **AASHTO** | **A-1-a** | **A-1-a** | **A-1-b** | **A-1-b or A-2-6** | **A-1-b** | **A-3 or A-1-b** | **A-2-4 or A-2-5** | **A-2-6 or A-2-7** | **A-4 or A-5** | **A-6** | **A-4** | **A-5 or A-7-5** | **A-7-6** |
| Emulsion FDR, Best if SE > 30 and $P_{200} < 20$ | | | | | | | | | | | | | |
| Foamed asphalt, $P_{200}$ 5% to 20% and follow maximum density gradation | | | | | | | | | | | | | |
| Portland cement, PI < 10 | | | | | | | | | | | | | |
| Lime, P I> 10 and $P_{200} < 25$ or PI 10-30 and $P_{200} > 25$, $SO_4$ in clay < 3,000 ppm | | | | | | | | | | | | | |

Shaded cells indicate recommended conditions.

USCS = unified soil classification system; AASHTO = American Association of State and Highway Transportation Officials; FDR = full-depth reclamation; SE = sand equivalent; $P_{200}$ = percent passing No. 200 sieve; PI = plasticity index; ppm = parts per million.

*Source:* The recommendations in this table are from Thomas, T. (2010). Mix Designs for FDR, CIR, and HIR. Paper presented at the Northeast and Mid-Atlantic States In-Place Recycling Conference, Harrisburg, PA, August 24-26.

# APPENDIX B

# RESULTS OF LABORATORY TESTING

**Table B1. Gradation and Asphalt Content Analysis, State Route 40 Trial Section (cores collected at 5 months)**

| Sieve Size | Asphalt Emulsion | | | Foamed Asphalt | | | |
|---|---|---|---|---|---|---|---|
| | E2 (upper) | E2 (lower) | W4 | E3 (upper) | E3 (lower) | W2 (upper) | W2 (lower) |
| 1¼ in | 100 | 100 | 100 | 100 | 100 | 100 | 100 |
| 1 in | 97 | 93 | 99 | 100 | 100 | 98 | 100 |
| ¾ in | 93 | 93 | 97 | 99 | 95 | 98 | 100 |
| ½ in | 91 | 87 | 92 | 98 | 94 | 96 | 98 |
| 3/8 in | 86 | 81 | 87 | 93 | 88 | 90 | 92 |
| No. 4 | 68 | 61 | 66 | 71 | 67 | 69 | 68 |
| No. 8 | 52 | 48 | 51 | 55 | 51 | 51 | 51 |
| No. 16 | 42 | 38 | 40 | 43 | 40 | 39 | 40 |
| No. 30 | 34 | 31 | 32 | 34 | 32 | 31 | 31 |
| No. 50 | 26 | 23 | 25 | 25 | 24 | 24 | 25 |
| No. 100 | 18 | 15 | 17 | 17 | 15 | 18 | 18 |
| No. 200 | 10.4 | 8.0 | 10.1 | 9.3 | 8.7 | 11.2 | 11.5 |
| Asphalt Content, % | 7.33 | 6.95 | 7.2 | 9.32 | 8.52 | 7.4 | 7.99 |

E = eastbound direction; W = westbound direction.

**Table B2. Results of Resilient Modulus and Indirect Tensile Strength Tests, State Route 40 Trial Section (cores collected at 5 months)**

| Stabilizing Agent | Core ID | Upper or Lower | Resilient Modulus, psi (20°C) | Indirect Tensile Strength, psi |
|---|---|---|---|---|
| Asphalt emulsion | E2 | Upper | 660,700 | 108 |
| | | Lower | 423,900 | 90 |
| | W4 | [a] | 413.200 | 130 |
| Foamed asphalt | E3 | Upper | 610,300 | 131 |
| | | Lower | 632,300 | 133 |
| | W2 | Upper | 468,100 | 79 |
| | | Lower | 397,700 | 73 |

E = eastbound direction; W = westbound direction.

[a] Core W4 was not tall enough to create two specimens.

**Table B3. Results of Dynamic Modulus Test, State Route 40 Trial Section (cores collected at 5 months)**

| Temperature, °C | Frequency, Hz | Dynamic Modulus, psi | |
|---|---|---|---|
| | | Asphalt Emulsion | Foamed Asphalt |
| 20 | 0.1 | 81,482 | 125,095 |
| | 0.2 | 94,985 | 145,763 |
| | 0.5 | 114,420 | 176,946 |
| | 1 | 143,254 | 213,931 |
| | 2 | 174,770 | 252,366 |
| | 5 | 218,427 | 312,846 |
| | 10 | 246,564 | 354,182 |
| | 20 | 287,175 | 388,556 |
| 4 | 0.1 | 470,212 | 471,373 |
| | 0.2 | 526,922 | 528,227 |
| | 0.5 | 602,197 | 611,044 |
| | 1 | 666,158 | 678,776 |
| | 2 | 730,700 | 750,570 |
| | 5 | 818,303 | 851,081 |
| | 10 | 887,051 | 939,119 |
| | 20 | 997,714 | 1,107,508 |

**Table B4. Results of Resilient Modulus and Indirect Tensile Strength Tests, State Route 40 Trial Section (cores collected at 33 months)**

| Stabilizing Agent | Core ID | Upper or Lower | Resilient Modulus, psi | | | Indirect Tensile Strength, psi |
|---|---|---|---|---|---|---|
| | | | 4°C | 20°C | 38°C | |
| Asphalt emulsion | E1 | Upper | 847,700 | 408,900 | 343,900 | 41.3 |
| | | Lower | 1,263,600 | 932,700 | 746,100 | 89.1 |
| | E2 | Upper | 999,500 | 506,600 | 428,600 | 56.7 |
| | | Lower | 858,600 | 576,800 | 219,600 | 67.9 |
| | W4 | Upper | 922,700 | 448,300 | 316,500 | 95.9 |
| | | Lower | 1,142,600 | 841,100 | 340,100 | [a] |
| Foamed asphalt | E3 | Upper | 898,900 | 379,100 | 219,400 | 84.7 |
| | | Lower | 990,800 | 581,700 | 250,300 | 87.9 |
| | E4[b] | Lower | 1,481,400 | 1,059,500 | 525,000 | 70.6 |

E = eastbound direction; W = westbound direction.
[a] No test for W4 (lower) performed.
[b] No test for E4 (upper) was performed.

# APPENDIX C

# RESULTS OF FALLING-WEIGHT DEFLECTOMETER TESTING

Table C1. Summary of Falling-Weight Deflectometer Testing for Eastbound State Route 40 Trial Section

| Date | Average Surface Temperature, °F | Average Deflection ($D_0$), mils | Average Subgrade Resilient Modulus, psi | Average Pavement Modulus, psi | Average Effective Structural Number, $SN_{eff}$ | Layer Coefficient | | |
|---|---|---|---|---|---|---|---|---|
| | | | | | | Average | Standard Deviation | Coefficient of Variation, % |
| **Asphalt Emulsion Section** | | | | | | | | |
| Before | 68.1 | 15.9 | 20,860 | 69,676 | 3.55 | n/a | n/a | n/a |
| 6/16/2008 | 110.5 | 26.8 | 18,432 | 46,941 | 2.53 | 0.12 | 0.035 | 28.6% |
| 6/30/2008 | 95.5 | 23.2 | 19,303 | 58,265 | 2.72 | 0.14 | 0.039 | 27.3% |
| 7/14/2008 | 106.3 | 22.6 | 19,019 | 63,206 | 2.78 | 0.15 | 0.044 | 29.1% |
| 7/28/2008 | 98.4 | 19.5 | 19,967 | 74,912 | 2.95 | 0.17 | 0.042 | 24.7% |
| 8/25/2008 | 102.7 | 18.4 | 21,121 | 75,941 | 2.97 | 0.17 | 0.043 | 25.0% |
| 9/25/2008 | 70.2 | 15.9 | 21,755 | 87,559 | 3.12 | 0.19 | 0.044 | 23.6% |
| 1/21/2009 | 32.9 | 9.8 | 24,815 | 199,471 | 4.05 | 0.28 | 0.081 | 28.4% |
| 3/11/2009 | 76.4 | 15.5 | 20,683 | 90,176 | 3.30 | 0.21 | 0.051 | 24.5% |
| 4/28/2009 | 93.5 | 19.6 | 19,337 | 84,088 | 3.23 | 0.20 | 0.046 | 23.1% |
| 6/24/2009 | 105.0 | 17.4 | 19,490 | 80,118 | 3.18 | 0.20 | 0.044 | 22.5% |
| 12/3/2009 | 52.7 | 11.3 | 22,690 | 134,235 | 3.79 | 0.26 | 0.053 | 20.7% |
| 4/7/2010 | 88.9 | 14.6 | 21,250 | 104,688 | 3.47 | 0.23 | 0.053 | 22.8% |
| **Foamed Asphalt Section** | | | | | | | | |
| Before | 70.4 | 12.4 | 25,181 | 92,000 | 3.89 | n/a | n/a | n/a |
| 6/16/2008 | 112.5 | 11.0 | 23,805 | 143,045 | 3.41 | 0.18 | 0.027 | 15.2% |
| 6/30/2008 | 91.4 | 9.5 | 25,342 | 178,727 | 3.69 | 0.21 | 0.017 | 8.2% |
| 7/14/2008 | 106.5 | 9.1 | 25,567 | 203,818 | 3.84 | 0.22 | 0.030 | 13.4% |
| 7/28/2008 | 105.1 | 8.4 | 25,737 | 222,227 | 3.97 | 0.23 | 0.023 | 10.1% |
| 8/25/2008 | 106.7 | 7.4 | 28,632 | 239,864 | 4.08 | 0.24 | 0.014 | 6.0% |
| 9/25/2008 | 68.9 | 7.7 | 31,721 | 214,727 | 3.93 | 0.23 | 0.017 | 7.4% |
| 1/21/2009 | 35.9 | 6.2 | 32,784 | 334,182 | 4.53 | 0.29 | 0.035 | 12.3% |
| 3/11/2009 | 77.6 | 7.5 | 26,503 | 210,773 | 4.46 | 0.28 | 0.023 | 8.3% |
| 4/28/2009 | 96.4 | 8.6 | 25,628 | 232,136 | 4.60 | 0.29 | 0.029 | 9.8% |
| 6/24/2009 | 106.7 | 7.6 | 26,506 | 228,000 | 4.57 | 0.29 | 0.032 | 11.0% |
| 12/3/2009 | 54.2 | 6.1 | 29,948 | 293,045 | 4.97 | 0.33 | 0.033 | 10.1% |
| 4/7/2010 | 96.1 | 7.5 | 26,290 | 231,727 | 4.60 | 0.29 | 0.031 | 10.4% |

Table C2. Summary of Falling-Weight Deflectometer Testing for Westbound State Route 40 Trial Section

| Date | Average Surface Temperature, °F | Average Deflection ($D_0$), mils | Average Subgrade Resilient Modulus, psi | Average Pavement Modulus, psi | Average Effective Structural Number, $SN_{eff}$ | Layer Coefficient | | |
|---|---|---|---|---|---|---|---|---|
| | | | | | | Average | Standard Deviation | Coefficient of Variation, % |
| **Asphalt Emulsion Section** | | | | | | | | |
| Before | 65.3 | 17.5 | 17,617 | 68,036 | 3.09 | n/a | n/a | n/a |
| 6/16/2008 | 99.8 | 23.3 | 16,104 | 57,000 | 2.58 | 0.12 | 0.014 | 11.9% |
| 6/30/2008 | 92.1 | 19.6 | 17,533 | 72,714 | 2.80 | 0.14 | 0.018 | 12.9% |
| 7/14/2008 | 96.0 | 19.9 | 16,620 | 72,143 | 2.80 | 0.14 | 0.017 | 11.8% |
| 7/28/2008 | 90.1 | 17.2 | 18,169 | 82,857 | 2.94 | 0.15 | 0.016 | 10.5% |
| 8/25/2008 | 90.1 | 16.0 | 18,966 | 85,571 | 2.98 | 0.16 | 0.018 | 11.0% |
| 9/25/2008 | 66.9 | 13.8 | 20,636 | 103,714 | 3.18 | 0.18 | 0.016 | 9.2% |
| 1/21/2009 | 44.3 | 9.8 | 23,883 | 170,385 | 3.73 | 0.23 | 0.028 | 12.2% |
| 3/11/2009 | 72.5 | 12.2 | 20,495 | 131,821 | 3.42 | 0.20 | 0.026 | 13.2% |
| 4/28/2009 | 87.9 | 13.9 | 18,412 | 113,385 | 3.24 | 0.18 | 0.017 | 9.4% |
| 6/24/2009 | 99.9 | 15.7 | 17,690 | 98,115 | 2.91 | 0.16 | 0.017 | 10.9% |
| 11/18/2009 | 46.9 | 11.1 | 22,577 | 146,577 | 3.32 | 0.20 | 0.027 | 13.2% |
| 4/7/2010 | 81.9 | 12.9 | 19,385 | 125,731 | 3.16 | 0.18 | 0.020 | 10.8% |
| **Foamed Asphalt Section** | | | | | | | | |
| Before | 61.7 | 18.1 | 18,560 | 68,500 | 3.08 | n/a | n/a | n/a |
| 6/16/2008 | 93.9 | 13.6 | 26,820 | 197,214 | 3.16 | 0.22 | 0.029 | 13.1% |
| 6/30/2008 | 90.8 | 12.9 | 19,901 | 154,962 | 3.35 | 0.25 | 0.036 | 14.4% |
| 7/14/2008 | 92.1 | 11.9 | 30,026 | 170,607 | 3.45 | 0.26 | 0.043 | 16.5% |
| 7/28/2008 | 85.4 | 11.0 | 22,847 | 185,643 | 3.57 | 0.27 | 0.032 | 11.7% |
| 8/25/2008 | 84.0 | 10.0 | 29,847 | 194,500 | 3.63 | 0.28 | 0.030 | 11.0% |
| 9/25/2008 | 70.0 | 9.5 | 26,979 | 208,143 | 3.72 | 0.29 | 0.033 | 11.3% |
| 1/21/2009 | 42.3 | 9.1 | 27,499 | 224,893 | 3.81 | 0.31 | 0.025 | 8.2% |
| 3/11/2009 | 66.9 | 8.9 | 26,661 | 257,357 | 3.98 | 0.33 | 0.027 | 8.4% |
| 4/28/2009 | 83.1 | 9.8 | 22,067 | 235,464 | 3.86 | 0.31 | 0.032 | 10.4% |
| 6/24/2009 | 96.2 | 10.6 | 21,946 | 210,033 | 3.72 | 0.29 | 0.039 | 13.3% |
| 11/18/2009 | 46.6 | 8.4 | 28,434 | 266,667 | 4.02 | 0.32 | 0.051 | 15.9% |
| 4/7/2010 | 76.2 | 9.8 | 21,578 | 276,467 | 4.02 | 0.33 | 0.064 | 19.2% |

**Table C3. Summary of Falling-Weight Deflectometer Testing for State Route 6 Trial Section[a]**

| Date | Average Surface Temperature, °F | Average Deflection ($D_0$), mils | Average Subgrade Resilient Modulus, psi | Average Pavement Modulus, psi | Average Effective Structural Number, $SN_{eff}$ | Layer Coefficient | | |
|---|---|---|---|---|---|---|---|---|
| | | | | | | Average | Standard Deviation | Coefficient of Variation, % |
| **Eastbound** | | | | | | | | |
| 8/27/2008 | 67.6 | 9.0 | 16,770 | 386,703 | 4.34 | 0.26 | 0.061 | 23.6% |
| 9/9/2008 | 77.1 | 9.6 | 15,373 | 367,297 | 4.27 | 0.25 | 0.059 | 23.4% |
| 9/24/2008 | 67.4 | 8.6 | 17,206 | 400,459 | 4.41 | 0.27 | 0.054 | 20.3% |
| 10/7/2008 | 63.4 | 8.5 | 17,052 | 414,459 | 4.46 | 0.27 | 0.057 | 20.9% |
| 12/2/2008 | 40.2 | 8.2 | 21,410 | 347,973 | 4.23 | 0.25 | 0.045 | 18.0% |
| 2/12/2009 | 60.1 | 7.8 | 16,069 | 508,149 | 4.79 | 0.30 | 0.054 | 17.8% |
| 4/16/2009 | 83.5 | 10.6 | 13,783 | 375,797 | 4.32 | 0.26 | 0.056 | 21.7% |
| 6/23/2009 | 94.3 | 9.5 | 13,504 | 451,351 | 4.60 | 0.29 | 0.056 | 19.5% |
| 12/1/2009 | 46.7 | 7.4 | 19,130 | 507,162 | 4.78 | 0.30 | 0.057 | 18.6% |
| 4/26/2010 | 66.4 | 7.7 | 15,950 | 558,697 | 4.93 | 0.32 | 0.063 | 19.6% |
| **Westbound** | | | | | | | | |
| 8/27/2008 | 66.6 | 9.8 | 17,816 | 469,432 | 4.05 | 0.26 | 0.086 | 32.4% |
| 9/9/2008 | 79.0 | 10.6 | 14,811 | 454,297 | 4.02 | 0.26 | 0.080 | 30.9% |
| 9/24/2008 | 79.5 | 10.0 | 15,674 | 492,662 | 4.12 | 0.27 | 0.087 | 31.8% |
| 10/7/2008 | 58.9 | 9.4 | 17,571 | 491,892 | 4.13 | 0.27 | 0.080 | 29.2% |
| 12/2/2008 | 44.9 | 8.6 | 21,121 | 437,043 | 3.97 | 0.25 | 0.077 | 30.4% |
| 2/12/2009 | 67.4 | 8.7 | 15,607 | 480,878 | 4.42 | 0.19 | 0.239 | 124.8% |
| 4/16/2009 | 83.5 | 10.5 | 13,862 | 495,703 | 4.17 | 0.28 | 0.070 | 25.0% |
| 6/23/2009 | 103.0 | 10.1 | 12,638 | 621,176 | 4.49 | 0.32 | 0.076 | 23.9% |
| 12/1/2009 | 54.3 | 8.1 | 17,848 | 622,865 | 4.50 | 0.32 | 0.077 | 24.0% |
| 4/26/2010 | 78.6 | 9.2 | 14,018 | 668,592 | 4.57 | 0.33 | 0.085 | 26.0% |

[a]No before FWD testing was conducted on State Route 6.

Table C4. Summary of Falling-Weight Deflectometer Testing for State Route 13 Trial Section

| Date | Average Surface Temperature, °F | Average Deflection ($D_0$), mils | Average Subgrade Resilient Modulus, psi | Average Pavement Modulus, psi | Average Effective Structural Number, $SN_{eff}$ | Layer Coefficient | | |
|---|---|---|---|---|---|---|---|---|
| | | | | | | Average | Standard Deviation | Coefficient of Variation, % |
| **Eastbound** | | | | | | | | |
| Before | 99.6 | 20.4 | 14,861 | 113,122 | 2.13 | N/A | N/A | N/A |
| 8/11/2008 | 96.3 | 8.3 | 22,693 | 430,921 | 4.12 | 0.26 | 0.069 | 26.5% |
| 8/25/2008 | 78.9 | 6.5 | 23,929 | 576,882 | 4.57 | 0.31 | 0.066 | 21.4% |
| 9/8/2008 | 96.8 | 7.2 | 20,701 | 575,184 | 4.53 | 0.31 | 0.083 | 27.1% |
| 9/29/2008 | 84.8 | 6.9 | 20,444 | 612,974 | 4.63 | 0.32 | 0.081 | 25.6% |
| 10/27/2008 | 52.2 | 6.2 | 25,554 | 533,263 | 4.47 | 0.30 | 0.057 | 19.2% |
| 12/4/2008 | 52.7 | 6.9 | 26,854 | 493,579 | 4.32 | 0.28 | 0.071 | 25.0% |
| 2/26/2009 | 71.0 | 7.0 | 20,922 | 571,566 | 4.54 | 0.31 | 0.076 | 24.8% |
| 4/15/2009 | 53.4 | 6.6 | 22,980 | 547,342 | 4.48 | 0.30 | 0.071 | 23.5% |
| 6/22/2009 | 96.0 | 7.5 | 18,958 | 600,000 | 4.61 | 0.31 | 0.078 | 24.9% |
| 11/16/2009 | 74.0 | 6.3 | 21,553 | 705,763 | 4.87 | 0.34 | 0.082 | 23.9% |
| 4/23/2010 | 77.5 | 6.8 | 19,561 | 670,329 | 4.79 | 0.33 | 0.077 | 23.1% |
| **Westbound** | | | | | | | | |
| Before | 87.9 | 23.6 | 14,777 | 102,932 | 2.06 | N/A | N/A | N/A |
| 8/11/2008 | 93.1 | 8.4 | 20,991 | 429,724 | 4.36 | 0.25 | 0.081 | 32.1% |
| 8/25/2008 | 74.9 | 7.0 | 24,929 | 518,829 | 4.67 | 0.29 | 0.079 | 27.7% |
| 9/8/2008 | 90.2 | 7.2 | 22,406 | 542,750 | 4.74 | 0.29 | 0.081 | 27.5% |
| 9/29/2008 | 79.0 | 6.9 | 22,846 | 561,474 | 4.80 | 0.30 | 0.081 | 27.0% |
| 10/27/2008 | 56.3 | 6.8 | 26,366 | 488,316 | 4.59 | 0.28 | 0.074 | 26.6% |
| 12/4/2008 | 58.2 | 6.5 | 23,506 | 506,789 | 4.65 | 0.28 | 0.071 | 24.9% |
| 2/26/2009 | 62.0 | 7.3 | 23,205 | 499,816 | 4.60 | 0.28 | 0.083 | 29.6% |
| 4/15/2009 | 52.8 | 7.2 | 23,541 | 464,066 | 4.50 | 0.27 | 0.076 | 28.2% |
| 6/22/2009 | 88.5 | 7.7 | 20,541 | 593,039 | 4.87 | 0.31 | 0.090 | 29.1% |
| 11/16/2009 | 72.3 | 6.5 | 23,081 | 654,013 | 5.04 | 0.33 | 0.088 | 27.0% |
| 4/23/2010 | 65.8 | 7.0 | 22,652 | 570,105 | 4.80 | 0.30 | 0.089 | 29.7% |

# APPENDIX D

# RESULTS OF CONDITION TESTING

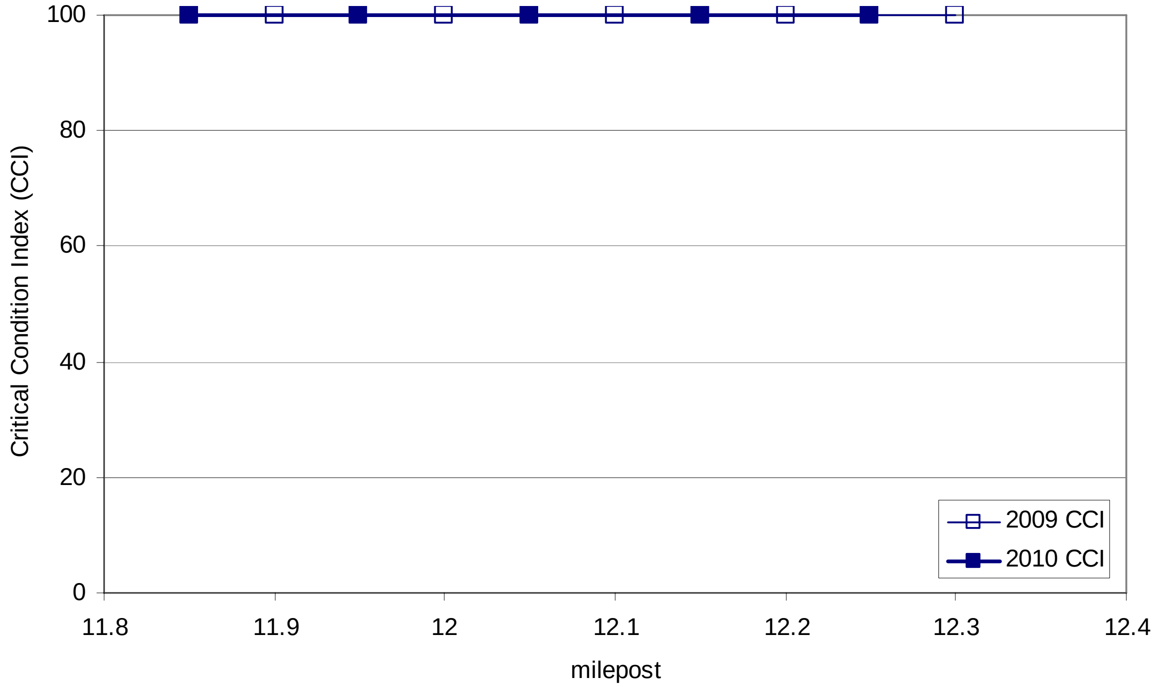


**Figure D1. Critical Condition Index (CCI) of State Route 40 Trial Section, Eastbound Direction (emulsion section = milepost 11.85–12.17, foamed asphalt section = milepost 12.17–12.39)**

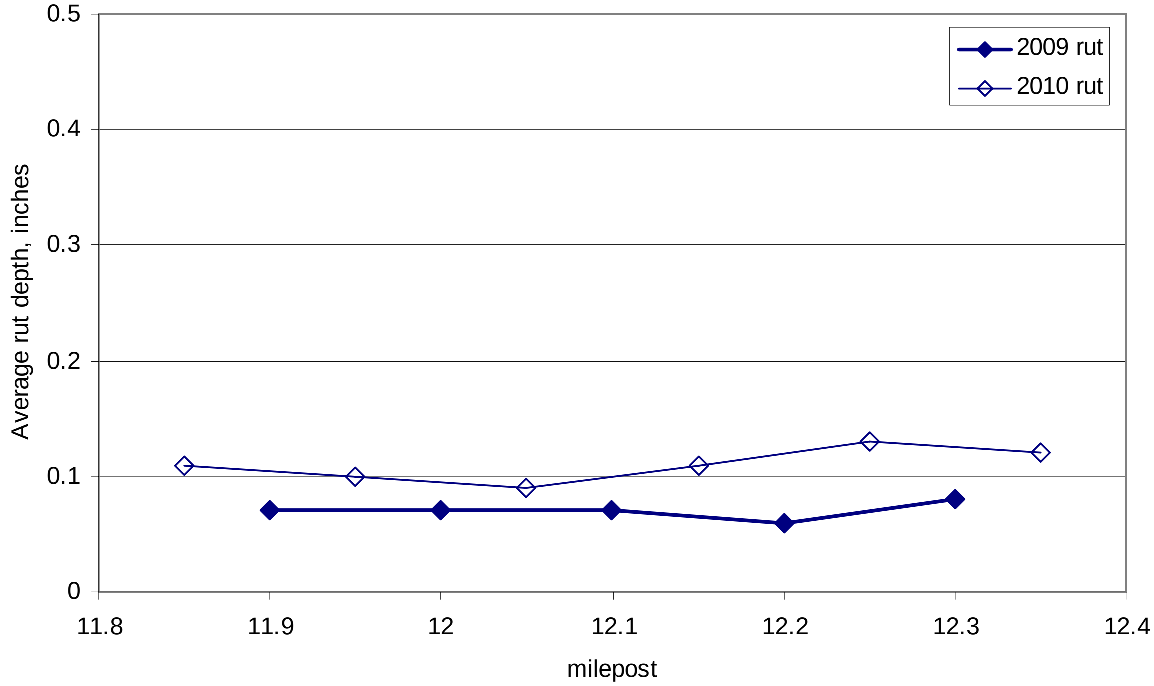


**Figure D2. Rut Depth of State Route 40 Trial Section, Eastbound Direction (emulsion section = milepost 11.85–12.17, foamed asphalt section = milepost 12.17–12.39)**

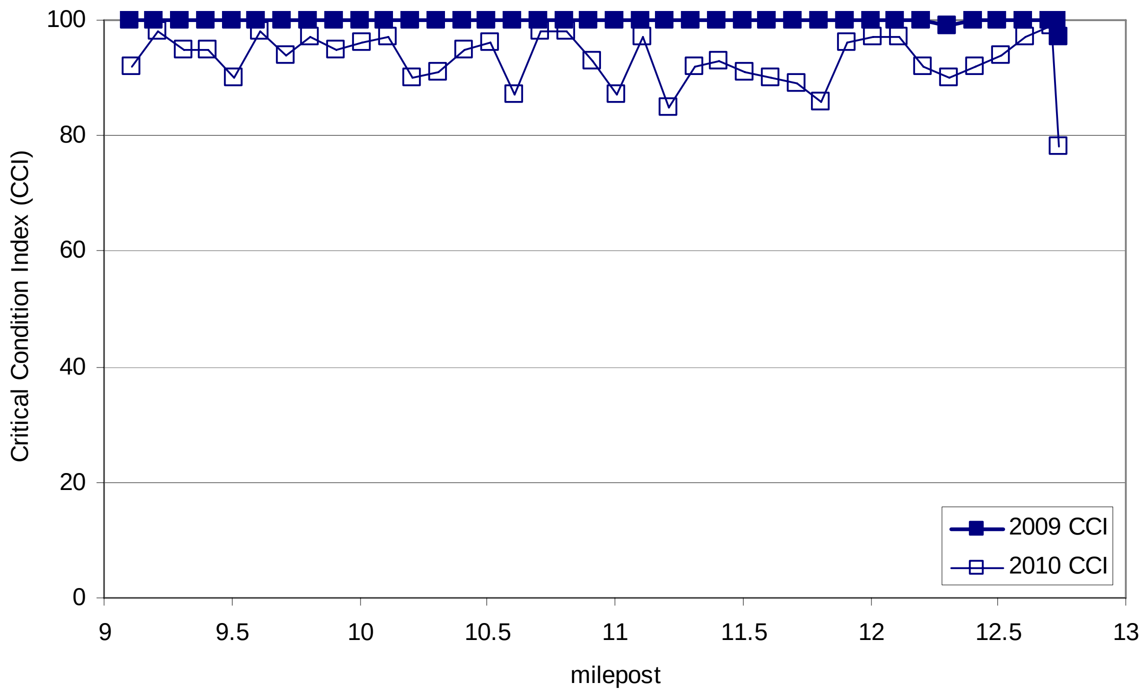


**Figure D3. Critical Condition Index (CCI) of State Route 13 Trial Section, Eastbound Direction**

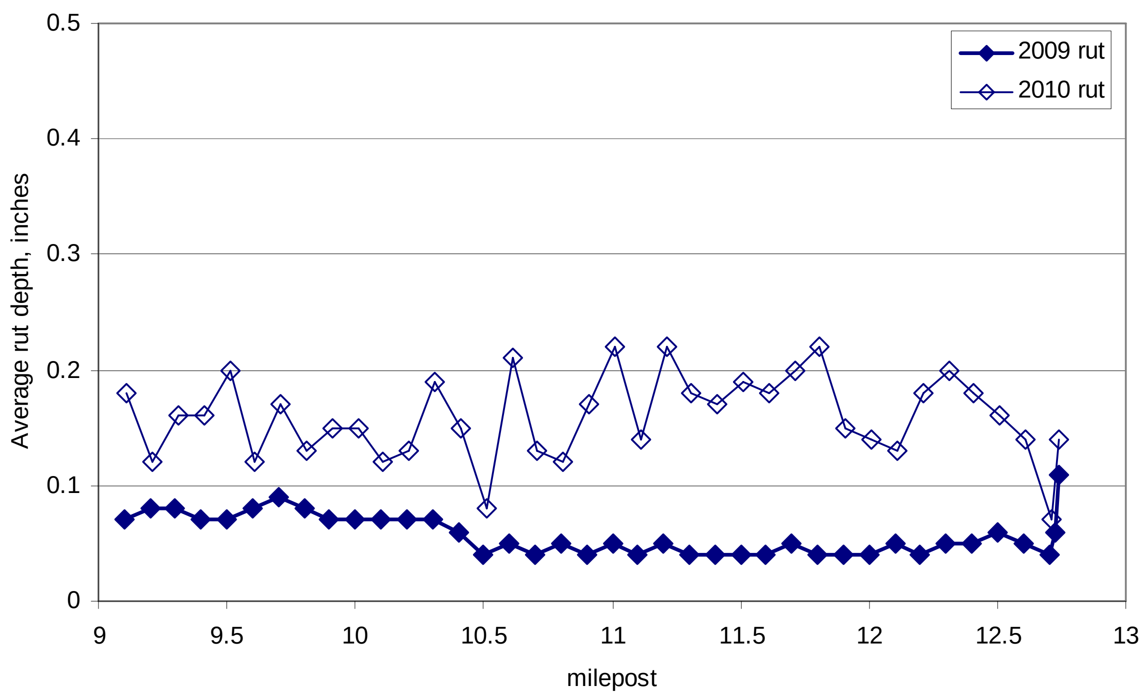


**Figure D4. Rut Depth of State Route 13 Trial Section, Eastbound Direction**

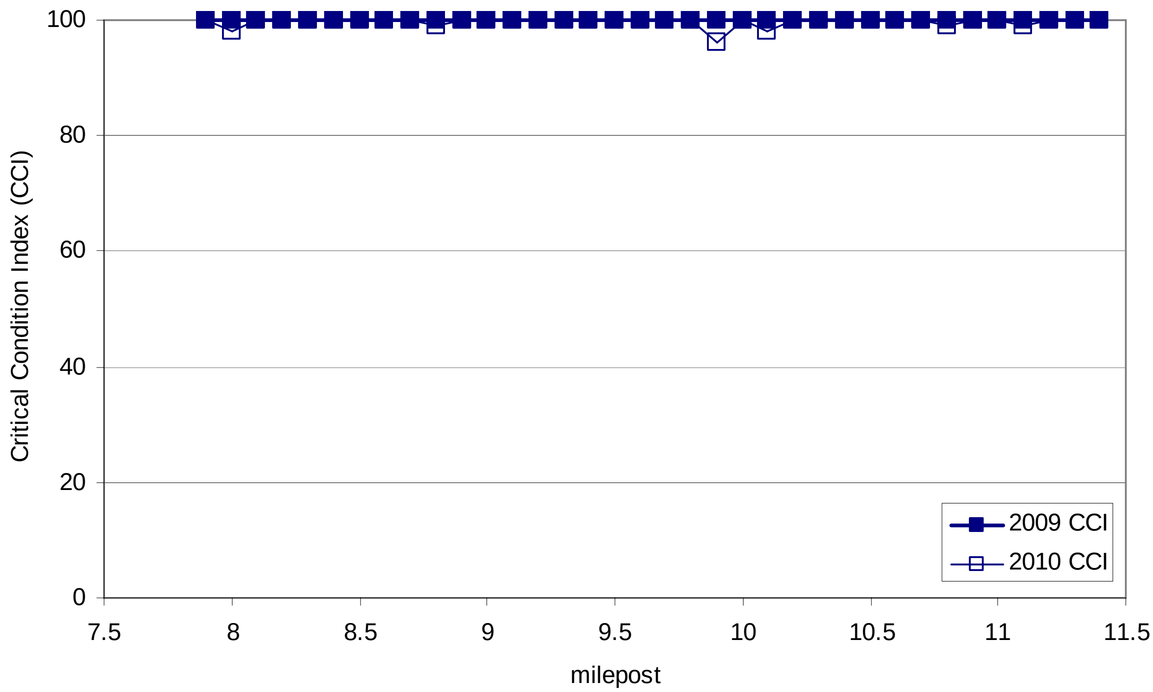


**Figure D5. Critical Condition Index (CCI) of State Route 6 Trial Section, Eastbound Direction**

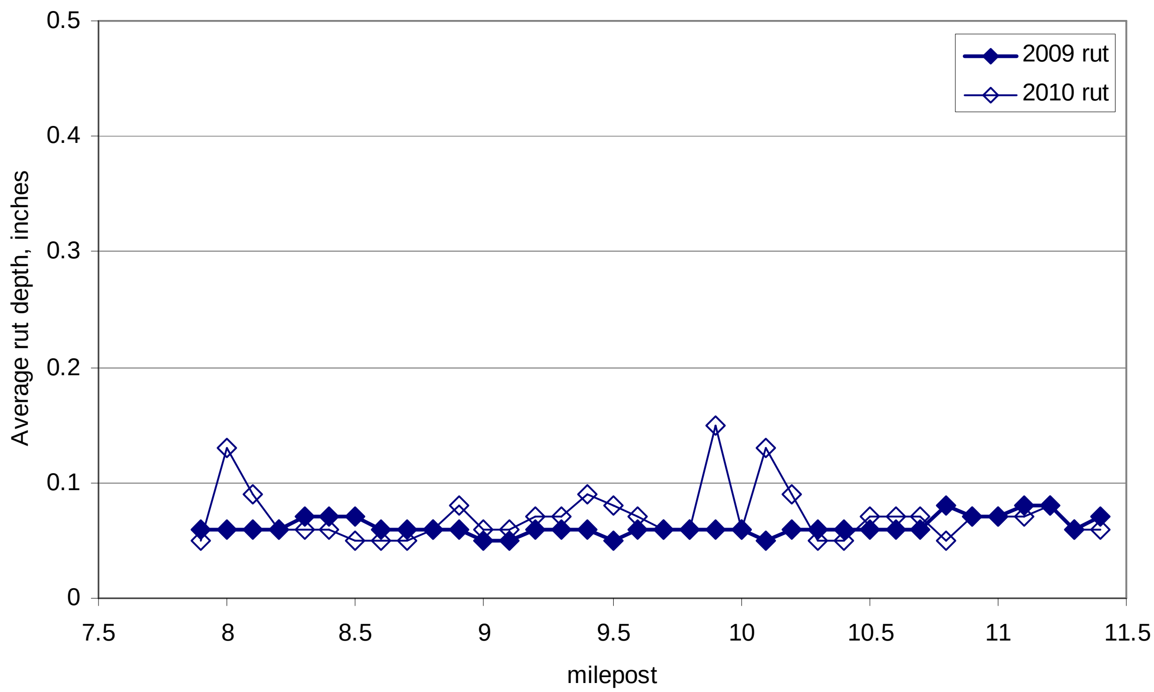


**Figure D6. Rut Depth of State Route 6 Trial Section, Eastbound Direction**