NATIONAL SUSTAINABLE PAVEMENT CONSORTIUM

TPF 5 (268)

Pre-Print

Amarh, E., Fernandez-Gomez, W., Flintsch G.W., Diefenderfer, B., Bowers, B., (2017) “Non-Destructive In Situ Characterization of Elastic Moduli of Full-Depth Reclamation Base Mixtures,” *Transportation Research Record: Journal of the Transportation Research Board*, 2017, vol. 2641 pp 1-11.

NON-DESTRUCTIVE IN SITU CHARACTERIZATION OF ELASTIC MODULI OF FULL-DEPTH RECLAMATION BASE MIXTURES

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Word count: 4991 words text + 7 tables/figures x 250 words (each) + 30 references = 6741/7000 words

Submission Date: November 12, 2016

ABSTRACT
State highway agencies are searching for more cost-effective methods of rehabilitating roads. One sustainable solution is full-depth reclamation (FDR), which is a pavement rehabilitation technique that involves pulverizing and reusing materials from existing distressed pavements in place. However, there is limited information on the long-term properties of these recycled materials. One important property, the elastic modulus, indicates the structural capacity of pavement materials and is highly recommended for design purposes by the *Mechanistic Empirical Pavements Design Guide* (MEPDG). The elastic modulus directly impacts selection of the overall pavement thickness, and an accurate estimation of the modulus is therefore key to a cost-effective pavement design. This research investigated the elastic modulus trends of three in-service pavements rehabilitated with the FDR technique during the 2008 Virginia Department of Transportation (VDOT) construction season. Foamed asphalt (2.7% with 1% cement), asphalt emulsion (3.5%), and Portland cement (5%) were used as stabilizing agents for the FDR layers. The results of the moduli measured for the recycled base materials varied significantly over time. These changes were attributed to curing after construction, seasonal effects, and subgrade moisture. The structural capacity of the pavements improved irrespective of the stabilizing agent used.

Keywords: recycled base, full-depth reclamation, time-dependent response, Critical Condition Index

INTRODUCTION

The FHWA (*1*) estimates the total length of flexible pavements in the United States at approximately 2.5 million centerline miles. A 2013 ASCE report on the nation's infrastructure suggests that 32% of those roads are in poor to mediocre condition (*2*). Given the high costs and environmental impacts of repairs at such a scale, traditional methods to rehabilitate or reconstruct these roads are not feasible or sustainable. Pavement recycling offers an innovative option that is gaining increasing acceptance among state departments of transportation (DOTs) as a method of restoring the service life of pavements with medium to high traffic.

Pavement recycling is often performed in place, reusing materials from the existing pavement to form a base layer usually requiring an asphalt concrete overlay. Pavement recycling offers many advantages over traditional rehabilitation methods, such as lower construction costs and a smaller environmental footprint (*3*) from reducing the use of virgin materials and eliminating the need to haul materials to landfill sites. Pavement recycling may be performed as hot-in-place, cold recycling or full depth reclamation (FDR). Cold recycling is usually carried out as cold in-place recycling (CIR) or cold central-plant recycling (CCPR) depending on the severity of the pavement distresses among other reasons.

FDR is better suited for deteriorated pavements with distresses penetrating deep into the layers. FDR involves pulverizing the existing pavement materials usually up to a depth of 12 in., remixing, and finally compacting the mixture to form a uniform base. The pulverized base mixture is strengthened through the addition of stabilizers (such as foamed asphalt or asphalt emulsion, Portland cement, lime, or fly ash) prior to compaction during the FDR process. Many state DOTs have used FDR to successfully restore the structural capacity of pavements (*4-18*). For example, in 2011 the Virginia DOT (VDOT) used three recycling techniques (CIR, CCPR and FDR) all together in a strategically layered configuration to rehabilitate sections of Interstate-81 (*19*).

Studies of the structural properties of FDR materials have focused on the structural number and layer coefficients (*4, 7, 16, 18, 20, 21*) as most agencies continue to rely on empirical methods for new pavement and rehabilitation design. There is, however, a paradigm shift in the design of pavements as more state DOTs implement or plan to implement practices from AASHTO's *Mechanistic-Empirical Pavement Design Guide* (MEPDG) (*22*). The MEPDG recommends the use of the elastic modulus for characterizing the structural properties of materials for design purposes. Yet, only a few studies have investigated the elastic modulus of FDR base mixtures, and these have been limited to laboratory evaluations or accelerated pavement tests using heavy vehicle simulators (HVSs). The long-term, in situ properties and related performance of FDR mixtures have not received much attention, and several questions remain unanswered regarding the behavior of FDR mixtures over time. The objective of this research was therefore to evaluate changes in the elastic modulus over time and its sensitivity to temperature and seasonal effects for three road rehabilitation projects in Virginia constructed in 2008.

METHODOLOGY

Overview

To assess the feasibility of implementing the FDR technique in future road maintenance projects under the climatic conditions in the state of Virginia, three trial sections were constructed in 2008 using FDR with different stabilizing agents and were monitored over a 2-year period. The trial sections were located on State Routes (SRs) 40, 13, and 6 in Franklin, Powhatan, and Goochland Counties, respectively. Foamed asphalt (with 1% cement) and asphalt emulsion (66% residual asphalt with 33% water) were used as stabilizing agents for two adjacent sections in the SR 40 project, while Portland cement was used in sections along SR 13 and SR 6. The construction and reclamation processes were similar and have been documented by Diefenderfer and Apeagyei (*7*), together with a comprehensive description of the project sites. The authors assessed the performance of the projects at early ages. In the three projects, the existing pavement was pulverized to a predetermined depth, followed by the injection and mixing of the stabilizers, and subsequently compacted to contract specifications. A motor grader, a pad foot, and steel-wheeled and rubber-tired rollers were used for the construction. Table 1 summarizes the projects.

Assessment of FDR Layer Elastic Modulus

Diefenderfer and Apeagyei (*6, 7*) conducted a series of laboratory and field evaluations to assess the mechanical properties and structural capacity of the in-service FDR projects with field cores and two non-destructive techniques at early ages. As VDOT and many other state DOTs use empirical methods for pavement design, the authors conducted the structural evaluation in accordance with the 1993 AASHTO *Guide for Design of Pavement Structures (23)*, placing emphasis on investigating the structural number of the pavement and estimating layer coefficients for the FDR base layer through the AASHTO design equations. In this current research, however, emphasis was placed on evaluating the elastic modulus of individual layers as a mechanical property supported by the MEPDG.

Data Collection

Ground penetration radar (GPR) was used to estimate the layer thicknesses of the three FDR projects after construction. GPR has been used in several other projects to ensure specifications for thickness and material uniformity were met during construction. The results from the GPR testing were used in the deflection data analysis for subsequent evaluation of the structural capacity of the reclaimed sections.

A Dynatest Model 8000 falling weight deflectometer (FWD) was used in carrying out deflection measurements in both directions along the reclaimed pavements for all test sites. The equipment had nine sensors at radial distances of 0, 8, 12, 18, 24, 36, 48, 60, and 72 in. from the center of a load plate located in the wheelpath. At SR 40, testing was conducted at 100-ft intervals and at four load levels (6,000, 9,000, 12,000, and 16,000 lbf). The SR 13 and SR 6 sites were tested at 250-ft. intervals with three load levels (6,000, 9,000, and 12,000 lbf).

Analysis of Deflection Data

The collected deflection data were analyzed in accordance with the backcalculation methodology and procedures outlined in Von Quintus, Rao, and Irwin (*24*), together with ASTM D5858, *Standard Guide for Calculating In Situ Equivalent Elastic Moduli of Pavement Materials Using Layered Elastic Theory,* and *Guidance Notes on the Backcalculation of Layer Moduli and Estimation of Residual Life Using Falling Weight Deflectometer Test Data* (*25*). The analysis was performed using Dynatest ELMOD (Evaluation of Layer Modulus and Overlay Design) software, version 6. ELMOD provides two methods to backcalculate the elastic modulus, radius of curvature and deflection basin fit. These approximate methods are based on the Boussinesq equations and Odermark's method of equivalent thickness. The difference between the methods is that the deflection basin fit method runs additional iterations until the calculated deflections match the measured deflections to within the defined tolerance.

The pavement structure was modeled in ELMOD as a three-layered structure: a top HMA layer, an intermediate FDR base layer, and a subgrade layer at the bottom. The deflection data obtained from each FWD test were examined to ensure that data points with large fluctuations or inconsistencies such as non-decreasing deflections were removed prior to the backcalculation. The accumulated differences of the center deflections were used to divide the length of the test run into homogeneous sections in ELMOD. The accumulated difference (A.d.) at the i^{th} station was defined as follows:

$$A.d. = \sum \delta_i - i\mu$$

where

$\sum \delta_i$ = sum of deflections from the 1st station to the i^{th} station inclusively;

i = number of stations from δ_1 to δ_i inclusively;

μ = mean deflection of the test run.

A section was considered homogeneous when the cumulative differences continued in the same upward or downward trend. A significant change in trend marked a change in section. Figure 1 shows the output of the deflection data consistency checks and the resulting sectioning of SR 40.

The thickness of each layer in a homogeneous section was then input into ELMOD from the results of the GPR testing. The Poisson ratios selected for the pavement layers were within the range of typical values recommended in ASTM D5858 and other literature. A value of 0.35 was used for the HMA layer and subgrade. Values of 0.35 and 0.26 were used for the bitumen-stabilized and cement-stabilized bases respectively (*26, 27*).

ELMOD carries out the backcalculation using the following procedure (*25, 28*):

1. The program uses the Boussinesq equations to calculate the surface modulus from the surface deflections. The surface modulus, i.e., the weighted mean modulus of a semi-infinite space, at a certain distance r gives a rough estimate of the modulus at the same equivalent depth $z = r$. Under the condition that the subgrade behaves as a linear elastic semi-infinite space, the surface modulus should be the same at varying distances. Based on this, the subgrade

modulus is estimated using the outer deflections as these are almost entirely controlled by the subgrade.

2. The accuracy of results from the backcalculations is usually impacted by the presence of a stiff layer and subgrade moisture. The presence of a stiff layer at some depth beneath the subgrade is checked by evaluating the change in modulus with varying distances from the center of the load plate by calculating the surface modulus, E_0:

$$E_0(0) = \frac{2 \cdot a \cdot \sigma_0 \cdot (1-\mu^2)}{d_0(0)} \tag{1}$$

$$E_0(r) = \frac{a \cdot \sigma_0 \cdot (1-\mu^2)}{r \cdot d_0(r)}, \qquad (r > 2a) \tag{2}$$

where
$E_0(r)$ = surface modulus at distance r;
μ = Poisson's ratio of the subgrade;
σ_0 = uniform stress on the plate;
a = load plate radius;
r = distance from the center load;
$d_0(r)$ = surface deflection at distance r.

3. If a stiff layer is not found, a check is performed for non-linearity due to high moisture content resulting from the presence of a ground water or a wet layer. The coefficients C and n are obtained from the following equation:

$$E_0 = C \cdot \left({}^{\sigma_1}\!/_{\sigma} \right)^n \tag{3}$$

where
E_0 = surface modulus;
σ_1 = major principal stress;
σ = reference stress (usually 160 MPa);
C = constant;
n = negative constant.

C decreases linearly with the increase in moisture content, and n usually measures the non-linearity; the subgrade is linear elastic when n is zero, with the non-linearity becoming more evident as n decreases.

4. The moduli of the HMA layer and FDR base are then determined through a series of iterations using the center deflection and the shape of the basin under the load plate.
5. The subgrade modulus is then adjusted according to the estimated stress level under the load center. The outer deflections are checked and additional iterations are carried out if necessary.
6. The calculated and measured deflection profiles are then matched with the percentage difference (root mean square [RMS]) between the calculated and measured values reported.

The iterations are performed with the objective of minimizing the RMS as the convergence criteria.

A total of 26 FWD files with 36 stations were analyzed for SR 40, 26 FWD files with 38 stations for SR 13, and 37 FWD files with 21 stations for SR 6. A total of 16,205 deflection basins were analyzed for this study.

Evaluating Long-Term Performance
Deterioration models were developed and analyzed to assess the long-term performance of the projects. The critical condition index (CCI), along with the pavement age were obtained from the VDOT Pavement Management System (PMS). VDOT currently uses the generalized form:

$$CCI = 100 - \exp\left(a + b \times c^{\log\left(\frac{1}{Age}\right)}\right) \tag{4}$$

where
CCI = Critical Condition Index (100 when pavement age is zero)
a, b, c = model coefficient
Age = age of pavement after treatment was last applied

The model was calibrated with the CCI and age data for the sections to obtain numerical values for the coefficients a, b, and c. The values of a, b, and c for primary roads managed by VDOT are 15.32, 15.38 and 1.11. These were used as initial values for the FDR sections in the calibration process.

RESULTS

Deflection Data Analysis
FWD testing was undertaken for all three recycling projects starting at approximately 3 weeks after reclamation at varying intervals for 28 months. The deflection data files (the same as used by Diefenderfer and Apeagyei (*6, 7*) were analyzed as a three-layered structure. Acceptable results were defined as an RMS less than or equal to 3%, with the range of backcalculated moduli for each layer falling within an acceptable range for each layer type and category based on the default range of values included in the MEPDG (*24*).

Elastic Modulus Trends

SR 40

Strength of the Base Layer Figure 2a shows the results of the average moduli for the FDR base layer. The moduli for the foamed asphalt section, in both directions, were in the approximate range of 133 to 638 ksi (920 to 4,400 MPa). The moduli for the asphalt emulsion sections were lower and within the approximate range of 36 to 485 ksi (250 to 3,350 MPa). Quick and Guthrie (*29*) reported low moduli results for deflection tests undertaken in the month of June 2010 (89.6 ksi) as observed for this study.

For both FDR treatment types, there was a general increase in the moduli up to the sixth month after reclamation. Strong seasonal variations were evident in both sections, with the foamed asphalt and asphalt emulsion sections attaining the highest and lowest moduli in the winter and summer seasons.

Subgrade Strength and Support The purpose of the FDR base layer is to protect the subgrade from damage. It is therefore key to report how well each of the treatments served this purpose. Figure 2b shows the results of the subgrade moduli for the bitumen-stabilized sections. The moduli were higher for the foamed asphalt sections, with a range of 12 to 28 ksi (85 and 190 MPa). The range of the moduli for the asphalt emulsion sections was between 8 and 16 ksi (50 to 110 MPa). There was no evidence of bedrock in these sections. The trends were impacted by seasonal variations. For both sections, the highest subgrade modulus was in the winter season, which may be representative of the frozen conditions underneath the pavement structure. The lowest moduli for both projects were in the spring, which represents the end of freeze-thaw cycles. However, lower results were reported for the emulsion asphalt sections than the foamed asphalt sections during the same period, an indication of higher susceptibility to moisture effects. This observation is confirmed by the subgrade non-linearity constants, C (moisture content) and n (measure of non-linearity), obtained from the backcalculation. C values of 11.3 ksi and 16.7 ksi for the asphalt emulsion and foamed asphalt, respectively, indicate a higher moisture content in the asphalt emulsion section. The addition of 1% cement in the foamed asphalt base mixture as a chemical additive may have contributed to the reduced impact of moisture on those sections. Generally, the area around the site tended to drain towards the eastern end of the emulsion section with the roadway rising to either side of the section. It is also possible that the increased moduli of the overlaying layers may have contributed to the increased moduli for the foamed sections as a result of the backcalculation process.

SR 13 and SR 6

Strength of the Base Layer Figure 3a shows the average moduli reported from the SR 13 and SR 6 FDR projects stabilized with Portland cement. The moduli for the SR 13 project were higher, within the range of 550 to 1,045 ksi (3,800 and 7,200 MPa), than the moduli for SR 6, which ranged from 377 to 776 ksi (2,600 to 5,350 MPa). The impact of seasonal variations was not evident in either of the projects even though the modulus fluctuated with time over the analysis period. Additional results from the deflection data analysis revealed very non-linear conditions in the pavement structure for the SR 6 project, indicating high moisture content in the subgrade for the majority of the sections analyzed. The section also had a higher annual average daily traffic (AADT) than the SR 13 section. These two factors may have contributed to the low stiffness results compared with SR 13 and requires further investigation. The source of the high moisture content was not investigated in this study.

Subgrade Strength/Support Figure 3b shows the results of the moduli of the subgrade for the Portland-cement stabilized projects. The backcalculated moduli were generally higher in the SR

13 project, ranging from 17 to 35 ksi (120 to 240 MPa), than the results for the SR 6 project, which were in the range of 10 to 20 ksi (70 to 140 MPa).

The results were similar to the subgrade resilient modulus values obtained by Diefenderfer and Apeagyei (*7*). The impact of a seasonal trend was not significant, and the moduli remained almost stable over the period analyzed for both projects. The difference in the stiffness results for the two projects may be explained by the moisture contents in the subgrade. The SR 6 project showed higher moisture content during the non-linearity check from the backcalculations. Average C values of 26.9 ksi and 14.2 ksi were obtained for SR 13 and SR 6 respectively.

Temperature Sensitivity

SR 40

Asphalt materials are sensitive to changes in temperature. Figure 4a shows the variations in moduli of the FDR base layer with changes in temperature for the SR 40 project. The temperature used in the analysis was the pavement mid-depth temperature calculated from the BELLS3 equation using the average of the previous day high and low temperature. The moduli for both sections generally decreased with increasing temperature (change in stiffness over time considered). The variation, however, is greater for the emulsion asphalt section compared with the foamed asphalt sections. This may be a result of the 1% cement addition to the foamed asphalt sections as the cement reduces the viscoelastic properties. If this relationship is true, then the time of the day at which field evaluations are undertaken must be taken into account as this will impact the results obtained for both the AC overlay and FDR base layers.

SR 13 and SR 6

The relationship between the backcalculated moduli and temperature is shown in Figure 4b for the Portland cement-stabilized projects. As expected, the effect of temperature on the modulus results does not seem to be significant as the modulus does not change with variation in temperature for both projects. This result confirms that the addition of cement as the stabilizing agent not only improves the strength of the base layer but also reduces the effect of temperature on the stiffness of the materials.

Time-Dependent Variations

SR 40

Figure 5a shows the variation in the average eastbound and westbound moduli for the SR 40 projects over time. For both the foamed asphalt and asphalt emulsion sections, the moduli increased significantly over the first 2-year period following construction. The FDR process incorporating foamed asphalt with 1% cement resulted in a stiffer base mixture than reclamation with asphalt emulsion stabilization. For both sections, a sharp increase in the moduli was evident in the first 4 months after reclamation before leveling out around a long-term average of 174 and 420 ksi (1,200 and 2,900 MPa) for asphalt emulsion and foamed asphalt, respectively, in the last 18 months. Within the first 4 months, the foamed asphalt section achieved a stiffness of 362 ksi

(2,500 MPa) close to its long-term average, from an initial stiffness of 110 ksi (758 MPa). Within the same period, the emulsion asphalt section achieved a stiffness of 115 ksi (790 MPa) close to its long-term average, from an initial stiffness of 24 ksi (165 MPa). This confirms the slower curing time for emulsion asphalt widely reported in the literature (*29-30*). The authors believe the addition of cement may have contributed significantly to the curing time and overall higher stiffness of the foamed asphalt section.

A logarithmic trend line was added to predict the variation of stiffness of the FDR base materials during the curing process. Due to the temperature sensitivity of the bitumen-treated bases observed from the modulus-temperature analysis, two data points representing the highest (June 2009) and lowest (January 2009) temperatures of the year were eliminated from analysis to reduce the effect of temperature on the model.

As the trend does not show any significant damage within the analysis period, it was not possible to develop equations to predict long-term damage as applied in the MEPDG for rehabilitation design. Diefenderfer and Apeagyei (*6*) used similar logarithmic regression equations to predict the variations in the structural layer coefficient over time. The following regression equations with their corresponding goodness-of-fit (R^2) values were obtained for the SR 40 project:

Foamed asphalt: $y = 110.7\ln(x) + 94.4$ ($R^2 = 0.53$)
Asphalt emulsion: $y = 57.71\ln(x) + 10.5$ ($R^2 = 0.83$)

SR 13 and SR 6

Figure 5b shows the variation of the average eastbound and westbound moduli for the Portland cement-stabilized projects over time. The long-term strength gain for the SR 13 project was similar to that for the SR 6 project. The moduli of the SR 6 project at the end of the 2 years were comparable to those obtained for the foamed asphalt (plus 1% cement) sections from the SR 40 project even though the initial stiffness was higher for the SR 6 project. Similar logarithmic regression equations were developed to predict the moduli as a variation of time. The regression equations and corresponding R^2 values are presented below:

Portland cement (SR 13): $y = 123.72\ln(x) + 640.76$ ($R^2 = 0.75$)
Portland cement (SR 6): $y = 94.018\ln(x) + 416.78$ ($R^2 = 0.80$)

A prediction relationship using the logarithmic regression through all points for SR 13 and SR 6 over time was developed:

Portland cement: $y = 127.14\ln(x) + 521.42$ ($R^2 = 0.50$)

("*x*" and "*y*" in the logarithmic regression equations are the pavement age and elastic modulus with units months and ksi respectively).

Long-Term Performance

Figure 6 shows predictions of the functional performance of the FDR projects over time. The model coefficients calibrated for the CCI curves (Equation 4) are as follows;

Foamed asphalt: $a = 21.86$, $b = -22.06$, $c = 1.521$ ($SSE = 4.13$)

Asphalt emulsion: a = 22.01, b = -29.02, c = 1.652 (SSE = 3.89)
Portland cement: a = 22.13, b = - 21.95, c = 1.193 (SSE = 7.97)

A pavement with a CCI value of 100 has no discernible distresses, while pavement sections with a CCI value of zero indicates a heavily deteriorated pavement condition. Typically, VDOT considers pavement sections with a CCI value below 30 (very poor) as deficient, requiring further evaluation for corrective action. The analysis shows that all three projects are comparable in terms of performance. The bitumen-stabilized project remained in excellent condition for a longer period of time compared to the cement projects. The cement projects however, were projected to be approximately 4 years more durable than the bitumen-stabilized projects using the CCI of 30 as a trigger for rehabilitation.

SUMMARY

The strength of the stabilized FDR bases for all three projects varied over time. The FDR sections having bitumen-stabilized bases showed more sensitivity to seasonal changes than those having Portland cement-stabilized bases. The cement-stabilized bases were stiffer and provided better support to the subgrade than the bitumen-treated bases. Based on the Critical Condition Index (CCI) for each recycling project from before and after the reclamation process (obtained from the VDOT PMS and shown in Table 1), the functional condition of the pavements was significantly improved as the pavements were restored to above 98 CCI 1 year after reclamation.

Of the three projects, the bitumen-stabilized sections showed the most sensitivity to temperature effects. However, the section having foamed asphalt (plus 1% cement) was less sensitive compared with the section having an emulsion-stabilized base. The Portland cement-stabilized bases showed little to no sensitivity to temperature, leading to the conclusion that the addition of cement as a chemical additive compared with other stabilizers not only improves the strength but also reduces the temperature sensitivity of the material. The analysis also suggested that the field evaluations of the structural capacity of bitumen-stabilized bases must consider temperature. If evaluations are undertaken only once after construction, the results may underestimate the actual condition if conducted at a high temperature for bitumen-stabilized projects.

The stiffness of the FDR bases in all three projects increased over time. FDR bases stabilized with Portland cement had the highest initial stiffness, taking about 2 months to attain the average maximum strength over the 2-year analysis period. The bitumen-stabilized bases took a longer time to cure, with asphalt emulsion showing lower initial stiffness and lengthier time to gain maximum strength than the foamed-asphalt-stabilized bases. This trend may be, in part, due to the softer PG 58-22, as suggested by Diefenderfer and Apeagyei (*6, 7*). Also, the presence of cement may have slightly improved the curing time for the foamed asphalt sections. Pavement design engineers should note this curing trend for FDR projects and implement measures to improve short-term and overall long-term performance of these projects. Increasing the design strength or design traffic (AADT), exploring mix designs with combined stabilizing using emulsion asphalt bases with cementitious materials in appropriate contents, delaying the opening to traffic, or limiting usage to light traffic in the short term may be measures to consider.

The regression equations developed to predict the evolution of the elastic modulus with time for the three projects can be used to predict short-term strength gains. The equations predict an ever increasing strength with passage of time.

The preliminary condition modeling suggested that the Portland cement-stabilized projects had approximately four years of remaining service life compared to the bitumen-stabilized sections using a CCI of 30 as a trigger for rehabilitation. The durability and long-term condition of the bitumen-stabilized sections were similar even though the foamed asphalt (plus 1% cement) sections were significantly stiffer than the asphalt emulsion section. A longer time analysis is needed to confirm these findings because of the high variability of the CCI measurements.

CONCLUSIONS

The following conclusions and recommendations were made based on the results of this study.

- The effect of temperature should not be overlooked in the evaluation of the structural capacity of bitumen-stabilized FDR projects. The authors propose further work on developing more robust temperature models to improve the current practice.
- A general enhancement in the material properties from the initial strength after construction was noticed with time in all the stabilized FDR projects studied. These time-dependent variations in strength are significant for FDR projects and must be considered in the mechanistic-empirical analysis.
- The addition of cement as a primary stabilizing agent or as a chemical additive in bitumen-stabilized FDR bases not only increases strength but also reduces the effects of temperature on the moduli for FDR projects.
- Based on the stiffness trends of all three projects and their overall comparable performance in the long term, design moduli of approximately 170 ksi and 720 ksi for asphalt emulsion and Portland cement respectively are recommended for use in MEPDG for the FDR layer of projects with similar stabilizing agent content. When foamed asphalt, with Portland cement as a chemical additive is used, 450 ksi is recommended for design.
- Condition modeling was used to estimate the long-term performance of the projects and the analysis suggested that FDR projects with stiffer base materials do not necessarily perform better even though they may have slightly longer remaining service life in this research.

ACKNOWLEDGEMENT

The authors acknowledge Bent Lund of Dynatest Denmark A/S for providing a licensed copy of ELMOD version 6 for the analysis of deflection data in this research.

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LIST OF TABLES AND FIGURES

TABLE 1 Summary of VDOT's FDR Demonstration Project

	SR 40	SR 40	SR 13	SR 6
County	Franklin	Franklin	Powhatan	Goochland
Relative Milepost	21.70 – 21.97	21.97 – 22.24	16.25 – 19.93	67.48 – 71.11
Stabilizing Agent	Foamed Asphalt	Asphalt Emulsion	Portland Cement	Portland Cement
Stabilizing Agent Content[a]	2.7%	3.5%	5.0%	5.0%
Chemical Additive	1% cement	na	na	na
AADT[b] (% Trucks)	4,400 (4%)	4,400 (4%)	2,300 (5%)	3,900 (7%)
Approx. Project Length (Lane-Miles)	0.5	0.5	7.4	7.3
CCI[c]: Before Intervention	41/100	41/100	58/100	47/100
CCI: 1 Year after Intervention	99/100	99/100	100/100	98/100
AC[d] Overlay Thickness (in.)	2.4	2.4	3.6	3.6
FDR Base Thickness (in.)	9.4	9.8	9.2	9.1
Reclamation Dates	May 13–15, 2008	May 19–22, 2008	June 18–July 21, 2008	July 21–August 7, 2008
Time from Reclamation to AC Overlay	3 weeks	3 weeks	1 week	1 week

[a]Percentage by Weight [b]Annual Average Daily Traffic [c]Critical Condition Index [d]Asphalt Concrete na = not applicable

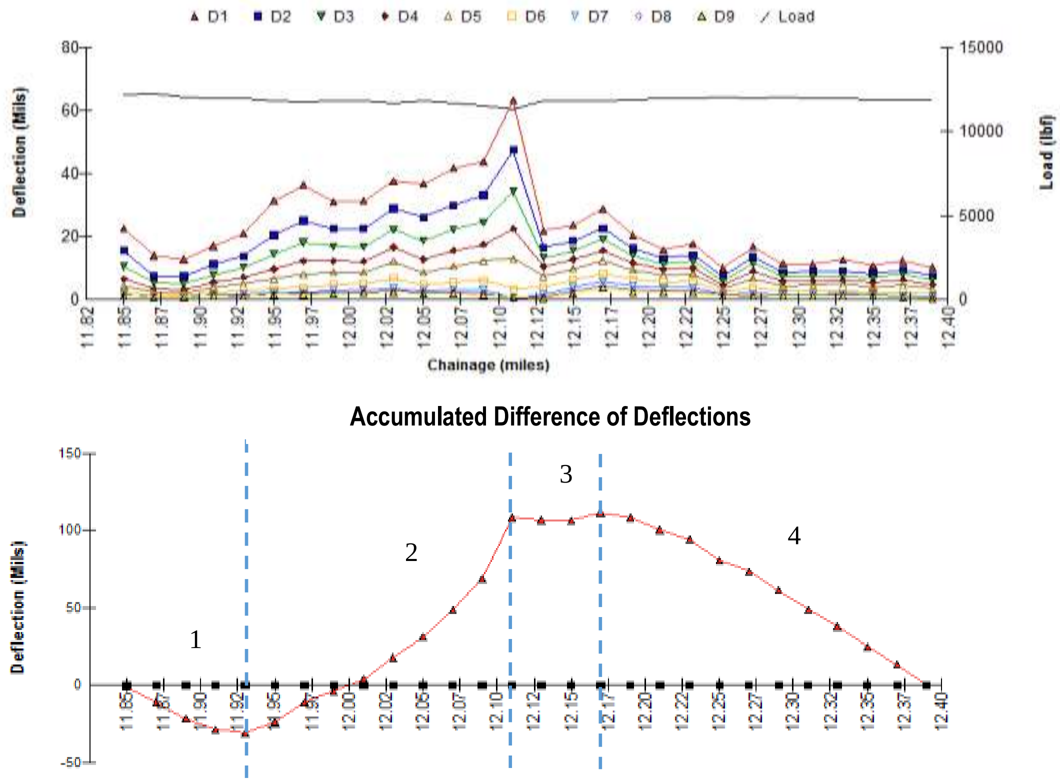
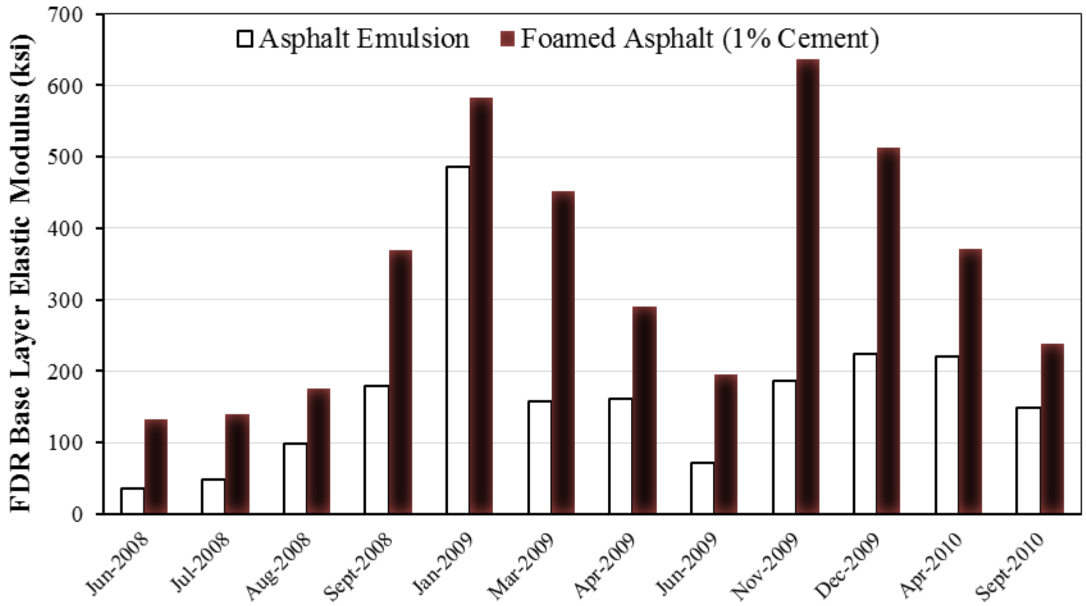
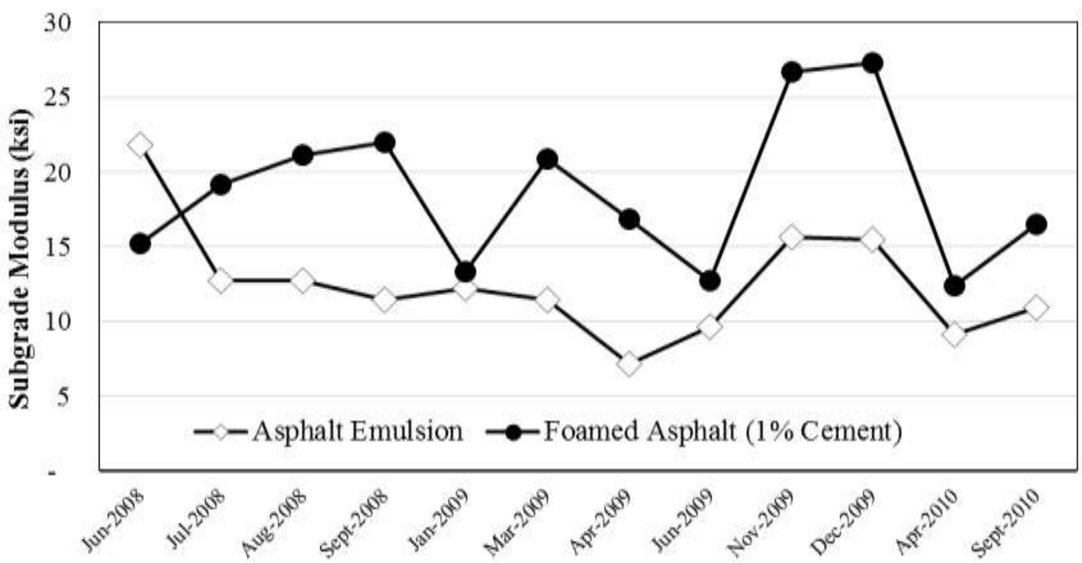


FIGURE 1 Deflection data checks and sectioning of SR 40.

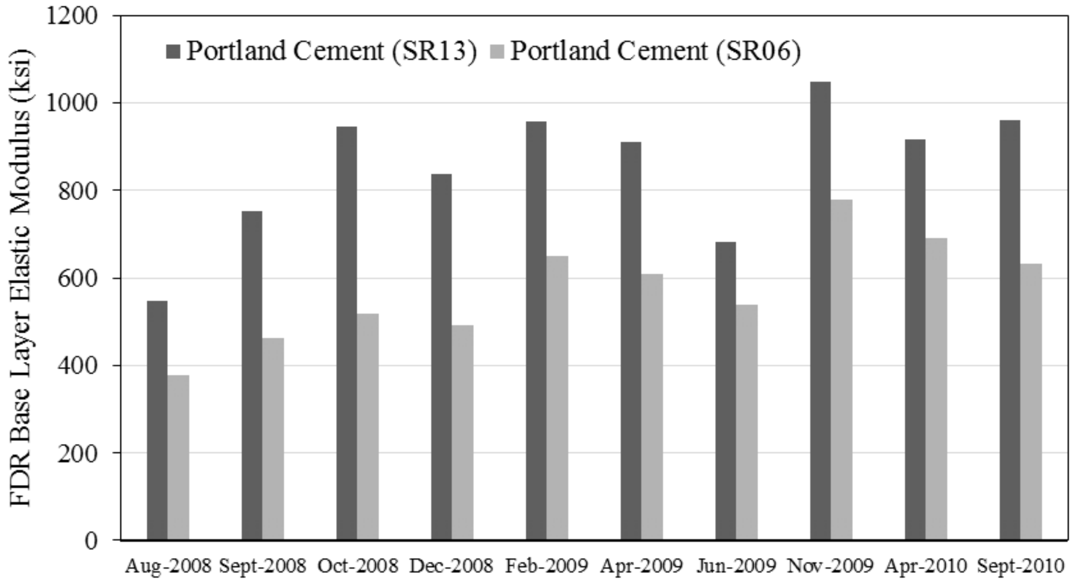


(a)



(b)

FIGURE 2 Comparison of backcalculated moduli for SR 40 project: (a) bitumen-stabilized FDR layer and (b) subgrade.



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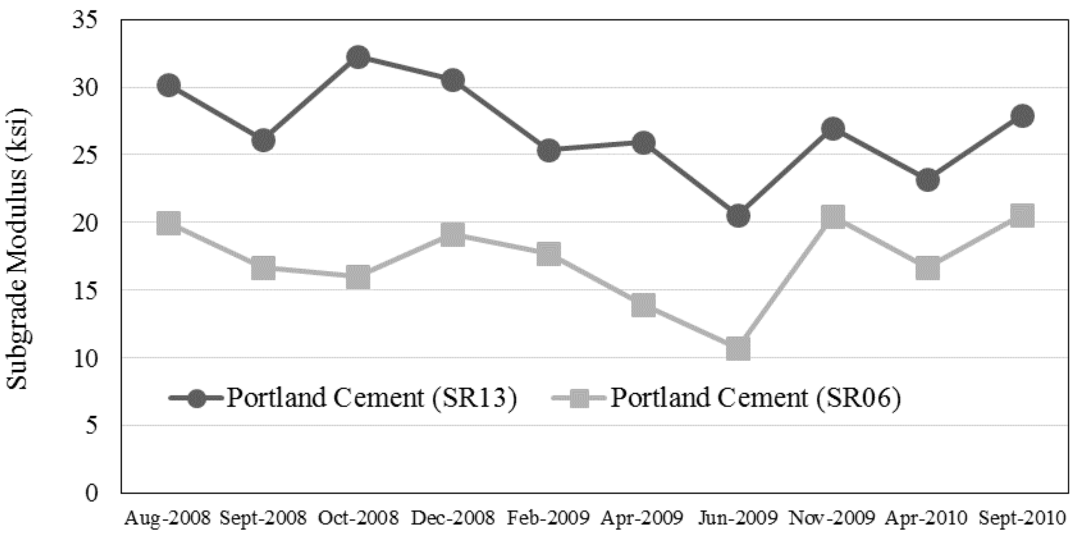
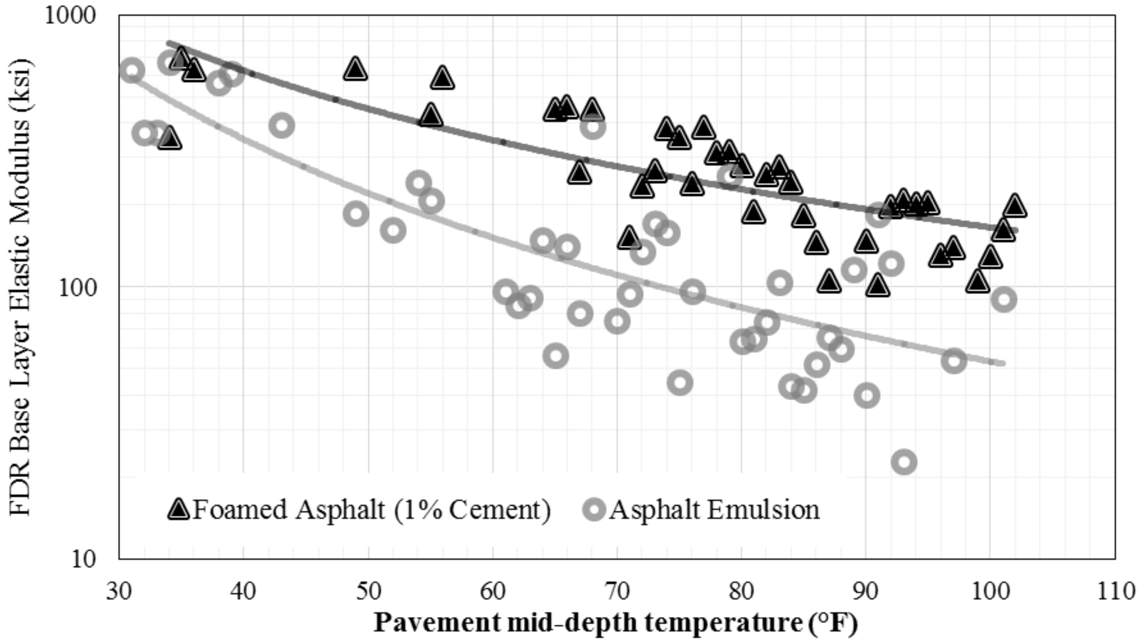
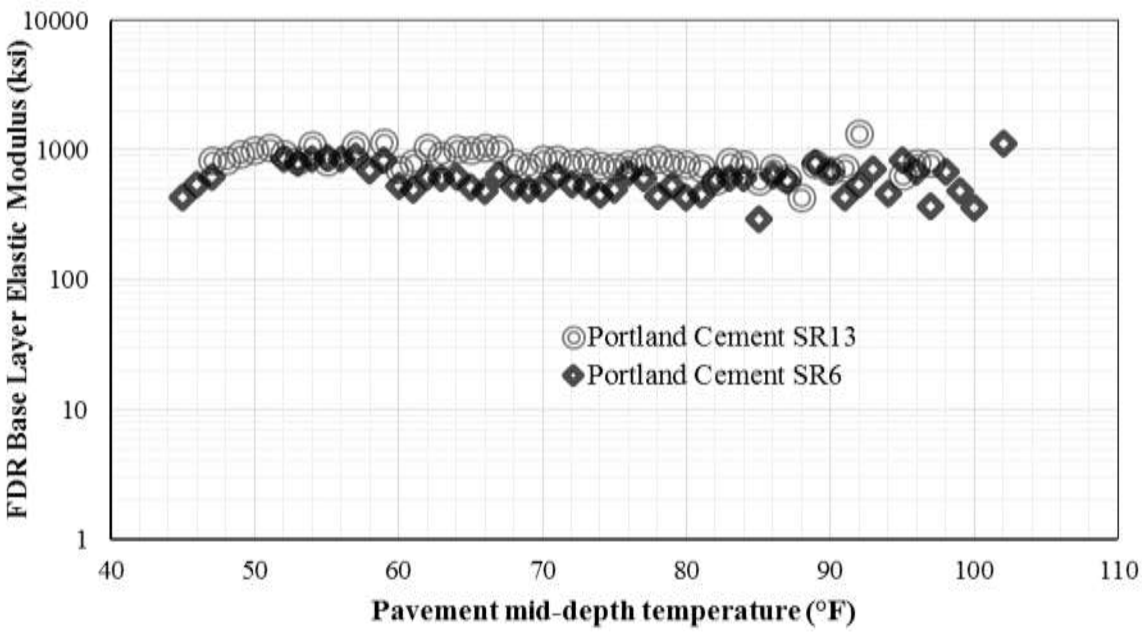


(b)

FIGURE 3 Comparison of backcalculated moduli for SR 13 and SR 6 projects: (a) cement-stabilized FDR layer and (b) subgrade.

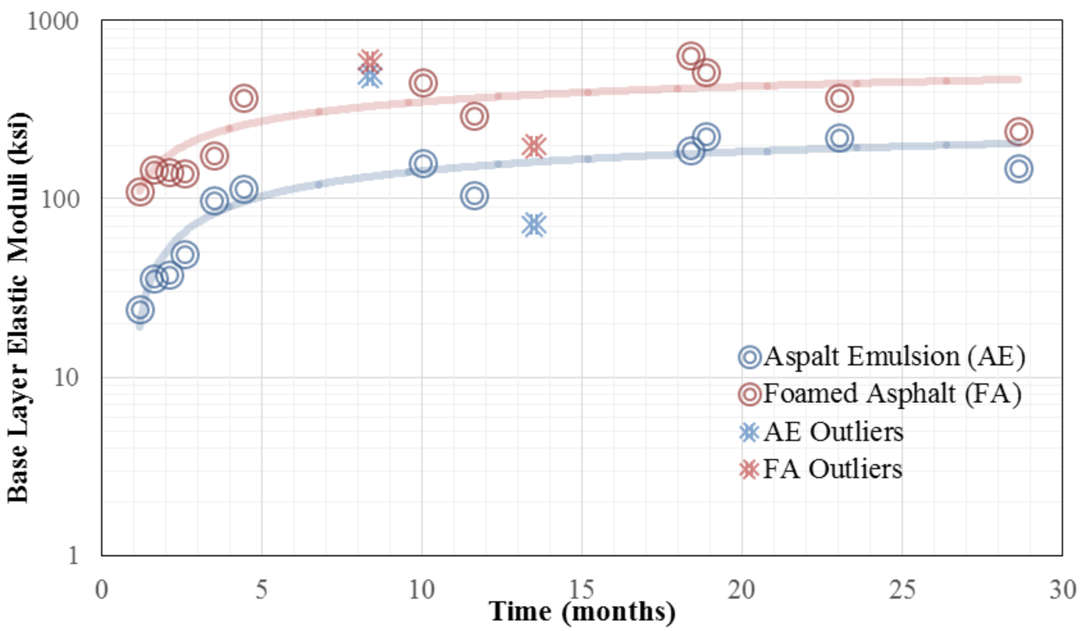


(a)

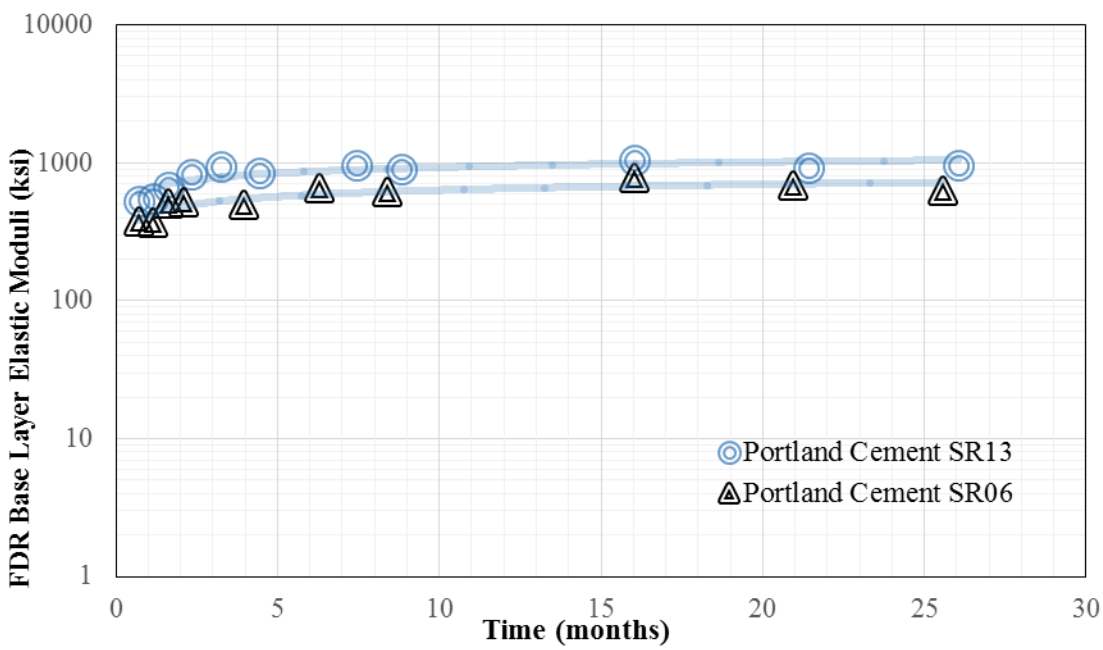


(b)

FIGURE 4 Variation of elastic modulus with temperature for (a) bitumen-stabilized FDR base layers and (b) cement-stabilized FDR base layers.



(a)



(b)

FIGURE 5 Evolution of backcalculated elastic modulus with time for (a) bitumen-stabilized FDR bases and (b) cement-stabilized FDR bases.

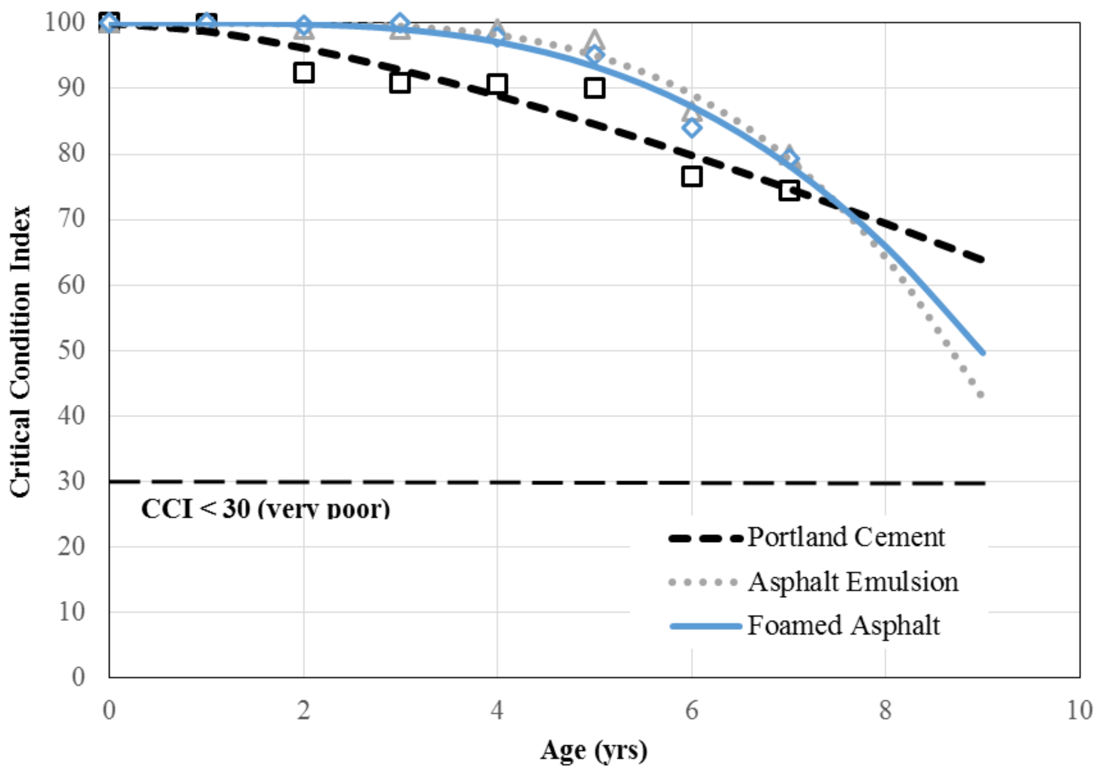


Figure 6: CCI curves showing pavement deterioration with age for the FDR projects