

GEORGIA DOT RESEARCH PROJECT 18-22

FINAL REPORT

**FULL DEPTH PAVEMENT RECLAMATION:
PERFORMANCE ASSESSMENT AND
RECOMMENDATIONS FOR BEST
PERFORMANCE**



**OFFICE OF PERFORMANCE-BASED
MANAGEMENT AND RESEARCH**

**600 WEST PEACHTREE STREET NW
ATLANTA, GA 30308**

TECHNICAL REPORT DOCUMENTATION PAGE

1. Report No.: GDOT-GA-20-1822	2. Government Accession No.: N/A	3. Recipient's Catalog No.: N/A	
4. Title and Subtitle: Full Depth Pavement Reclamation: Performance Assessment and Recommendations for Best Performance		5. Report Date: September 2020	
		6. Performing Organization Code: N/A	
7. Author(s): Jayhyun Kwon (PI), Ph. D., P.E. (https://orcid.org/0000-0001-7084-7942); Youngguk Seo (coPI), Ph. D.; Adam Kaplan (coPI), Ph. D.; Jidong Yang (coPI), Ph. D., P.E.		8. Performing Organization Report No.: 18-22	
9. Performing Organization Name and Address: Kennesaw State University Civil and Environmental Engineering 655 Arntson Drive, Marietta, GA 30060 Phone: (470) 578-5080 E-mail: jkwon9@kennesaw.edu		10. Work Unit No.: N/A	
		11. Contract or Grant No.: KSU431509-190023 GDOT PI#0015842	
12. Sponsoring Agency Name and Address: Georgia Department of Transportation Office of Performance-based Management and Research 600 West Peachtree St. NW Atlanta, GA 30308		13. Type of Report and Period Covered: Final; August 2018-September 2020	
		14. Sponsoring Agency Code: N/A	
15. Supplementary Notes: Conducted in cooperation with the U.S. Department of Transportation, Federal Highway Administration.			
16. Abstract: This study investigates variability in the FDR base layer from the results of field and laboratory data of a reconstruction project. The results of the UCS tests show 7-day strengths are more stable with less variability across the sites than those of 3 days. Deflection data analysis results also show that the FDR layer provides more uniform stiffness as curing continues. Mechanistic analysis results indicate both the FDR base thickness and strength have a significant influence on the predicted pavement responses. Some of the pertinent conclusions from this study are: ▪ It is recommended that GDOT specifications be modified to include pre-mix in specification 315 section 3.03. ▪ A stricter control on the cement spread rate and the treatment depth is necessary to reduce variability in strength. ▪ Deflection testing with either LWD or FWD should be performed to monitor the stiffness characteristics (or strength gains) of the FDR base during construction. ▪ It is recommended that GDOT develop CTA fatigue cracking model calibration coefficients to model FDR base pavement sections using Pavement ME Design (PMED) software.			
17. Keywords: Full Depth Reclamation, Mechanistic analysis, Unconfined compressive strength, Spatial variability, Deflection test		18. Distribution Statement: No Restrictions	
19. Security Classification (of this report): Unclassified	20. Security Classification (of this page): Unclassified	21. No. of Pages: 79	22. Price: Free

GDOT Research Project 18-22

Final Report

FULL DEPTH PAVEMENT RECLAMATION: PERFORMANCE
ASSESSMENT AND RECOMMENDATIONS FOR BEST PERFORMANCE

By

Jayhyun Kwon, Ph.D., P.E.

Assistant Professor – Department of Civil and Environmental Engineering

Youngguk Seo, Ph.D.

Assistant Professor – Department of Civil and Environmental Engineering

Adam Kaplan, Ph.D.

Associate Professor – Department of Civil and Environmental Engineering

Jidong Yang, Ph.D., P.E.

Associate Professor – Department of Civil and Environmental Engineering

Kennesaw State University Research and Service Foundation

Contract with

Georgia Department of Transportation

In cooperation with

U.S. Department of Transportation

Federal Highway Administration

September 2020

The contents of this report reflect the views of the authors, who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views of the Georgia Department of Transportation or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

SI* (MODERN METRIC) CONVERSION FACTORS				
APPROXIMATE CONVERSIONS TO SI UNITS				
Symbol	When You Know	Multiply By	To Find	Symbol
LENGTH				
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
AREA				
in ²	square inches	645.2	square millimeters	mm ²
ft ²	square feet	0.093	square meters	m ²
yd ²	square yard	0.836	square meters	m ²
ac	acres	0.405	hectares	ha
mi ²	square miles	2.59	square kilometers	km ²
VOLUME				
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft ³	cubic feet	0.028	cubic meters	m ³
yd ³	cubic yards	0.765	cubic meters	m ³
NOTE: volumes greater than 1000 L shall be shown in m ³				
MASS				
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
TEMPERATURE (exact degrees)				
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
ILLUMINATION				
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m ²	cd/m ²
FORCE and PRESSURE or STRESS				
lbf	poundforce	4.45	newtons	N
lbf/in ²	poundforce per square inch	6.89	kilopascals	kPa
APPROXIMATE CONVERSIONS FROM SI UNITS				
Symbol	When You Know	Multiply By	To Find	Symbol
LENGTH				
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
AREA				
mm ²	square millimeters	0.0016	square inches	in ²
m ²	square meters	10.764	square feet	ft ²
m ²	square meters	1.195	square yards	yd ²
ha	hectares	2.47	acres	ac
km ²	square kilometers	0.386	square miles	mi ²
VOLUME				
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m ³	cubic meters	35.314	cubic feet	ft ³
m ³	cubic meters	1.307	cubic yards	yd ³
MASS				
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
TEMPERATURE (exact degrees)				
°C	Celsius	1.8C+32	Fahrenheit	°F
ILLUMINATION				
lx	lux	0.0929	foot-candles	fc
cd/m ²	candela/m ²	0.2919	foot-Lamberts	fl
FORCE and PRESSURE or STRESS				
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in ²

* SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)

TABLE OF CONTENTS

CHAPTER 1. LITERATURE REVIEW	5
INTRODUCTION	5
FIELD SAMPLING AND MIX DESIGN	7
Field Sampling	7
Mix Design.....	9
QUALITY ASSURANCE AND QUALITY CONTROL.....	14
Microcracking.....	15
Opening to Traffic	15
TECHNIQUES FOR FDR ROADWAY CONSTRUCTION.....	16
In-situ Tests for Pavement Structural Capacity Evaluation	16
Ground Penetrating Radar (GPR).....	19
CHAPTER 2. ASSESSMENT OF GDOT’S FULL DEPTH RECLAMATION (FDR)	
PROJECT	23
INTRODUCTION	23
CONSTRUCTION ACTIVITIES	25
UNCONFINED COMPRESSITVE STRENGTH.....	32
DEFLECTION DATA ANALYSIS – LIGHT WEIGHT DEFLECTOMETER AND	
FALLING WEIGHT DEFLECTOMETER	36
FDR THICKNESS MEASUREMENT USING GROUND PENETRATING RADAR....	40
CHAPTER 3. MECHANISTIC SENSITIVITY ANALYSIS.....	46
AASHTOWare PAVEMENT ME DESIGN (PMED).....	46
LAYERED ELASTIC ANALYSIS	52
CHAPTER 4. RECOMMENDATIONS.....	61
ACKNOWLEDGEMENTS.....	63
REFERENCES	64

LIST OF FIGURES

Figure 1. Illustration. Full-Depth Reclamation (FDR) process [3].	6
Figure 2. Photos. SR 70 pavement condition prior to rehabilitation.	23
Figure 3. Map. Project location.	24
Figure 4. Photo. Asphalt milling.	25
Figure 5. Photo. Pre-mix.	25
Figure 6. Photos. Dry cement placement and cement spread rate check.....	26
Figure 7. Photos. Pulverization and compaction of the FDR base.	27
Figure 8. Photos. UCS sample preparation.....	28
Figure 9. Photos. GPR and LWD testing on test section.....	28
Figure 10. Photo. Density, moisture content and thickness verification (performed by GDOT)..	29
Figure 11. Photos. Placement of the chip seal layer.....	30
Figure 12. Photo. FDR core sampling for GPR calibration.....	31
Figure 13. Photos. Asphalt concrete surface placement (10/3/2018).	31
Figure 14. Photos. UCS test.	32
Figure 15. Graphs. LWD results – modulus and deflection changes over time.	37
Figure 16. Graph. FWD backcalculated subgrade moduli.	38
Figure 17. Graph. FWD results - ISM changes over time.....	39
Figure 18. Photos. GPR assembly.	42
Figure 19. Photo. GPR profile on the first day of FDR construction.	44
Figure 20. Photos. Time travel signals at station 131+00.	45
Figure 21. Chart. Modulus and unconfined compressive strength conversion chart – AASHTO [50].	48
Figure 22. Graphs. Predicted International Roughness Index (IRI).	50
Figure 23. Graphs. Predicted permanent deformation.....	51
Figure 24. Graphs. Predicted horizontal strain at the bottom AC.	55
Figure 25. Graphs. Predicted vertical strain at the bottom AC.....	56
Figure 26. Graphs. Horizontal stress distribution with depth (4 in. AC and 8 in. base).....	58
Figure 27. Equation. Fatigue cracking in chemically stabilized mixture [56].....	59
Figure 28. Graph. Number of repetitions to fatigue cracking of cement treated base.....	60

LIST OF TABLES

Table 1. Field sampling and site investigation methods.....	8
Table 2. Mix design methods used by various state highway agencies.....	13
Table 3. Construction criteria used by various state highway agencies.	14
Table 4. Daily cement spread rates.....	26
Table 5. Roadway compaction summary (% compaction required = 98%).	30
Table 6. Paired sample means by site.....	33
Table 7. Paired t test results.....	34
Table 8. Analysis of Variance – strength comparison by sections.....	35
Table 9. Comparison of 7-day strengths with core sample test.....	36
Table 10. Statistical analysis results – deflection test.	40
Table 11. Depth vs resolution.....	41
Table 12. Daily GPR survey schedule.....	43
Table 13. Pavement design input parameters (PMED Version 2.5.4).....	47
Table 14. AASHTOWare Pavement ME Design (PMED) analysis summary.....	49
Table 15. Linear Elastic Analysis input parameters.	52
Table 16. WinJULEA Linear Elastic Analysis summary.....	54

LIST OF ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
AC	Asphalt Concrete
ANOVA	Analysis of Variance
ASTM	American Society for Testing and Materials
COV	Coefficient of Variation
CTB	Cement Treated Base
DCP	Dynamic Cone Penetrometer
FAA	Federal Aviation Administration
FDR	Full Depth Reclamation
FWD	Falling Weight Deflectometer
GDOT	Georgia Department of Transportation
GPR	Ground Penetrating Radar
HMA	Hot Mix Asphalt
ISM	Impulse Stiffness Modulus
LEA	Layered Elastic Analysis
LWD	Light Weight Deflectometer
MDD	Maximum Dry Density
NCHRP	National Cooperative Highway Research Program
OMC	Optimum Moisture Content
PCA	Portland Cement Association
PCC	Portland Cement Concrete
PMED	Pavement ME Design
QA	Quality Assurance
RAP	Reclaimed Asphalt Pavement
SCDOT	South Carolina Department of Transportation
SR	State Route
UCS	Unconfined Compressive Strength
VDOT	Virginia Department of Transportation

EXECUTIVE SUMMARY

State of Practice

Full Depth Reclamation (FDR) with cement stabilization in pavement rehabilitation could improve pavement structural capacity with reduced costs for materials and hauling. State Highway Agencies (SHAs) have developed mix design, construction requirements, and quality acceptance criteria for FDR construction. A typical FDR thickness ranges from 6 to 9 inches. The proper amount of water and cement of the FDR layer is determined during mix design. The target 7 days Unconfined Compressive Strength (UCS) ranges from 200psi (Ohio for thin asphalt concrete pavement) to 550psi (California). An increase in the amount of Portland cement increases stiffness of the FDR base layer, but excessive cement content could lead to non-load related distresses, such as transverse shrinkage cracking.

The use of Ground Penetrating Radar (GPR) is recommended by the states of California and Texas for existing asphalt concrete thickness determination. The use of high Reclaimed Asphalt Pavement (RAP) content reduces the moisture-susceptibility of the materials used. After studying the effect of RAP content on the mechanical properties of FDR material, Guthrie et al. recommended increasing the RAP content limit as the 7-day UCS of the cement treated base with high RAP content is satisfactory.

State specifications require that the cement mixed layer be compacted to a minimum of 95% of the Maximum Dry Density. After compaction, the FDR requires a curing period of 3 to 7 days. Microcracking the cement-treated layer between 24 and 72 hours after construction is often recommended to limit severe block cracks. However, longer-term performance data show that microcracking has not always been successful in preventing cracking. The

FDR may be opened to lightweight local traffic if necessary prior to asphalt concrete placement.

Assessment of the FDR Project and Mechanistic Analysis

Field construction of FDR in Georgia was monitored as field samples and cores were collected for the UCS test. Deflection tests with the Falling Weight Deflectometer (FWD) and Light Weight Deflectometer (LWD) were conducted for up to five days after FDR construction to monitor modulus changes over the curing period. The results permit an evaluation of the effects of variability in measured properties on the performance of reconstructed pavement with FDR.

The key findings are summarized below:

- Field sampling resulted in reclaimed base with large chunks of asphalt concrete and solid rocks, indicating that this early-stage process should be improved to ensure the consistency in cement-aggregate mix. Laboratory samples also suggest that poorly-prepared base materials could lead to early damage caused by a loss of adhesion between the mix constituents. Repeated passes of a train of machines were less effective than anticipated to address this issue. A substantial portion of large asphalt lumps remained without being properly crushed and mixed.
- Contrary to our expectation, GPR scanning offered little information on the pavement layers, such as layer thickness and its variation along the path. This study has adopted an antenna suitable for mid-depth surveys based on literature reviews and technical consultation with other state agencies, including the Virginia DOT.

But the formation of layers was not clearly detected by GPR sensors especially during the first week of construction, in part due to moisture distributed across the depth of pavement. Therefore, GPR scanning is not recommended as a quality control (QC) tool, and certainly not a quality assurance (QA) tool. Instead, it would be practical to extend the current practice with boreholes to cover wider sections and acquire the layer information.

- The Analysis of variance (ANOVA) tests on the UCS data revealed both 3-day and 7-day strengths vary significantly by location within the project limit, which is likely due to the variation in the parent material and the cement contents in the mix. Although cement spread rate was maintained uniformly throughout the project area, variation in treatment depth could result in variation in cement contents. Deflection data obtained with FWD and LWD also show that stiffness increases while variability decreases with time. The LWD can be used for quality control and quality assurance of any FDR project especially in the first three days after treatment. Both LWD and FWD should be used to monitor spatial uniformity in layer stiffness.
- The mechanistic sensitivity analysis with varying modulus and thickness brings forth an important relationship. The predicted pavement responses are highly affected by the stiffness and thickness of the FDR base layer. The tensile strain at the bottom of the FDR base decreased with FDR modulus and thickness indicating that the asphalt concrete fatigue cracking would not be an issue. In FDR pavement, a high-strength base layer plays a similar role to a non-reinforced concrete slab. Therefore, surface asphalt on the FDR base can be treated as an asphalt overlay on

concrete pavement that is prone to reflective cracks. To warrant the designed service life of the surface layer, its construction quality must be controlled, primarily in compaction and selection of lift thickness and materials.

- Similar to asphalt concrete, the thickness and modulus of the FDR base layer have a significant influence on the horizontal stress at the bottom of the FDR base. The National Cooperative Highway Research Program (NCHRP) model for cement-treated aggregate clearly shows that the fatigue cracking performance of the FDR is highly sensitive to the tensile stress of the FDR base. FDR specifications should be developed to minimize variations in strength and thickness.

CHAPTER 1. LITERATURE REVIEW

INTRODUCTION

Full Depth Reclamation (FDR) is the process of in-situ pulverization of existing pavement and mixing with stabilizing agents such as cement or hydrated lime to create a new base layer. FDR is commonly used to rehabilitate structurally failed flexible pavements due to base and subbase issues. The FDR process is cost-effective and eco-friendly. Since this reclamation process involves the reuse of existing material, little or no material is required to rebuild the road.

The steps used in deciding if FDR is an appropriate rehabilitation strategy are to perform an in-depth pavement distress identification survey, to determine the cause of the pavement distress (functional or structural), to perform field testing to check field conditions, and to perform laboratory testing to produce an optimum mix-design procedure [1].

Construction sequence varies based on the scope of the project and stabilizers being used. An FDR construction process consists of four steps, including sizing, stabilization, shaping, and compaction. Sizing and stabilization are usually performed with a single-unit reclaimer using a two-pass method. The recycling process is shown in Figure 1.

Portland cement increases stiffness and reduces temperature sensitivity of the FDR base layer when used as a standalone stabilizer or as an additive with asphalt stabilizers. The long-term in-service performance of the FDR projects showed promising results comparable to similar projects rehabilitated with an AC overlay. An increase in the amount of Portland cement increases stiffness of the FDR base layer, but excessive cement content

could lead to non-load related distresses such as transverse shrinkage cracking. For this reason, the Virginia Department of Transportation (DOT) limits the cement contents [2].

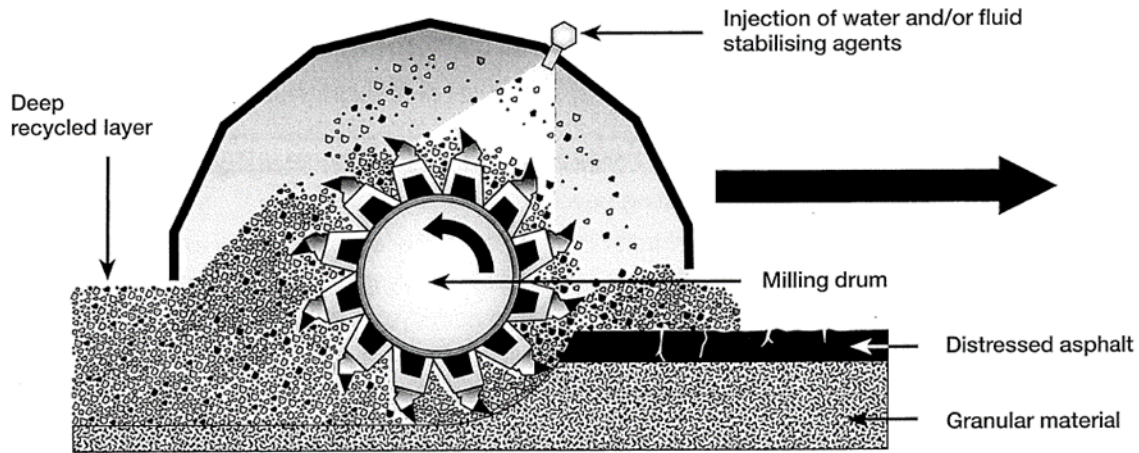


Figure 1. Illustration. Full-Depth Reclamation (FDR) process [3].

The Georgia Department of Transportation (GDOT) has conducted several FDR projects. The GDOT's new supplemental standard specification, Cement Stabilized Reclaimed Base Construction (Section 315), will serve as a specification for the design and construction of FDR sections. Mix design, construction requirements, and quality acceptance criteria are listed in the specification section 315. In Georgia, the quality of FDR is accepted through a compaction test, gradation test, and measurement of the finished surface profile and thickness while an unconfined compressive strength test is no longer in the quality acceptance [4].

A literature review on FDR using cement was conducted to consolidate information about FDR. The main objective of this chapter is to review the specifications and guidelines used by various agencies to refine current specifications to ensure good performance of FDR pavements. The findings presented in this chapter cover the field sampling and mix

design, installation, and techniques that can be used for site investigation, quality control and quality assurance measures.

FIELD SAMPLING AND MIX DESIGN

The effectiveness of FDR is governed by the condition of the existing pavement material, variability in materials and subgrade conditions. The FDR mix design process includes a sampling of the existing pavement material and laboratory mix design testing to achieve desired strength and durability. Most state highway agencies require the contractor to develop an FDR mix design based on the characterization of the existing pavement condition with a cement content that achieves the target strength, and the agency performs periodic QA testing.

In this section, field sampling and mix design methods of full-depth reclamation with cement used by various state highway agencies are summarized.

Field Sampling

Field investigation is required to evaluate the existing pavement layer structure and thicknesses of the pavement layers used in the FDR project. The work is accomplished by various in-situ and laboratory tests. Coring is commonly used to establish the surface layer thickness samples and its condition. Although coring has been a standard means of determining layer thickness for many years, Ground Penetrating Radar (GPR) equipment has been used to provide continuous evaluation of pavement layer thickness [5], [6], [7]. Table 1 summarizes site investigations and material sampling requirements of various agencies.

Table 1. Field sampling and site investigation methods.

State / Agency	Pavement Thickness Evaluation		Strength Evaluation	
	Coring/sampling	GPR	DCP	FWD
CA [5]	Obtain 500-lb of material in every 1,500 ft in the center of the lane	Recommended	Every 1,500 ft	21 deflection measurements per lane-mile
NY [8]	4 cores per lane mile	Not specified	Not specified	Not specified
OH [9]	Every 500 square yards, but not less than 4 samples for a project	Not specified	Not specified	Not specified
PA [10]	Obtain 100-lb of material in every 500 ft	Not specified	Recommended	Recommended
SC [11]	140-lb	Not specified	Not specified	Not specified
TX [12]	200-lb	Recommended	Recommended	Recommended
VA [13]	Every 2500 ft, with a minimum of six locations for each mix design.	Not specified	Not specified	Not specified
PCA [14]	Arterial and industrial streets – 350-lb in every 2000 ft	Not specified	Recommended	Not specified

In addition, Falling Weight Deflectometer (FWD) and Dynamic Cone Penetrometer (DCP) can be used to evaluate the stiffness of the pavement foundation below the anticipated FDR and locate weak areas requiring improvement or stabilization before the FDR process. The FWD can be used to determine pavement strength and uniformity after construction [5], [14], [15]. Laboratory evaluation of the sampled base and subgrade materials includes gradation analysis, Atterberg limits test, and a moisture content test. In

addition, the effect of chemical components such as sulfate content, pH, and organic content of the soil that will be incorporated into the mix should be determined to ensure proper soil cement reaction [5], [14], [15]. The sulfate concentration and organic content in the material should not exceed 0.3 % (3,000 ppm) and 2.0% (2,000ppm), respectively. A minimum pH of 4.0 for soils is recommended for proper cement bonding.

Mix Design

Mix design is then carried out with reclaimed materials to determine an appropriate cement content, optimum moisture content, and maximum density to achieve target engineering properties of the FDR layer and long-term performance of the road after rehabilitation. The FDR mixture consists of reclaimed materials, new aggregate (if required), Portland cement and water.

Elements that can affect the overall FDR performance include the amount of existing asphalt pavement, existing base course material and/or subgrade material that will be incorporated in the FDR layer. The Texas Department of Transportation limits the amount of recycled asphalt pavement to less than 50% of the cement-treated mix [15].

The target dry density and Optimum Moisture Content (OMC) of pulverized material depend on Reclaimed Asphalt Pavement (RAP). RAP content may vary greatly across a project site due to the variation in asphalt thickness. This makes the quality control of an FDR project quite challenging. Further, the variation in material properties leads to variation in Unconfined Compressive Strength (UCS) values of the stabilized base course.

Guthrie et al. investigated the effects of Recycled Asphalt Pavement (RAP) content, RAP type, and base type on the mechanical properties of cement treated base materials in

northern Utah [16]. The strength, stiffness, and moisture susceptibility of laboratory specimens were evaluated in a full-factorial experimental design to fulfill these objectives. The authors stated that utilizing as much RAP as possible reduces pavement reconstruction costs and demonstrates environmental responsibility. The use of high RAP content also reduces the moisture-susceptibility of the materials used. This improvement is needed in areas with high water tables, repeated freeze-thaw cycles, sustained freezing temperatures that lead to frost heave, or poor drainage. The researchers reported that the addition of 25% RAP caused a 29% decrease in strength compared with the neat base material, and the strength declined 13% to 15% with each additional 25% increase in RAP content. However, the authors recommended re-evaluating agency policies and specifications to increase the RAP content limit as the cement treated base with high RAP content can be expected to achieve satisfactory strength after 7 days of curing [17].

The amount of RAP in the mix could affect the moisture-density relationship. In general, the cement treated mix with a high RAP content does not produce a peak with the moisture density curve. For this reason, the NYSDOT fixed the moisture content with a high RAP content between 2% and 3% [16].

The mix design procedure for FDR involves a determination of the Optimum Moisture Content (OMC), Maximum Dry Density (MDD), and the Unconfined Compressive Strength (UCS) of the FDR. Mix design methods used by various state highway agencies are summarized in Table 2.

California [5], [18]

The California method uses the average UCS value of the samples prepared with 3 different cement contents (target and $\pm 1\%$ of specified content by dry weight of FDR-cement).

Samples are cured in an oven for 7 days at 100 degrees °F. The average UCS should be greater than 350psi and less than 550psi.

Indiana and Ohio [19], [20]

The target UCS depends on the thickness of the HMA overlay. In general, the required UCS increases for a pavement section with a relatively thin HMA surface. The Indiana DOT requires the UCS of FDR sample to be greater than 300psi when the HMA overlay is greater than 4.5 in, while a UCS of 500psi is required when the HMA overlay is less than 2.5 in. Similarly, the Ohio DOT requires a UCS of 300psi when the HMA thickness is less than 3 in. The required UCS of the FDR sample is reduced to 200psi when the HMA thickness is at least 3in.

Mississippi [21]

Samples are prepared in accordance with the Mississippi Test Method, MT-25. Samples are mixed at an estimated cement content and $\pm 1\%$ of the specified content by dry weight of FDR-cement.

The Mississippi Department of Transportation conducted a research study to characterize properties of FDR and to provide design, construction, and performance guidance for FDR layers in high traffic applications. The researchers developed a plastic mold compaction set that could be used for mix design, quality control, strength and elastic modulus testing [22].

New York [8]

The job formula is accepted if the UCS of the sample is between 350psi and 800psi.

Pennsylvania [10]

The Pennsylvania method requires samples for UCS tests to be prepared in accordance with ASTM 1633, method A. After a 7-day curing period, the specimens are tested for UCS. The job formula is accepted if the UCS of the sample is between 200 and 500psi for roads with an overlay thickness of more than 3 inches and between 300 and 500psi for roads with an overlay thickness of less than 3 inches.

South Carolina [11]

The South Carolina method requires samples for UCS tests mixed at the OMC at 3.0 percent, 6.0 percent, and 9.0 percent cement. Specimens are cured for 7 days at $73^{\circ} \pm 4^{\circ}\text{F}$ and a relative humidity of greater than 95%. The specimens are soaked overnight in water and tested for unconfined compressive strength. The cement content is determined based on the desired strength from the average strength – cement content chart.

Texas [23]

The Texas method requires determination of the maximum dry density and the OMC for a soil-cement mixture containing 6% cement. The samples are mixed at the OMC and at 4, 8, and 10% cement contents. A 6 x 8-inch mold and a 10-lb hammer are used to compact the samples into test specimens. The compaction should be done in four layers with 50 blows per layer. After curing for 7 days, the specimens are tested for UCS.

Table 2. Mix design methods used by various state highway agencies.

State / Agency	Target Unconfined Compressive Strength (psi) ¹	Curing Period and Condition for Laboratory Test	Moisture - Density	Aggregate Size Requirement
CA	350 to 550psi	Cure in an oven for 7 days at 100F°	AASHTO T-99	100% passing 3in. sieve and 85 % passing 1½in. sieve.
IN	HMA > 4.5in.: 300psi. 4.5>HMA > 2.5in.: 400psi. HMA < 2.5in.: 500psi	Cured in a moist curing room maintaining 72±3°F and a relative humidity of 100% for 7 days	AASHTO T-180	100% passing 2in. sieve and 55% passing no. 4 sieve
MS	14-day compressive strength of 300psi.	7 and 14 days	Mississippi Test Method, MT-9, Method "A."	98% passing 2in. sieve and 95% passing 1.5in. sieve
NY	350 to 800psi	Not specified	AASHTO T-99	Not specified
OH	200psi when HMA thickness is at least 3in. 300psi when HMA thickness is less than 3in.	7 days	AASHTO T-99	95% passing 2in. sieve
PA	300 to 500psi when HMA overlay thickness is at least 3in.	7 days	AASHTO T-99	95% passing 2in. sieve
SC	Target UCS is determined by engineer	Cured in a moist curing room or curing chamber maintaining 73° ±4°F and a relative humidity not less than 95%.	The maximum density test is to be conducted on a sample containing 6.0 percent cement	Not specified
TX	Target UCS is determined by engineer	7 days	10 lb. hammer, 18-inch drop, 50 blows/layer using 6 × 8in. mold Compact the specimen in four layers	100% passes 1-3/4in. sieve
VA	250 to 450psi	7 days	Not specified	100% passing 2in. sieve with 55% passing 3/8in. sieve.

¹ In accordance with ASTM D1633, Method A unless specified otherwise.

QUALITY ASSURANCE AND QUALITY CONTROL

Quality Assurance and Quality Control are critical to the success of an FDR construction.

Table 3 summarizes specifications/standards for weather limitations, curing method, compaction and aggregate size requirements of various agencies.

Table 3. Construction criteria used by various state highway agencies.

State / Agency	Minimum Air Temperature	Curing method	Compaction requirement
CA	40°F	Apply a coat of diluted asphaltic emulsions to the finished surface when the surface is damp but free of standing water.	97% of the in-place density of the test strip
IN	40°F	Minimum 3 days before placing the final surface.	95% Maximum Dry Density (MDD)
MS	45°F	Cure the surface for 7 days. A curing seal of emulsified asphalt shall be applied on the reclaimed layer.	97% MDD
NC	40°F	Cover the surface with asphalt curing seal and cure the surface for 7 days.	97% MDD
NY	45°F	Curing is the contractor's responsibility.	Not specified
OH	Not specified	Cover the surface with asphalt curing coat and cure the surface for 5 days.	Engineer will determine the minimum density for acceptance
PA	Not specified	Cure the surface for 5 days. Apply the asphalt prime coat within 24 hours of final construction.	98% of the in-place density of the control strip
SC	40°F	Cure the surface for 3 days. 4 curing methods are specified; Wet cure, Surface treatment, Wet cure and surface planning (HMA placement), and Surface treatment and surface planning.	95% MDD
TX	Not specified	Cure the surface for 3 days.	95% MDD
VA	40°F	Wet cure the surface until covered with an asphalt-based layer. Asphalt-based layer can be placed any time after finishing as long as the FDR can support the required construction equipment.	97% MDD
FAA	35°F	Wet cure the surface for 7 days.	95% MDD

Microcracking

One of the most promising measures, used in conjunction with appropriate mix designs, is that of microcracking the cement-treated layer between 24 and 72 hours after construction. In theory, this action creates a fine network of cracks in the layer that limit or prevent the wider and more severe block cracks typical of cement-treated layers. Limited research to assess microcracking as a crack mitigation measure has been completed on a number of projects in Texas, Utah, and New Hampshire [24].

Recommendations from these studies have been implemented by the Texas Department of Transportation and other state departments of transportation. Longer-term monitoring on a range of projects in Texas and other states has revealed that microcracking has not always been successful in preventing cracking, with some projects showing reflected transverse and block cracks in a relatively short time period, attributed to a number of factors including but not limited to cement spreading, method of curing, and interval between base construction and placement of surfacing.

Opening to Traffic

Under the South Carolina Department of Transportation (SCDOT) specification section 306, the completed section of the base course may be opened to light construction equipment after the 7-day curing period [25].

The Mississippi Department of Transportation requires a minimum of a 4-inch thick crushed lime stone layer to protect the treated course if the area must be opened to traffic prior to completion of the curing period [26]. The crushed limestone layer (buffer material) shall be removed prior to placing a surface treatment or HMA layer.

The North Carolina Department of Transportation allows the completed section of the base to be opened to light weight local traffic if necessary.

Miller et al. compared FDR base layer stiffness using a falling-weight deflectometer (FWD), a heavy Clegg hammer and a Geogauge at the field test site in New Hampshire and Maine. Field testing at both sites indicated that a substantial increase in strength and stiffness occurred in the CTB materials during the first two to three days of curing. The field stiffness survey results show that early trafficking reduces the initial strength gain and stiffness of CTB layers [16].

TECHNIQUES FOR FDR ROADWAY CONSTRUCTION

Pavement rehabilitation design must be based on reliable and appropriate information on the existing pavement. Sufficient investigation must be carried out to determine thickness of the pavement, material used in the construction of the existing pavement, and expected traffic. The pavements are not usually uniform. Thickness, materials, and type of distress will vary over long distances. The primary objective of the site investigation is to set boundaries between different pavement sections. This can be achieved by visual inspection and detailed condition assessment using non-destructive field tests, such as deflection measuring tools and Ground penetrating radar (GPR).

In-situ Tests for Pavement Structural Capacity Evaluation

Commonly used in-situ test methods include the DCP, LWD, and FWD tests. The primary objective of conducting these in-situ tests is to evaluate the subgrade strength and modulus for estimating pavement structural capacity. Both DCP and LWD can be used to assess in-

situ strength of the base and subgrade while FWD can be used to evaluate the strength of the pavement system before and after construction. Additional evaluation of base and subgrade materials may be performed in the laboratory using a sieve analysis, plasticity test, hydrometer particle size analysis, moisture content test, and proctor test [27].

Scullion et al. performed a comprehensive study to identify the key steps in the design and construction of the FDR process. Problems have been encountered with pavements built on expansive clays (most of east Texas). Edge drying and fallen trees bordering roadways are a problem when brittle stabilized layers are placed over them. When severe longitudinal cracks exist, the use of the DCP should be encouraged to identify the depth of slip planes and to aid in designing the appropriate edge support [28].

The LWD is a rapid test method for soil compaction quality and unbound base courses in earthwork and road construction. The light weight deflectometer measures the soil dynamic LWD modulus empirically correlated to the soil degree of compaction. For the same relative density, the LWD deflectometer modulus for siliceous sand is higher than the LWD modulus for calcareous sand.

The relationship between the LWD modulus and degree of compaction given in DIN 18196 is evaluated for estimating the degree of compaction for calcareous and siliceous sands. The measured degrees of compaction for calcareous and siliceous sands bound by the DIN relationship may be considered as an “average” for both mineralogies. The DIN 18134 relationship provides higher accuracy for estimating degrees of compaction higher than 95%, which may be explained by the fact that the DIN relationship is originally given for degrees of compaction higher than 95%. The zone of influence of

the light weight deflectometer is found to be 1.5 to 2 times the diameter of the LWD plate [29], [30].

It has been demonstrated that the stiffness modulus is sensitive to variation in the course of loading and unloading phases. In numerical analyses, it was found that the ground is not linear elastic, and the identification, due to the maximum amplitude, does not properly reflect the full process of deformation of the ground in the phase of loading, and especially in the phase of unloading. Variability of the stiffness modulus is related to the presence of highly nonlinear effects of dynamic deformation of soil. In procedures in which the evolution of the modulus in time was searched, the initial stiffness of the soil was very high (about 5300 MPa in the present case).

It was found that the stiffness modulus of soil decreases very rapidly. The instance of such large baseline values of the modulus can be explained by the inertial resistance of granular soil structure. After the initial phase of loading, it is noted that the numerical prediction of the dynamic stiffness modulus is in accordance with the value of the modulus determined by a standard test procedure of the Light Falling Weight Deflectometer. The values of the dynamic stiffness modulus in the phase of unloading are significantly higher than in the loading phase. The basic assumptions of the study of modulus E_{vd} , according to the manual of the device, may be accepted as the interpretation of the results leads to a correct evaluation of the average stiffness modulus of the soil in the loading phase, even though the whole process of the soil deformation is not taken into account in this approach [31].

Ground Penetrating Radar (GPR)

Ground penetrating radar (GPR) has been used by many highway agencies to measure the thickness of existing pavements. In addition to pavement thickness data, other anomalies in GPR signals are used to locate the areas of poor quality in new and old pavements [32]. The first radar designed to survey subsurface conditions was developed by MIT in the 1960's for the U.S. military.

The first commercial GPR was made available in 1970 and the first vehicle-mounted GPR was designed in 1985. Since the 1970's, the GPR technique has been used by highway agencies to evaluate the near-surface conditions of pavements. In 1998, a survey of GPR practices of 33 U.S. and Canadian state and provincial highway agencies was published in NCHRP Synthesis 255. The practices included: (a) measuring pavement thickness, (b) measuring base and subbase thicknesses, (c) locating voids beneath pavements, (d) detecting delamination, (e) detecting excess moisture, and (f) mapping underground utilities [33].

According to Morey, these practices have been more successful in determining the asphalt pavement thickness than any other applications listed above [33]. Thickness estimations were made within 10% error for asphalt pavements up to 0.5 m. Challenges and varying level of success were reported in concrete pavement thickness measurements and dry/wet void detections.

There are two basic types of GPR systems; the ground-coupled antenna system, in which the antenna is kept in contact with the ground surface and operated by pushing or pulling the cart housing the antenna unit, and the non-contact horn antenna system, where the antenna is suspended over the surface with the help of a frame attached to a truck or

van [34]. Ground-coupled antennas provide greater depth of signal penetration whereas non-contact-horn antennas allow faster data acquisition rates and high-speed surveys. GPR systems have been manufactured by several vendors including Geophysical Survey Systems Inc (GSSI), Sensors & Software, MALA Geoscience, ERA Technology, Utsi Electronics and others. However, they all operate under the same electromagnetic wave principles [35].

One of the great advantages of the GPR survey is that it is nondestructive and provides continuous thickness data. Additionally, if a non-contact horn antenna is used, the survey can be conducted without closing any lanes to traffic. As one of the leading examples, Texas Transportation Institute has developed guidelines for truck mounted 1 GHz non-contact horn antenna test procedures since the early 1990's [36].

The performance of GPR waves depends on electrical and magnetic dielectric properties of the material. Permittivity and conductivity are two electrical dielectrics, and permeability refers to magnetic dielectrics. The impact of magnetic dielectrics is more noticeable in unique conditions- not common in most construction materials- when the soil has ferromagnetic minerals, such as magnetite and maghemite [37]. Higher permittivity values generate a slower wave propagation, and higher conductivity values have a negative impact on the energy of the waves. While GPR waves propagate, reflections will occur at boundaries with a high contrast in electrical dielectrics of the material. Detection of such reflections helps determine the layer thickness of different pavement material. When the layers have similar dielectric values, the reflections become less distinctive, and therefore locating the boundaries between the layers becomes more difficult.

Water is a major factor in determining the electrical dielectric property of pavement materials. Among all-natural materials, water has the highest dielectric permittivity value. Therefore, electromagnetic waves will travel much slower in soil with a high water content. Interactions between the dielectric properties and the moisture content have been studied by several researchers since early 1980's. One can find a comprehensive review of dielectrics of soils with different forms of water in Ulayb et. al. [38].

In cement-bound materials such as concrete and soil-cement mix, reactions take place between cement and water. As time passes, water content decreases and the dielectric permittivity of cement-bound material changes. This time-dependent interaction has been discussed in great detail in papers since 1975 for fresh and dry concrete. The results showed that concrete is significantly dispersive when fresh, making it very difficult for the electromagnetic waves to travel [39]. For this reason, AASHTO R 37-04 recommends waiting 180 days to conduct any GPR survey on concrete pavements [40].

However, the literature review has yielded very little guidance regarding the time sensitive interaction of moisture content and dielectrics of soil-cement mixes used in FDR projects. Diefenderfer et. al. [41] and Chris et. al. [42] conducted GPR surveys 6 weeks and 12 months after construction, respectively. However, none of their research pointed to a published reference regarding the wait time after the construction.

AASHTO R 37-04 provides a general guideline for thickness measurements of concrete and asphalt pavements and void detections below such pavements by using GPR equipment but does not provide any specific guideline for FDR pavements [40].

The Florida Department of Transportation has conducted a series of field surveys to evaluate the accuracy and repeatability of GPR data in estimating the thickness of hot

mix asphalt (HMA) and Portland cement concrete (PCC) layers [43]. Their results show that GPR data is reliable in both accuracy and repeatability. Similar studies conducted by other highway agencies also show promising results.

Some agencies have used the GPR technique to estimate strength, hydrogeological and density properties of the pavement materials by correlating the electrical dielectrics of the material to the relevant physical and strength property of the core samples [44], [45], [46].

GPR surveys have been used in estimating the depth of reclaimed layers in Full-Depth Reclamation (FDR) projects by several transportation agencies [41], [42], [47]. Although there are no published GPR guidelines specifically for FDR projects, the results from the available case studies show that, when validated with core samples, GPR is a powerful tool to estimate the thickness of the reclaimed base.

CHAPTER 2. ASSESSMENT OF GDOT'S FULL DEPTH RECLAMATION (FDR) PROJECT

INTRODUCTION

The State Route (SR) 70 from South Fulton Parkway to SR 92 in Fairburn, Georgia was rehabilitated during the 2018 construction season using FDR with cement. The SR 70 repaving project was divided into two segments; SR 92 to Ridge Road and Cedar Grove Road to South Fulton Parkway. The research team performed field and laboratory tests to assure the quality of FDR construction on segment 2 of the project. The 4.83-mile-long pavement section shows severe distress as shown in Figure 2.



Figure 2. Photos. SR 70 pavement condition prior to rehabilitation.

FDR construction of segment 2 was completed in three days. A 500-ft long test section was selected each day for field testing as listed below. Figure 3 shows the location map of the project area.

- Test section 1: Station 130+00 to 134+00 (South Bound Lane)

- Test section 2: Station 054+00 to 058+00 (North Bound Lane)
- Test section 3: Station 047+00 to 051+00 (South Bound Lane)

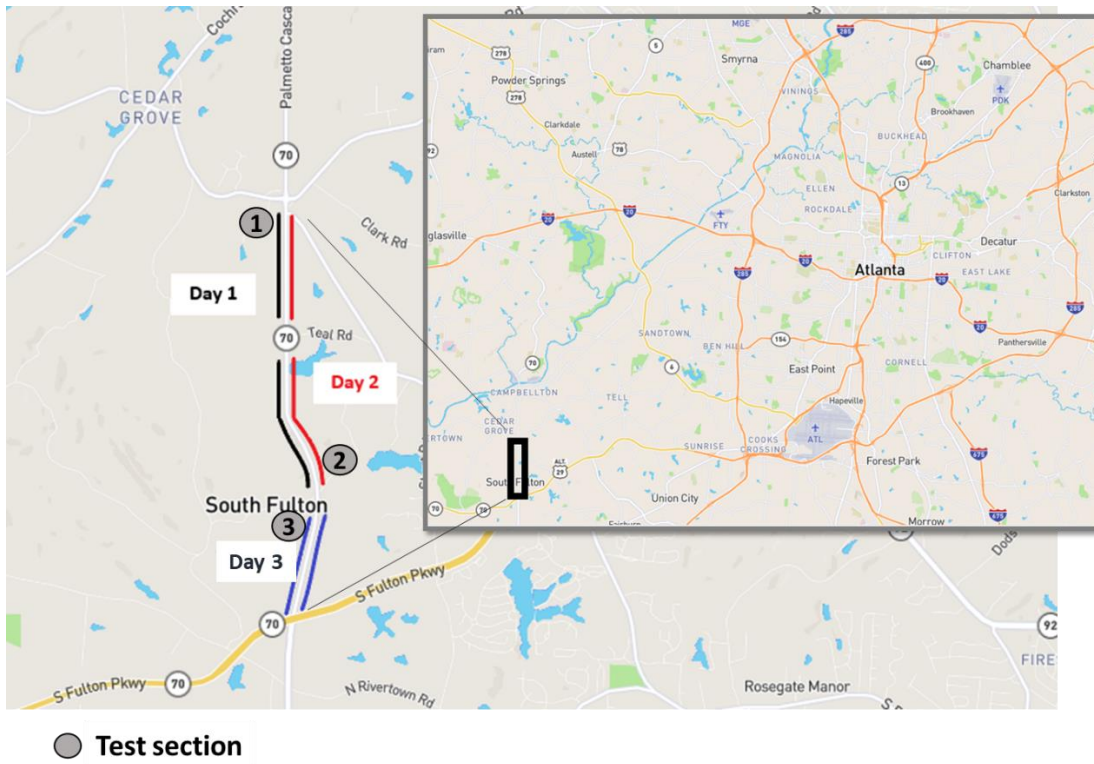


Figure 3. Map. Project location.

The research team performed field and laboratory tests to assure the quality of FDR construction on the second part of the project. Falling Weight Deflectometer (FWD) and Light Weight Deflectometer (LWD) testing were conducted at the test site to characterize the elastic modulus of the FDR layer. Ground Penetrating Radar (GPR) was also utilized to determine FDR layer thickness variation. Field samples were obtained from the compacted FDR mix for Unconfined Compressive Strength (UCS) tests to assess variability in the strength of the stabilized base course material.

CONSTRUCTION ACTIVITIES

Construction work began at the north end of the project site by milling down old asphalt across the entire project section (Figure 4). The entire section was pulverized with reclaimer to a specified depth and compacted with rollers a few days before the FDR construction (Figure 5). The purpose of the pre-mix was to produce a homogeneous mix and locate soft spots.



Figure 4. Photo. Asphalt milling.



Figure 5. Photo. Pre-mix.

The FDR process started with the application of cement to the premixed section (Figure 6). The target cement application rate was approximately 74 lbs/sy to achieve 8% cement of dry weight of the parent material. The cement spread rate on the pulverized pavement was measured throughout the section as listed in Table 4.

Table 4. Daily cement spread rates.

Date	Spread Rate: (lbs. /syd.)	Target Spread Rate: (lbs. /syd.)
9/17/2019	78.0	74
9/18/2019	75.7	74
9/19/2019	74.3	74



Figure 6. Photos. Dry cement placement and cement spread rate check.

After spreading the cement, the reclaimer made passes to cover the section. A water truck was attached to the reclaimer to add water to the mixture during mixing operations. As the reclaimer advanced, the FDR base was compacted with the tamping foot (sheep's

foot) roller. Then graders removed depressions from the tamping foot roller. A smooth-drum vibratory roller and pneumatic tire roller finished the FDR base surface (Figure 7).



Figure 7. Photos. Pulverization and compaction of the FDR base.

Material samples were obtained to evaluate the strength of the FDR layer. These samples were compacted and cured at the Kennesaw State University geotechnical engineering lab for up to 7 days. LWD and GPR were utilized at the test site to characterize the elastic modulus and the thickness of the FDR base layer (Figure 8 and Figure 9). Material sampling frequency and LWD testing intervals were set to 50 ft.



Figure 8. Photos. UCS sample preparation.

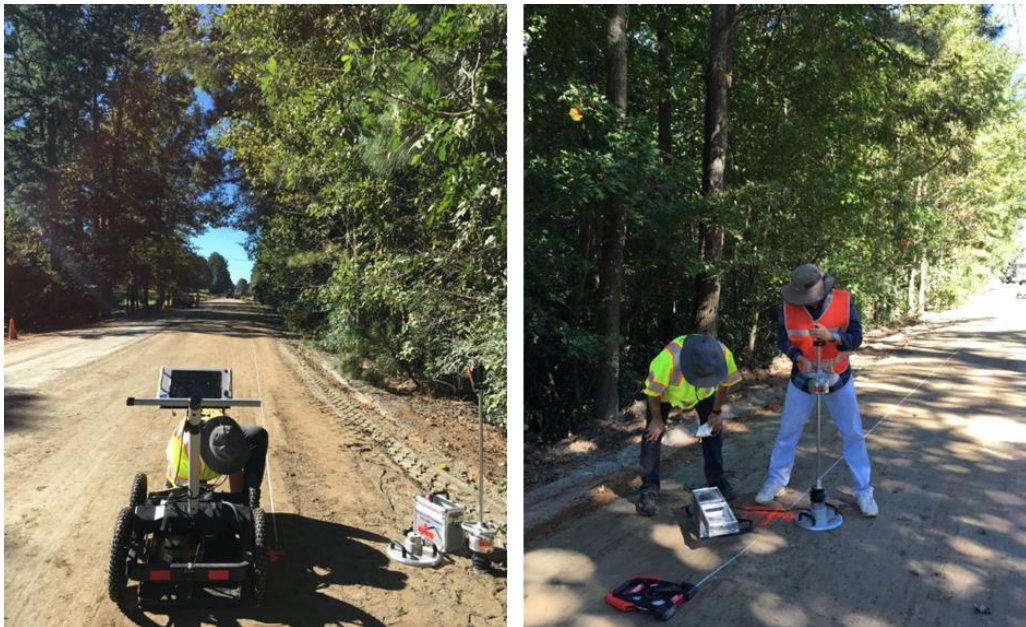


Figure 9. Photos. GPR and LWD testing on test section.

GDOT engineers checked the density using the nuclear density gauge and FDR base thickness (Figure 10). The maximum dry density for the in-place FDR mix was

determined according to GDT 67. Table 5 summarizes in-place density of the FDR mix. Due to the inconsistency of the in-place material, the field technician was unable to obtain passing compactions using the Max Dry Density (MDD) established in the mix design. Therefore, the GDT 67 procedure was used to determine the in-place MDD and percent compaction at each test location.



Figure 10. Photo. Density, moisture content and thickness verification (performed by GDOT).

A thin bituminous chip seal surface was added to protect the FBR base until placement of asphalt concrete was complete (Figure 11). The chip seal layer minimizes moisture loss during the curing period. The chip seal layer is also used as a bond breaker between the FDR base and the asphalt concrete wearing surface to retard reflective cracking of the FDR base.

Table 5. Roadway compaction summary (% compaction required = 98%).

Station	Location	Corrected Dry Density (pcf)	Max Dry Density ¹ (pcf)	% Compaction Obtained
130+00	Southbound Lane	115.92	114.00	101.7%
119+00	Southbound Lane	114.57	106.50	107.6%
045+00	Northbound Lane	122.67	116.00	105.8%
059+00	Northbound Lane	119.35	113.00	105.6%
077+00	Northbound Lane	117.41	118.90	98.7%
050+00	Southbound Lane	117.95	115.70	101.9%
038+00	Southbound Lane	120.51	113.50	106.2%
022+00	Southbound Lane	121.05	114.50	105.7%
069+00	Northbound Lane	125.49	119.40	105.1%

¹ The MDD used to determine the percent compaction in the field was determined using GDT 67.



Figure 11. Photos. Placement of the chip seal layer.

A total of 9 cores were taken at test strips (3 cores from each test strip) after a 5 to 7 day curing period for GPR calibration (Figure 12). An asphalt concrete wearing surface was placed 16 days after the construction start date. Local traffic was permitted on the road at the end of each day (Figure 13).



Figure 12. Photo. FDR core sampling for GPR calibration.



Figure 13. Photos. Asphalt concrete surface placement (10/3/2018).

UNCONFINED COMPRESSIVE STRENGTH

The representative FDR samples were then compacted into a 4 in. diameter mold with a height of 4.6 in. using the automatic compaction machine as shown in Figure 8. Standard Proctor compaction effort in general accordance with ASTM D698 was selected for sample preparation. However, higher compaction effort may be achieved in the field with a pneumatic roller. Upon completion of the curing period, the UCS was determined for each sample according to ASTM D1633 - Method A at different curing periods (3 and 7 days). The peak load sustained by each sample was used to calculate the UCS (Figure 14).



Figure 14. Photos. UCS test.

After all testing was complete, statistical analyses were performed. Samples were taken at 15 stations from three test sections as depicted in Figure 3. Six test specimens were fabricated from materials collected at each station; three samples were tested after 3 days and the other three samples after 7 days. Based on unconfined compressive strength test results, the 3-day and 7-day mean strengths for each station were computed and compiled in Table 6.

Table 6. Paired sample means by site.

Test Section	Station	3-Day Strength (psi)	7-Day Strength (psi)	Difference
1	130+00	131.57	136.34	4.77
	131+00	77.99	122.02	44.03
	132+00	116.98	155.18	38.20
	133+00	163.40	249.87	86.47
	134+00	195.50	215.12	19.63
2	054+00	89.39	123.08	33.69
	055+00	82.50	125.20	42.71
	056+00	85.41	113.80	28.38
	057+00	96.02	128.38	32.36
	058+00	131.04	164.73	33.69
3	047+00	137.40	154.12	16.71
	048+00	148.28	160.22	11.94
	049+00	176.93	179.85	2.92
	050+00	203.45	251.20	47.75
	051+00	192.05	198.68	6.63

A paired t test was performed by comparing the strength differences with a specified target from 0 to 20psi at an increment of 5psi. The t statistic and corresponding p-values are summarized in Table 7. As shown in Table 7, all null hypotheses based on the

difference up to 20 are rejected at the 0.05 significance level. An increase by 1psi in targeted difference beyond 20psi will result in a p-value exceeding 0.05. This indicates that the difference in 3-day and 7-day compressive strengths is significant up to 20psi at the 0.05 significance level.

Table 7. Paired t test results.

Null Hypothesis	t Statistic	p Value	Significance ¹
Difference = 0	5.394	4.728E-05	****
Difference = 5	4.495	2.519E-04	****
Difference = 10	3.596	1.461E-03	**
Difference =15	2.697	8.687E-03	**
Difference =20	1.797	4.695E-02	*

¹ Significance level: * 0.05, ** 0.01, *** 0.001, **** 0.0001

An Analysis of variance (ANOVA) was used to investigate the differences in UCS among sections. As shown in Table 8, the variance across sections is significantly greater than that within sections for both 3-day and 7-day test strengths, indicating that the strength varies significantly by location. By comparing between-groups variance with 3-day and 7-day strengths (Mean Squares 20993.510 versus 13762.048), 7-day strengths are more stable with less variability across the sites than those of 3 days.

Table 8. Analysis of Variance – strength comparison by sections.

Curing Period	Source of Variation	SS	df	MS	F	P-value	F crit
3-day	Between Groups	41987.02	2	20993.510	18.128	2.110E-06	3.220
	Within Groups	48638.73	42	1158.065			
	Total	90625.75	44				
7-day	Between Groups	27524.10	2	13762.048	8.354	8.821E-04	3.220
	Within Groups	69187.59	42	1647.324			
	Total	96711.69	44				

To verify the laboratory 7-day test strengths, field core samples were taken 5 to 7 days after FDR completion. 9 core samples (3 for each section) were taken, but two samples were broken during recovery. To compare the strength of the core samples with 7-day strengths of the laboratory samples, a t test was performed. As shown in Table 9, the laboratory 7-day strengths are generally less than the corresponding core sample strengths at the 0.05 significance level, except for Stations 55 and 56 (test section 2). Similar to the laboratory samples, the field cores from test section 2 exhibit much lower strength than the cores from other sections. Besides different curing periods for the field cores (i.e., 5-7 days), the inconsistency between the core samples and lab samples is likely due to the inherited variation in materials and thickness. This result shows that the strength decreases with increased FDR layer thickness, as the increased thickness results in a reduction of the cement content in the FDR mix.

Table 9. Comparison of 7-day strengths with core sample test.

Test section	Station	Thickness (in.)	Curing period	UCS (psi)				
				Field core samples	Laboratory mixed samples			
					Mean	SD	t stat	p value
1	132+00	14.25	7 days	282.59	155.18	10.44	21.15	0.00111
	130+00	12.25		435.02	136.34	16.35	31.63	0.00050
2	056+00	15.5	6 days	122.37	113.80	6.80	2.18	0.08026
	055+00	15		123.79	125.20	5.12	-0.48	0.66039
3	051+00	12	5 days	294.97	198.68	18.70	8.92	0.00617
	050+00	12		479.23	251.20	5.12	77.20	0.00008
	049+00	12.5		408.14	179.85	11.23	35.22	0.00040

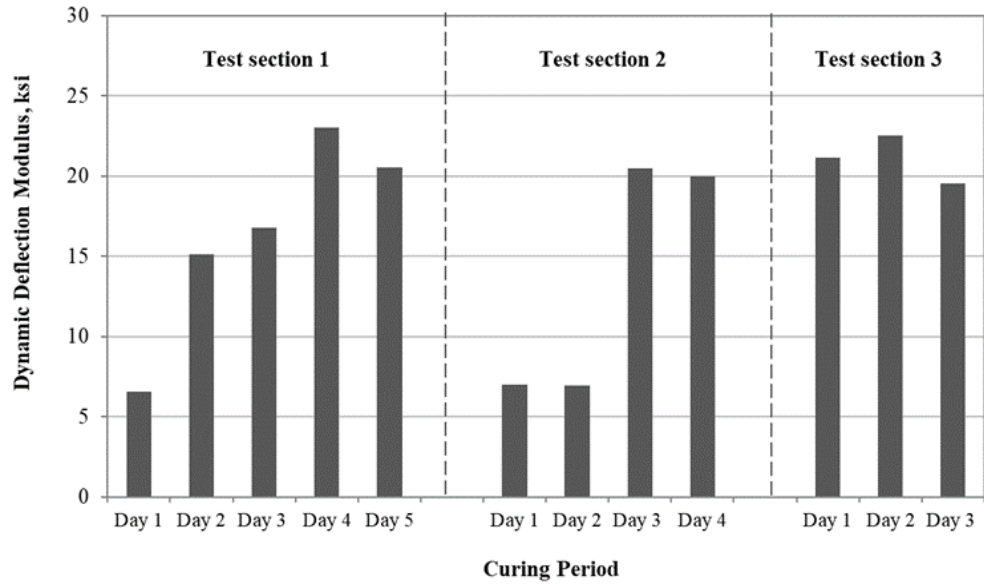
DEFLECTION DATA ANALYSIS – LIGHT WEIGHT DEFLECTOMETER AND FALLING WEIGHT DEFLECTOMETER

Pavement deflection measurements were carried out with Zorn LWD and Dynatest FWD. The LWD test was performed on selected pavement test locations for up to five days after FDR operation while the FWD test was performed on the entire site.

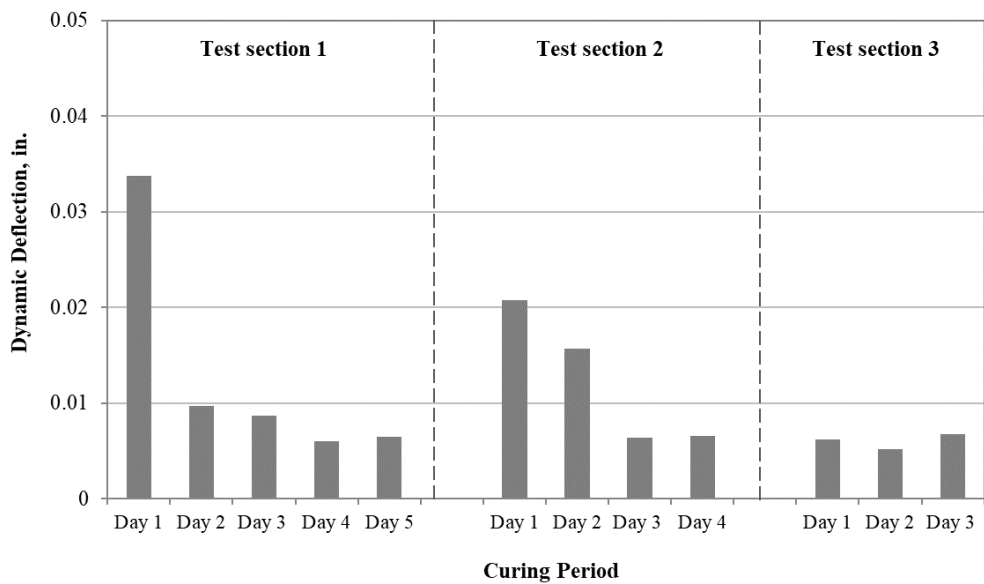
Figure 15 shows the relationship between the average LWD measured modulus, dynamic deflection and curing period.

The results indicate that the variability in modulus decreases with curing period. The average dynamic deflection moduli of test sections 1 and 2 increase significantly 2 days after FDR operation. Test section 3, however, achieved an average dynamic deflection of 150 MPa immediately after the FDR process. The UCS of field core and laboratory samples also shows that the samples from test section 3 have a higher strength than those from test sections 1 and 2. The UCS result of both field core and laboratory samples also show that the test section 3 has higher strength than test sections 1 and 2. The difference in

early age strength could be due to several reasons, including difference in parent material, construction quality, and moisture contents.



a) Dynamic deflection modulus (ksi)



b) Dynamic deflection (in.)

Figure 15. Graphs. LWD results – modulus and deflection changes over time.

Subgrade modulus values were computed using the equation developed by Thompson and Garg for aggregate surface/surface treated pavement section [48]. The average subgrade modulus was 10.4ksi with standard deviation of 3.5ksi. The results shown in figure 16 indicate several soft spots where the subgrade modulus is less than 5ksi. The back-calculated subgrade varies from 1.1ksi to 16.9ksi.

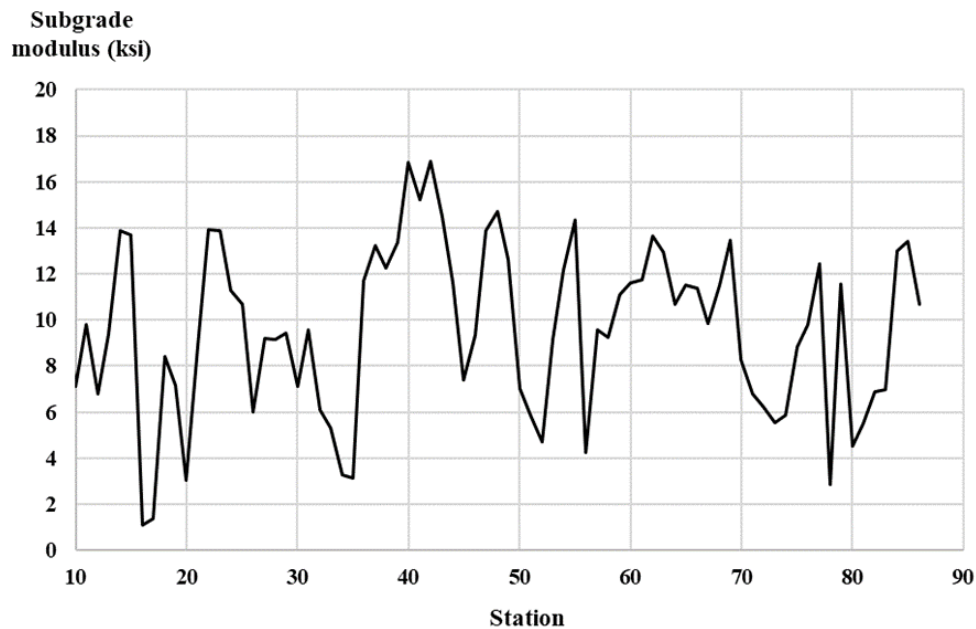


Figure 16. Graph. FWD backcalculated subgrade moduli.

Variability in thickness and subgrade modulus make it difficult to back calculate the FDR base modulus. Instead, FWD data were analyzed to characterize an entire pavement section in terms of pavement structure stiffness. The Impulse Stiffness Modulus (ISM), which is a normalization of the applied load by the resulting load plate deflection, was the basis for comparison. Figure 17 shows the relationship between the ISM and curing period. As expected, the stiffness modulus increases with time.

ISM (kips/in)

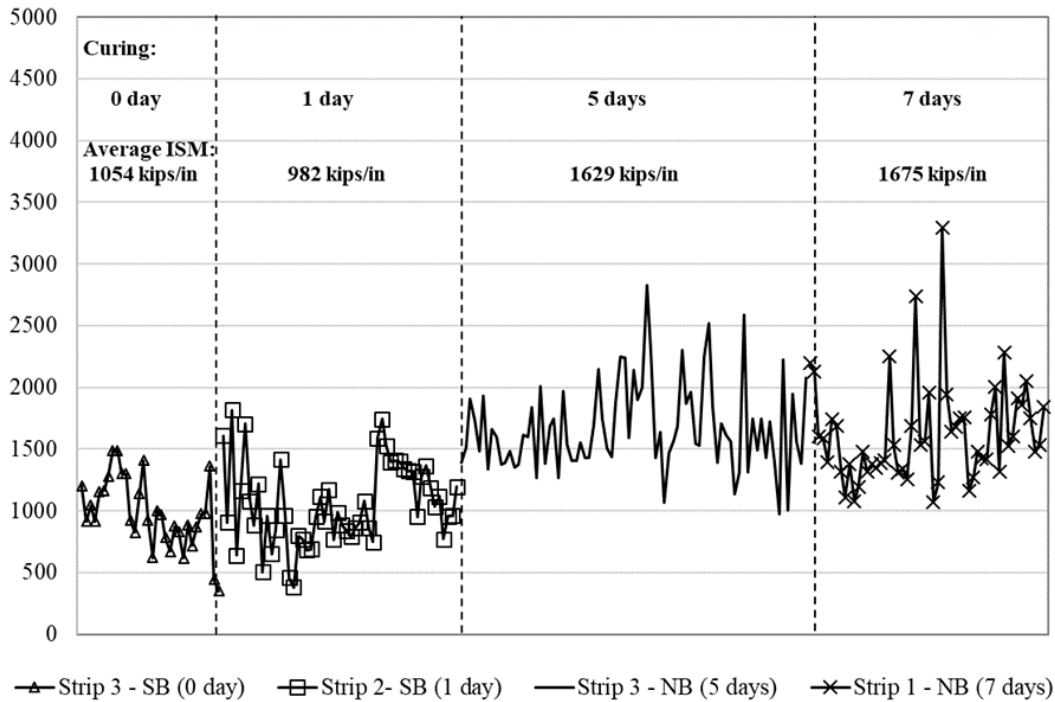


Figure 17. Graph. FWD results - ISM changes over time.

Deflection data obtained with FWD and LWD shows changes in spatial variability in stiffness with time. An Analysis of Variance (ANOVA) was used to assess the differences in LWD dynamic deflection modulus across test sections. As shown in Table 10, the variance reduces as curing time increases. The variance across days is only significant for section 2. On the other hand, the variance across sections is only significant for earlier days. A series of t tests was performed for the coefficient of variation of the ISM between successive curing periods. As can be seen in Table 10, the ISM variability is significantly reduced as the curing period increases from day 0 to day 1 and to day 5 to day 7. These results indicate that the stiffness variation decreases with time and the FDR layer provides more uniform stiffness.

Table 10. Statistical analysis results – deflection test.

LWD Analysis of Variance - Dynamic deflection modulus comparison									
	Test Section - variance across different curing periods)			Curing Period (Days) – variance across different test sections					
	1	2	3	1	2	3			
F statistic	2.36	6.66	2.60	4.59	13.49	1.29			
p value	0.13669	0.01018	0.09332	0.02453	0.00026	0.30517			
FWD Paired t Test Results – ISM (kips/in.)									
Age	Mean ISM	Standard Deviation	n	Coefficient of Variation (CV)	Unbiased CV	Standard Error	t ¹	p value	Significance level of 0.01
0	1053.6	330.80	54	0.314	0.315	0.030	n/a	n/a	n/a
1	982.2	285.42	32	0.291	0.293	0.037	-3.082	0.001385293	yes
5	1629.4	410.52	54	0.252	0.253	0.024	-6.049	0.000000019	yes
7	1674.8	360.89	79	0.215	0.216	0.017	-10.257	0.000000000	yes

¹ t test was performed by comparing the coefficient of variation for two successive curing periods (i.e., 1 day vs 0 day; 5 days vs 1 day; and 7 days vs 5 days)

FDR THICKNESS MEASUREMENT USING GROUND PENETRATING RADAR

In this study, a MALA brand ground-coupled GPR system with a 1.2 GHz antenna was used. AASHTO recommends using a high frequency antenna if the desired target is near the surface [40]. In many surveys with a non-contact horn antenna and speeds up to 80 km/h, frequencies from 0.5 GHz to 2.5 GHz have been reported to yield excellent resolution. As a general guideline, as shown in Table 11, Mala recommends a 1000 MHz antenna for a maximum depth of penetration of 2 ft., and a resolution of 0.05 ft. Given a typical FDR pavement thickness of 16 to 18 inches immediately below the surface, 1.2 GHz is a reasonable frequency selection for this study.

Table 11. Depth vs resolution.

<u>Antenna Frequency (MHz)</u>	<u>Approximate Resolution (feet)</u>	<u>Average Maximum Depth of Penetration (feet)</u>
2000	0.025	1
1000	0.05	2
800	0.1	6
500	0.3	15
200	0.8	30
100	1.6	60
50	3.2	100
25	6.4	150



There are four main components of the ground-coupled system used in GPR surveys:

- 1.2 GHz Antenna: This small unit (19cm x 11.5cm x 11cm) is fastened to the bottom tray using Velcro, so that it stays close to the ground surface as shown in Figure 18.
- ProEx Control unit: This unit is fastened to the plywood panel above the antenna as shown in Figure 18. Its role is to establish communication between the monitor and antenna.
- High Frequency module: This module is inserted into one of the slots on the ProEx controller unit and functions as a connector for the high frequency antenna.
- MALA XV monitor: After processing the reflected signals, this unit displays the anomalies in a graphical form. It also allows the user to change signal and display

parameters, and copy profile files to a USB drive. The files are then transferred to a PC for further interpretation using RadExplorer, software designed for GPR survey data processing.



Figure 18. Photos. GPR assembly.

In the field, once the components are assembled on a MALA Rough Terrain Cart with a data trigger system installed in one of its wheels, GPR surveys are simply conducted by pushing the cart at normal walking speeds as shown in Figure 9. In cases where GPR surveys are needed at specific points, such as core sample points, the components could be reassembled without the cart. GPR Surveys have been conducted for 5 days, at eight different sections. Table 12 shows the daily survey schedule. Sections 130+00 – 131+00

and 131+00 – 132+00 were surveyed for 5 consecutive days to see if the anomalies in the signal would change as the hydration takes place in the mix.

Table 12. Daily GPR survey schedule.

Section	Day 1 (09/17/2019)	Day 2 (09/18/2019)	Day 3 (09/19/2019)	Day 4 (09/20/2019)	Day 5 (09/21/2019)
130+00 – 131+00	x	x	x	x	x
131+00 – 132+00	x	x	x	x	x
132+00 – 133+00		x			
133+00 – 134+00		x			
49+00 – 50+00			x	x	x
50+00 – 51+00			x	x	x
54+00 – 55+00		x	x	x	x
55+00 – 56+00		x	x	x	x

Figure 19 is the profile of a GPR survey conducted on the first day of the FDR pavement construction. Data collection begins at the unmixed section and continues for 6 feet into a recently mixed section. This was done to understand how the signals would penetrate in wet FDR sections. The horizontal axis represents the distance on the ground surface in feet and the vertical axis represents two-way time (the time between the electromagnetic pulse leaving the antenna and reflecting back to the receiver) in nanoseconds, ns. The time axis is then converted to distance by calibrating the time difference between the anomalies on the profile with the length of the core samples. Figure 19 clearly shows that in the mixed section, the signal strength drops significantly, and

anomalies disappear. Therefore, it becomes very difficult to estimate FDR pavement thickness using GPR on the first few days of construction due to the high moisture content.

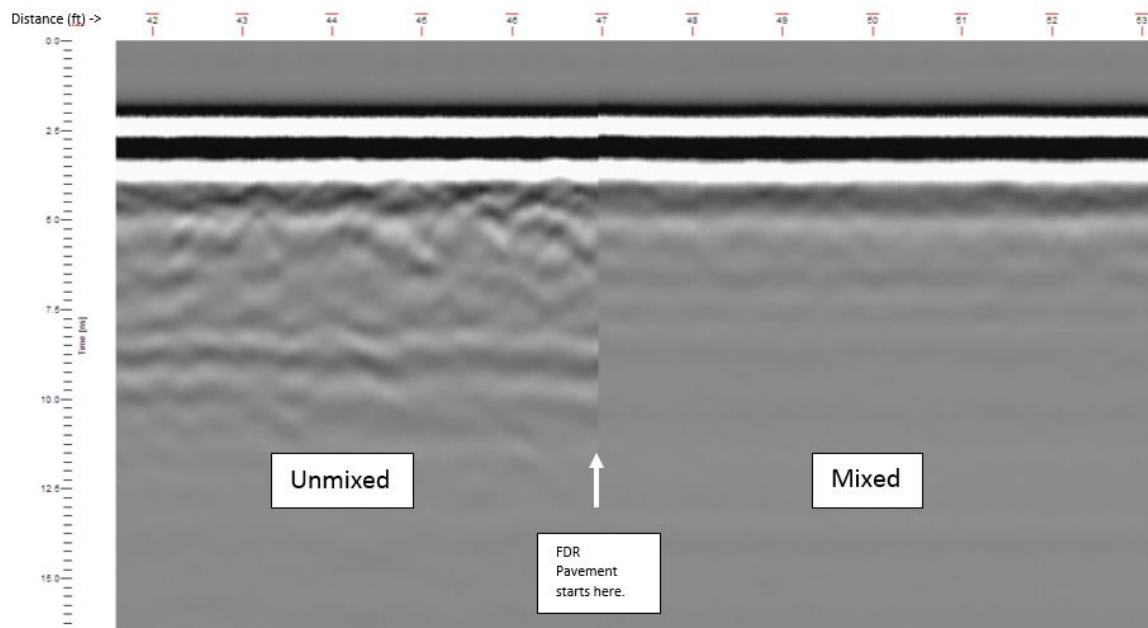


Figure 19. Photo. GPR profile on the first day of FDR construction.

Figure 20 shows the wave travel time signal on Day 1 and Day 5 for the same location. On Day 1, signals decay immediately after entering the pavement and no anomalies are observed. This was expected since the fresh mix has a high water content and more free electric charges. On Day 5, signals start showing slight anomalies indicating that the mix is becoming less conductive and signals can travel faster and deeper compared to Day 1. However, the signal pattern is not consistent along the survey path and anomalies are not strong enough to estimate FDR pavement thickness. Although ground-coupled antennas send more energy into the ground, reliable thickness estimates still cannot be observed within a few days of FDR pavement replacement.

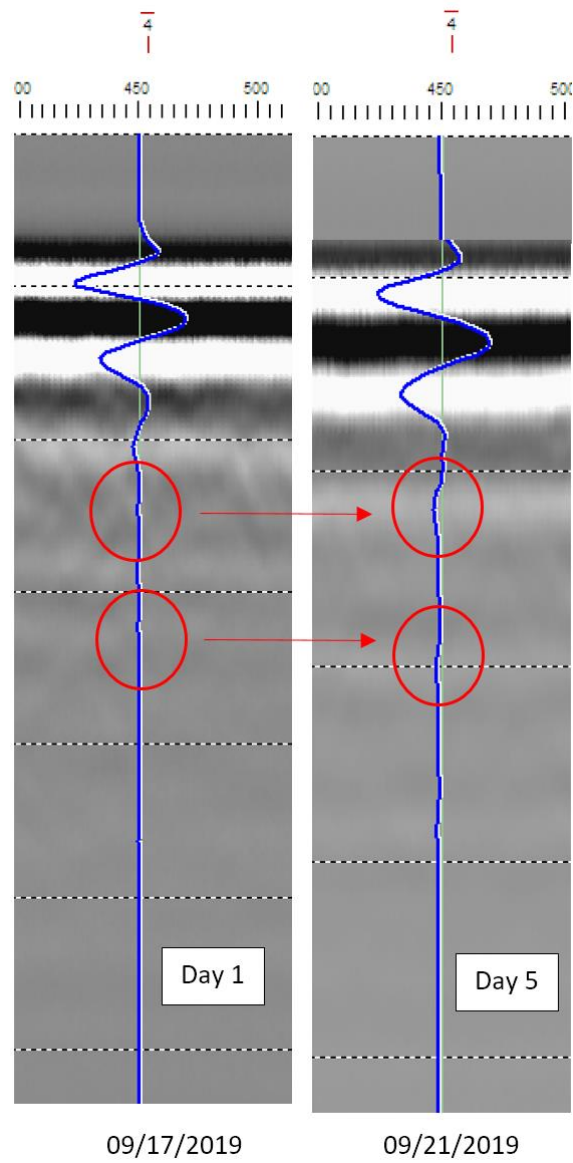


Figure 20. Photos. Time travel signals at station 131+00.

CHAPTER 3. MECHANISTIC SENSITIVITY ANALYSIS

AASHTOWARE PAVEMENT ME DESIGN (PMED)

A sensitivity analysis is performed to investigate the influence of FDR base quality on the predicted performance of pavement structures. This analysis will help identify the most influential FDR property on predicted performance. During the construction of the FDR base, samples were collected for laboratory determination of UCS. Further, field cores were collected and tested for compressive strength. Field and laboratory evaluation have shown some variability of the FDR base thickness and its compressive strength. Design FDR base thickness was selected for the sensitivity analysis using 2-inch variations according to field core sample thickness.

In Pavement ME Design, a flexible pavement system with an FDR base could be considered to be semi-rigid pavement. The AASHTOWare Pavement ME memorandum, FY2019.4 was issued to address questions regarding the back-calculated modulus and FDR inputs [49]. It was noted that the FDR is not included in the global calibration of the AASHTOWare Pavement ME Design (PMED). It was also found that some State Highway Agencies (SHAs) model the FDR base as a non-stabilized granular layer with a high resilient modulus. Following the other states' practice, the research team modeled the FDR layer as a non-stabilized granular base layer.

The pavement structure and other project input parameters used in the sensitivity analyses are shown in Table 13. The modulus of elasticity of the FDR base was determined from the AASHTO pavement design guide's layer coefficient correlation chart by using the unconfined compressive strength modulus values determined from the field and laboratory samples [50]. The unconfined compressive strength values of the laboratory and field core

samples range from 113 to 480psi. From the conversion chart shown in Figure 21, the unconfined compressive strength values would correspond to a modulus of about 400 to 600ksi.

Table 13. Pavement design input parameters (PMED Version 2.5.4).

Pavement section			
Layer type	Material type	Thickness (in.)	Modulus (psi)
Surface	Asphalt concrete (AC)	4	Any seed modulus
FDR Base	Crushed stone	8 to 12	400,000 to 600,000
Subgrade	A-6	Semi-infinite	4,000 and 8,000
Traffic			
Initial two-way AADTT: 13,500			
Climate			
Latitude and Longitude: 33.5, -84.375			

Table 14 is a summary of pavement distresses predicted for different thicknesses and moduli of the FDR base. The predicted distresses are below the allowed limit by a large margin; this may mean that no major damages are expected to occur during the design life of 20 years. Overall, the effect of design variables – layer thickness and modulus - on pavement distresses is not as significant, which may be due to a strong FDR base that limits the strains induced in upper and lower layers. Figure 22 compares the predicted International Roughness Index (IRI) with varying FDR and subgrade moduli and thicknesses. Although the predicted IRI decreases with increasing modulus and thickness, especially with the subgrade modulus, the predicted IRI is generally insensitive to variations of the modulus and thickness. It should be noted that the global coefficients are used in the analysis.

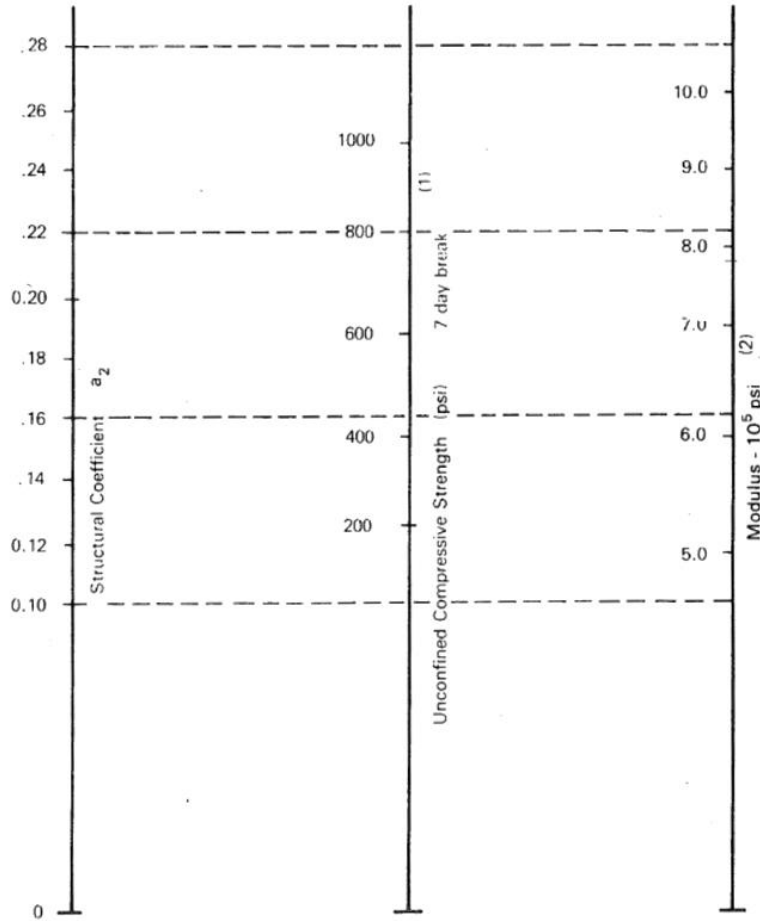
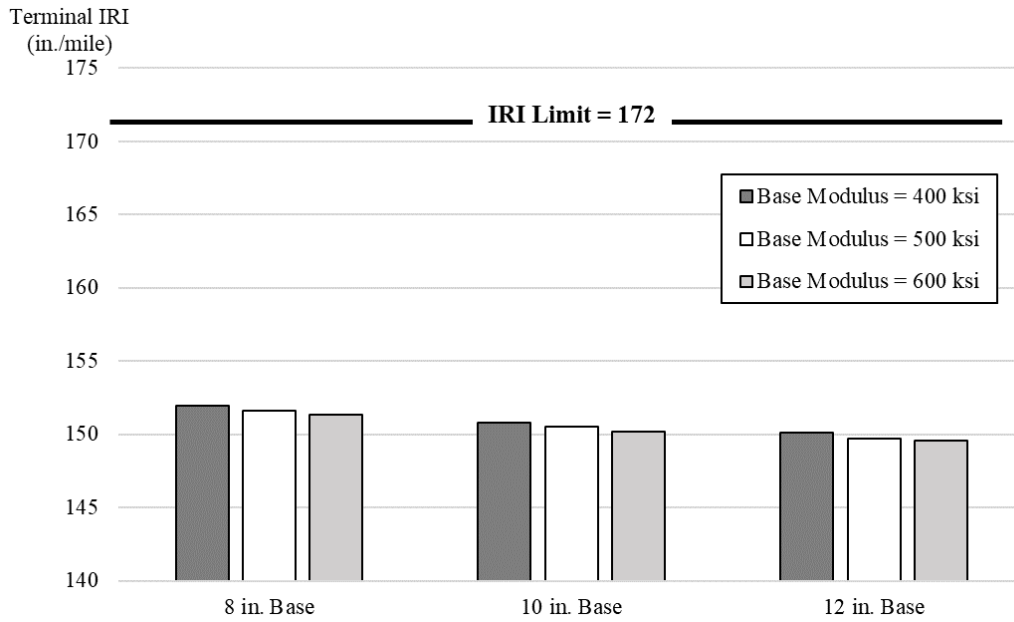


Figure 21. Chart. Modulus and unconfined compressive strength conversion chart – AASHTO [50].

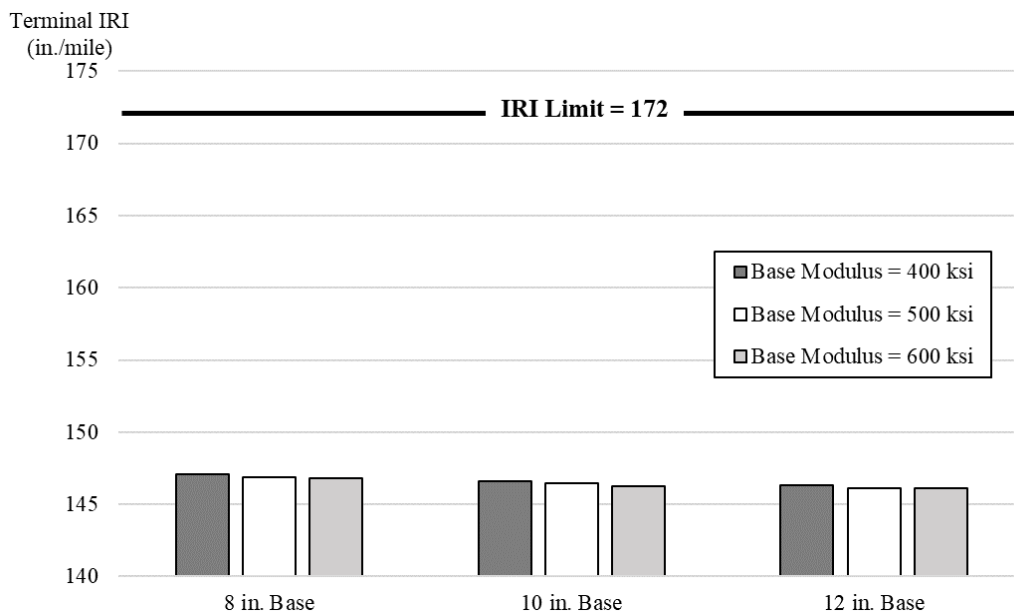
Predicted permanent deformations are shown in Figure 23. As expected, the deformation decreases when the FDR base thickness and base and subgrade modulus increase. This is due to the reduction of the vertical compressive strains in the base and subgrade layer with increased stiffness and thickness. As the vertical strains in the base and subgrade layer decrease, the asphalt concrete layer absorbs most of the vertical strains and therefore the deformation in the asphalt concrete layer increases.

**Table 14. AASHTOWare Pavement ME Design (PMED) analysis summary
(90% target reliability).**

4 in. Asphalt Concrete (AC) Surface, 8 in. FDR Base							
Distress Type	Distress Criteria	Predicted Distress for Varying Subgrade and Base Moduli					
		Subgrade Modulus: 4,000psi			Subgrade Modulus: 8,000psi		
		Base: 400ksi	Base: 500ksi	Base: 600ksi	Base: 400ksi	Base: 500ksi	Base: 600ksi
Terminal IRI (in/mile)	172	152.36	151.96	151.64	147.39	147.18	147
Permanent deformation - total pavement (in)	0.75	0.47	0.46	0.46	0.36	0.36	0.35
AC bottom-up fatigue cracking (% lane area)	25	1.45	1.45	1.45	1.45	1.45	1.45
AC top-down fatigue cracking (ft/mile)	2000	260.13	258.27	257.47	273.22	266.62	263.16
Permanent deformation - AC only (in)	0.25	0.20	0.21	0.21	0.22	0.22	0.23
4 in. Asphalt Concrete (AC) Surface, 10 in. FDR Base							
Distress Type	Distress Criteria	Predicted Distress for Varying Subgrade and Base Moduli					
		Subgrade Modulus: 4,000psi			Subgrade Modulus: 8,000psi		
		Base: 400ksi	Base: 500ksi	Base: 400ksi	Base: 500ksi	Base: 400ksi	Base: 500ksi
Terminal IRI (in/mile)	172	151.2	150.83	150.53	146.89	146.69	146.53
Permanent deformation - total pavement (in)	0.75	0.45	0.44	0.43	0.35	0.35	0.34
AC bottom-up fatigue cracking (% lane area)	25	1.45	1.45	1.45	1.45	1.45	1.45
AC top-down fatigue cracking (ft/mile)	2000	256.76	256.59	256.85	258.96	258.07	259.97
Permanent deformation - AC only (in)	0.25	0.22	0.23	0.24	0.23	0.24	0.25
4 in. Asphalt Concrete (AC) Surface, 12 in. FDR Base							
Distress Type	Distress Criteria	Predicted Distress for Varying Subgrade and Base Moduli					
		Subgrade Modulus: 4,000psi			Subgrade modulus: 8,000psi		
		Base: 400ksi	Base: 500ksi	Base: 400ksi	Base: 500ksi	Base: 400ksi	Base: 500ksi
Terminal IRI (in/mile)	172	150.29	149.94	149.66	146.48	146.3	146.14
Permanent deformation - total pavement (in)	0.75	0.43	0.42	0.41	0.34	0.34	0.33
AC bottom-up fatigue cracking (% lane area)	25	1.45	1.45	1.45	1.45	1.45	1.45
AC top-down fatigue cracking (ft/mile)	2000	256.98	258.38	260.96	260.14	265.01	272.12
Permanent deformation - AC only (in)	0.25	0.24	0.25	0.25	0.25	0.25	0.26

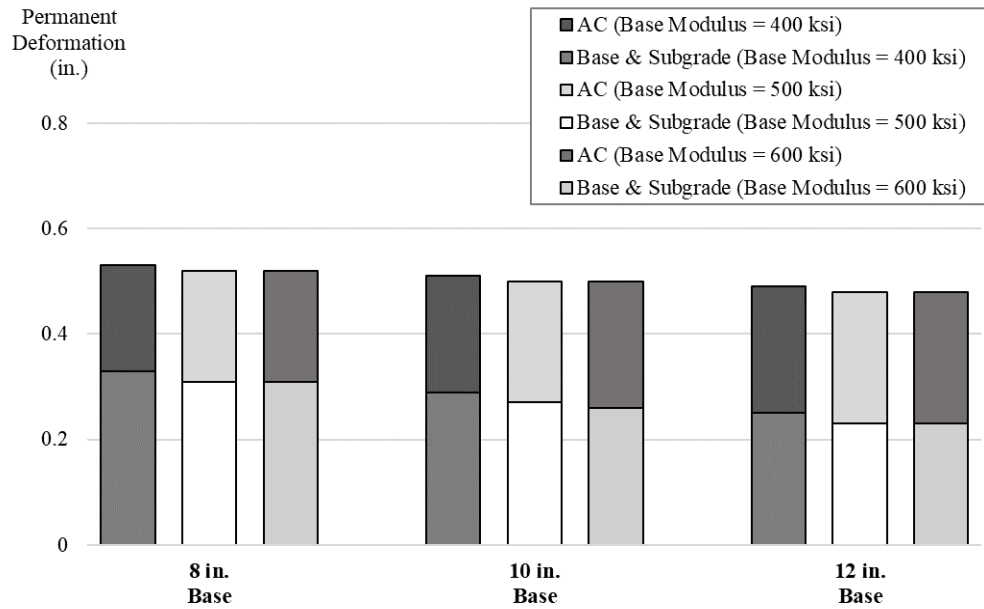


(a) Subgrade resilient modulus 4,000psi

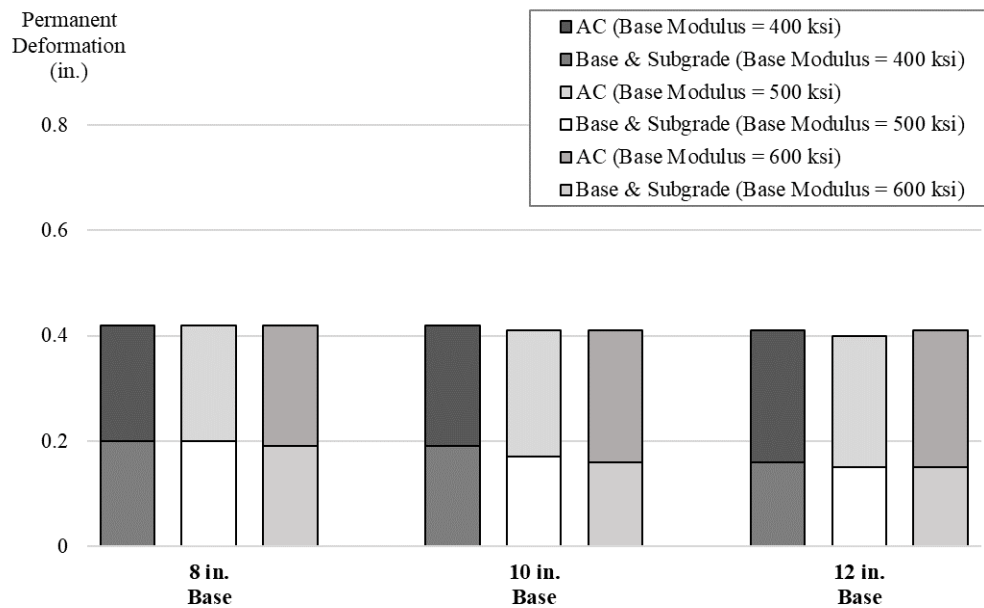


(b) Subgrade resilient modulus 8,000psi

Figure 22. Graphs. Predicted International Roughness Index (IRI).



(a) Subgrade resilient modulus 4,000psi



(b) Subgrade resilient modulus 8,000psi

Figure 23. Graphs. Predicted permanent deformation.

LAYERED ELASTIC ANALYSIS

A multi-layer linear elastic analysis of pavement software, WinJULEA, was used for modeling theoretical responses in the pavement sections included in this sensitivity analysis. Based on the original theories of layered elastic analysis introduced by Burmister, the flexible pavement materials are assumed as linearly elastic, isotropic, and homogeneous while the pavement response (stress or strain) is linearly proportional to the applied load [51], [52]. WinJULEA is a windows-based program of the JULEA (Jacob Uzan Layered Elastic Analysis), the response model integrated into the AASHTOWare Pavement ME software for flexible pavements [53].

The analysis input parameters in the sensitivity analyses are listed in Table 15. The elastic modulus of the asphalt concrete was assumed to be a constant 800ksi for all models. No slippage was assumed at the layer interfaces, and Poisson's ratios for the various materials were selected as 0.35 for the Asphalt Concrete (AC) Surface, 0.2 for the FDR base, and 0.45 for the subgrade. A 9000-lb circular uniformly distributed load is assumed to be acting on the pavement surface.

Table 15. Linear Elastic Analysis input parameters.

Layer Type	Modulus (psi)	Poisson's ratio	Thickness (in.)	Slip
Asphalt Concrete (AC) Surface	800,000	0.35	4	Rough interface
FDR Base	400,000 – 600,000	0.20	8 to 12	Rough interface
Subgrade	4,000 and 8,000	0.45	-	-

Critical pavement responses are computed at the bottom of AC, FDR base, and top of the subgrade foundation. All horizontal strains are used as the critical response for

fatigue cracking, while the vertical strain at the top of the subgrade is treated as rutting potential. The results are presented in Table 16.

The predicted surface deformation is highly affected by the stiffness of the underlying layers. The surface deformation decreases when the FDR base thickness and base and subgrade modulus increase. Similarly, the vertical strains at the top of the subgrade are reduced with increasing base thickness and modulus.

Figure 24 and Figure 25 show plots of the horizontal and vertical strain at the bottom of the asphalt concrete layer. The results show that very little to no tensile strains were found in the analysis. For a pavement with a relatively thin asphalt concrete layer and stiff underlying layer, the tensile strain at the bottom of the asphalt concrete layer becomes a compressive strain. The horizontal strains generated at the bottom of the asphalt concrete layer can be reduced by increasing base thickness and subgrade modulus. Based on the analysis results, it would be prudent to assume that the pavement section may not have significant fatigue cracking issues. However, AC rutting should be considered in asphalt concrete mix design to ensure a longer service life. Further, fatigue cracking in the FDR base layer could be reflected through the top AC layer unless a crack relief layer is placed between two layers. Predicted horizontal stress profiles are shown in Figure 26. The stresses at the bottom of the HMA are compressive while tensile stresses developed at the bottom of the FBR base layer. The results show that the magnitude of tensile stresses increases with the base modulus.

Table 16. WinJULEA Linear Elastic Analysis summary.

4 in. Asphalt Concrete (AC) Surface, 8 in. FDR Base							
Layer Type	Pavement Response ¹	Subgrade Modulus: 4,000psi			Subgrade Modulus: 8,000psi		
		Base: 400ksi	Base: 500ksi	Base: 600ksi	Base: 400ksi	Base: 500ksi	Base: 600ksi
AC surface	Surface deformation (in.)	2.20E-02	2.10E-02	2.02E-02	1.41E-02	1.34E-02	1.29E-02
	Horizontal strain ²	-1.97E-05	-9.75E-06	-2.96E-06	-2.13E-05	-1.18E-05	-5.30E-06
	Vertical strain ²	6.40E-05	5.50E-05	4.91E-05	6.60E-05	5.75E-05	5.18E-05
FDR	Horizontal strain ²	-1.27E-04	-1.09E-04	-9.70E-05	-1.11E-04	-9.63E-05	-8.57E-05
	Horizontal stress (psi) ³	-6.29E+01	-6.80E+01	-7.23E+01	-5.47E+01	-5.95E+01	-6.37E+01
Subgrade	Vertical strain ⁴	3.24E-04	2.85E-04	2.57E-04	2.74E-04	2.42E-04	2.19E-04
4 in. Asphalt Concrete (AC) Surface, 10 in. FDR Base							
Layer Type	Pavement Response ¹	Subgrade Modulus: 4,000psi			Subgrade Modulus: 8,000psi		
		Base: 400ksi	Base: 500ksi	Base: 600ksi	Base: 400ksi	Base: 500ksi	Base: 600ksi
AC Surface	Surface deformation (in.)	1.92E-02	1.83E-02	1.76E-02	1.24E-02	1.18E-02	1.13E-02
	Horizontal strain ²	-1.63E-05	-7.83E-06	-2.15E-06	-1.84E-05	-1.02E-05	-4.59E-06
	Vertical strain ²	6.23E-05	5.49E-05	5.01E-05	6.47E-05	5.76E-05	5.29E-05
FDR Base	Horizontal strain ²	-9.75E-05	-8.39E-05	-7.42E-05	-8.57E-05	-7.42E-05	-6.58E-05
	Horizontal stress (psi) ³	-4.84E+01	-5.21E+01	-5.53E+01	-4.23E+01	-4.59E+01	-4.89E+01
Subgrade	Vertical strain ⁴	2.46E-04	2.16E-04	1.94E-04	2.10E-04	1.85E-04	1.66E-04
4 in. Asphalt Concrete (AC) Surface, 12 in. FDR Base							
Layer Type	Pavement Response ¹	Subgrade Modulus: 4,000psi			Subgrade Modulus: 8,000psi		
		Base: 400ksi	Base: 500ksi	Base: 600ksi	Base: 400ksi	Base: 500ksi	Base: 600ksi
AC Surface	Surface deformation (in.)	1.71E-02	1.62E-02	1.56E-02	1.11E-02	1.05E-02	1.01E-02
	Horizontal strain ²	-1.55E-05	-7.88E-06	-2.85E-06	-1.77E-05	-1.02E-05	-5.15E-06
	Vertical strain ²	6.25E-05	5.61E-05	5.19E-05	6.50E-05	5.86E-05	5.45E-05
FDR Base	Horizontal strain ²	-7.72E-05	-6.63E-05	-5.85E-05	-6.81E-05	-5.87E-05	-5.21E-05
	Horizontal stress (psi) ³	-3.83E+01	-4.12E+01	-4.36E+01	-3.36E+01	-3.63E+01	-3.87E+01
Subgrade	Vertical strain ⁴	1.94E-04	1.69E-04	1.51E-04	1.66E-04	1.45E-04	1.30E-04

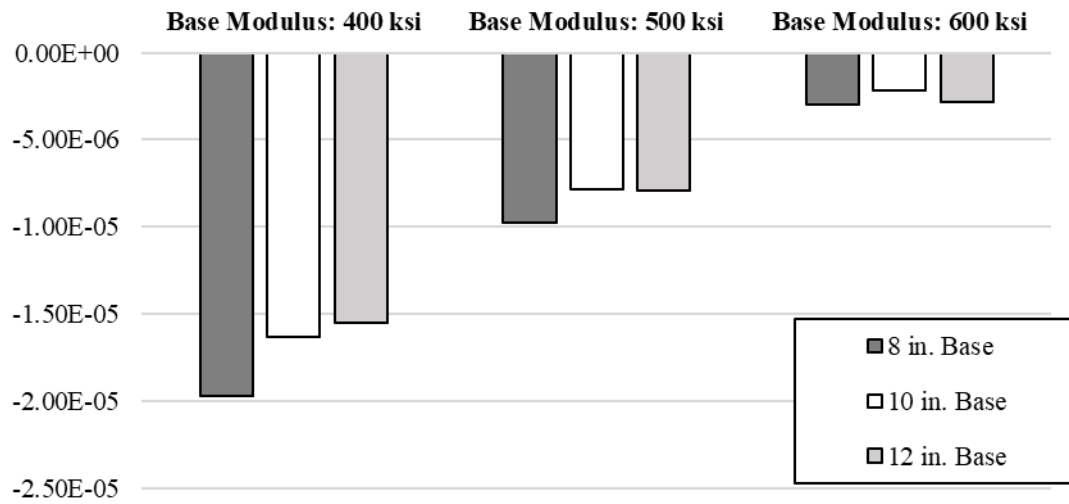
¹ A positive value corresponds to a compressive strain.

² Strains are calculated at the bottom of the layer.

³ Horizontal stresses are calculated at the bottom of the layer.

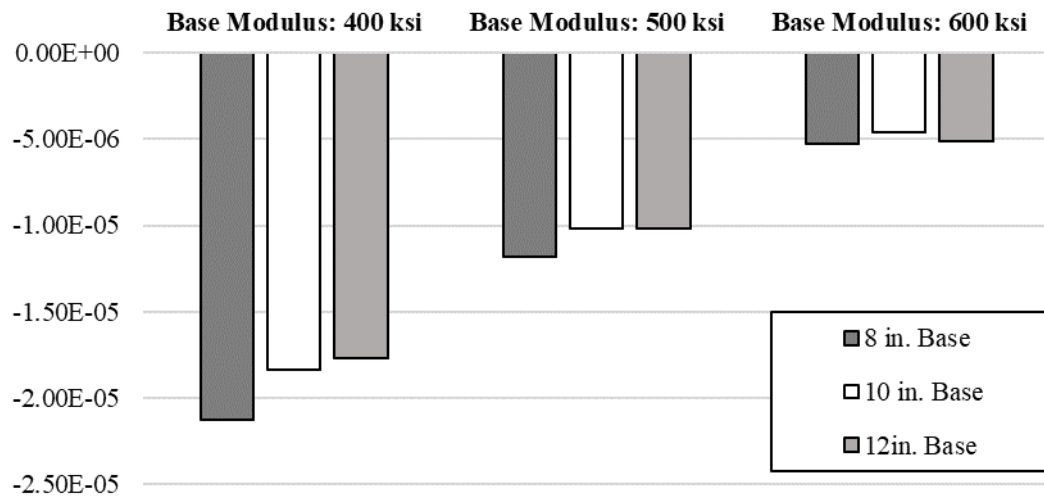
⁴ Vertical subgrade strains are calculated at the top of the layer.

Horizontal strain



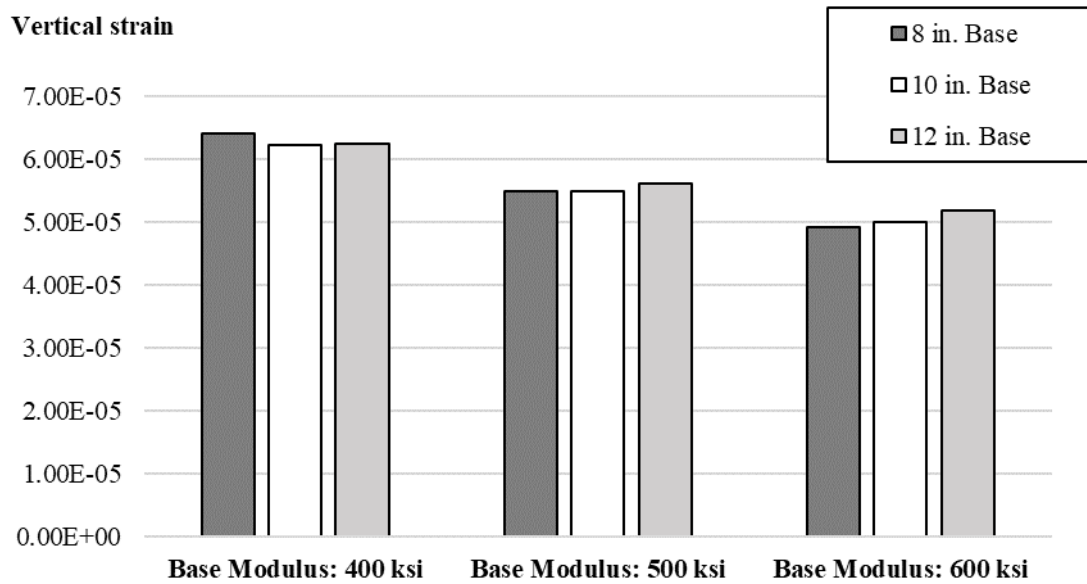
(a) Subgrade resilient modulus 4,000psi

Horizontal strain

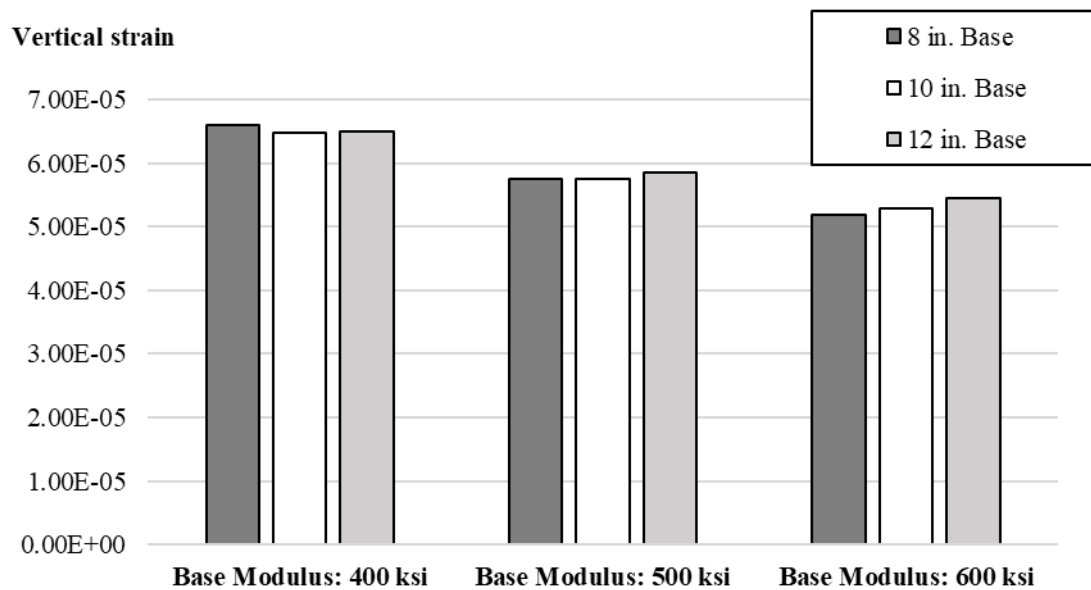


(b) Subgrade resilient modulus 8,000psi

Figure 24. Graphs. Predicted horizontal strain at the bottom AC.



(a) Subgrade resilient modulus 4,000psi

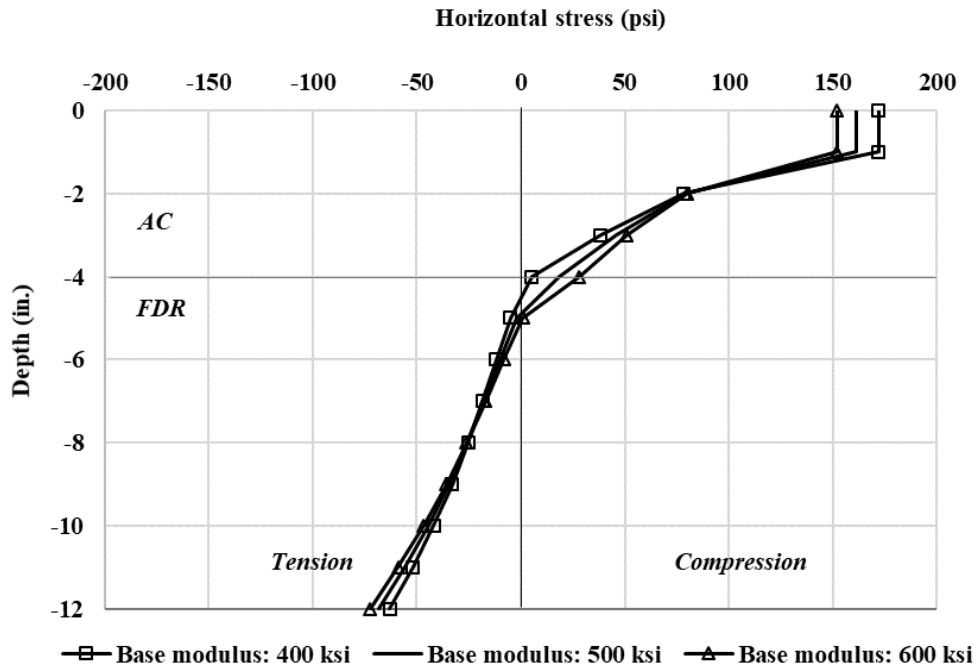


(b) Subgrade resilient modulus 8,000psi

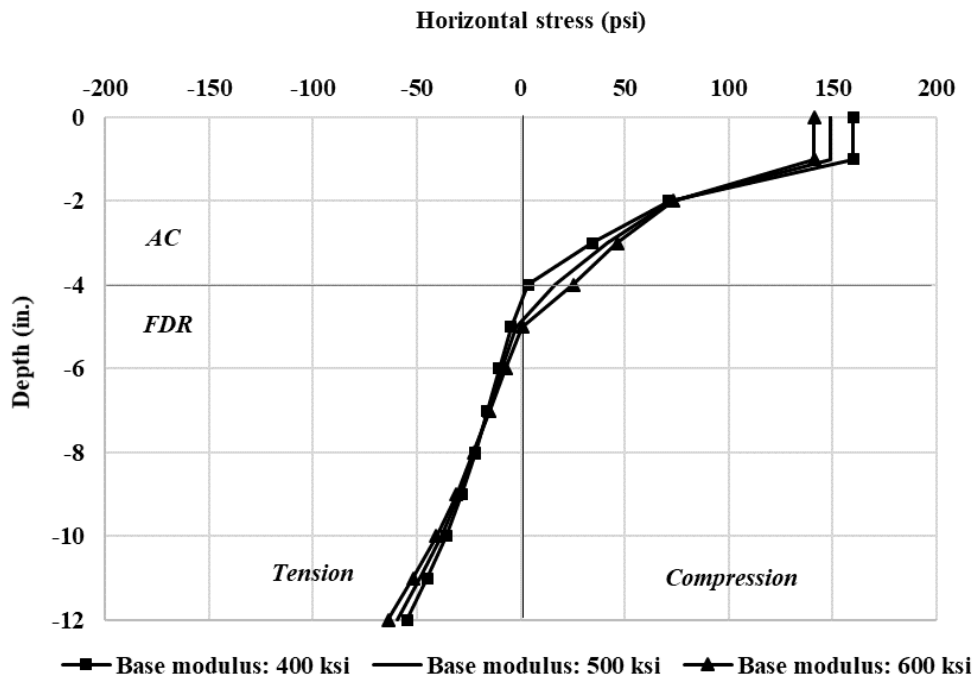
Figure 25. Graphs. Predicted vertical strain at the bottom AC.

The influence of FDR base thickness, modulus and subgrade modulus upon the tensile strain at the bottom of the FDR base shows that the magnitude of tensile strain at the bottom of the FDR base is decreased as base and subgrade modulus are increased. It can also be observed that the impact of the tensile strain at the bottom of the FDR base is directly related to the thickness of the FDR base layer.

The LEA predictions suggest that deep structural maintenance should not be required for flexible pavement sections with the FDR base as most damages are likely to be confined to the pavement surface. Instead, timely surface maintenance work can be more effective in lengthening the service life.



(a) Subgrade resilient modulus 4,000psi



(b) Subgrade resilient modulus 8,000psi

Figure 26. Graphs. Horizontal stress distribution with depth (4 in. AC and 8 in. base).

The allowable number of load repetitions to fatigue failure for ranges of tensile stress are calculated using the National Cooperative Highway Research Program (NCHRP) model (See Figure 27). The results are illustrated in Figure 28. The modulus of FDR base was assumed as 20 percent of the UCS values [54], [55].

$$\log N_f = \frac{0.972 \beta_{c1} - \frac{\sigma_s}{M_r}}{0.0825 \beta_{c2}}$$

Figure 27. Equation. Fatigue cracking in chemically stabilized mixture [56].

Where, N_f : number of repetitions to fatigue cracking of Cement Treated Base

M_r : Modulus of rupture (psi)

σ_s : Tensile stress (psi) at bottom of the layer

β_{c1} and β_{c2} : Calibration factors = 1.0

The prediction model clearly shows the sensitivity of fatigue life of the cement treated aggregate base to the tensile stress at the bottom of the FDR base. Even a small change in the stress could have a significant influence on the fatigue cracking performance of the FDR. Therefore, FDR specifications should be developed to minimize variations in the strength and thickness.

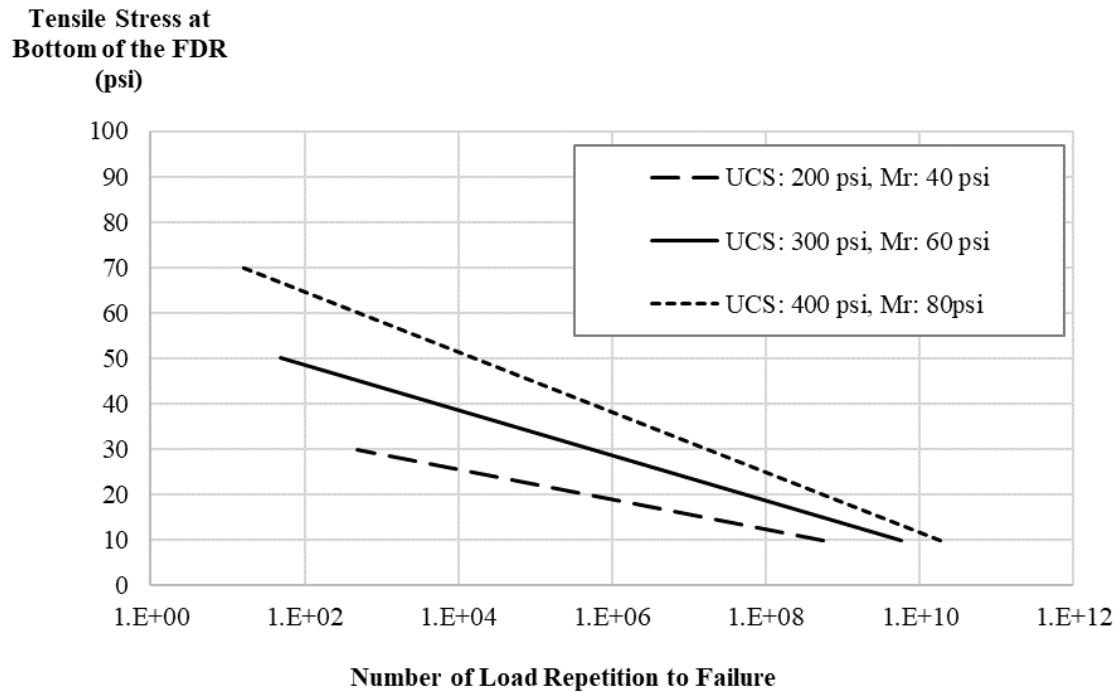


Figure 28. Graph. Number of repetitions to fatigue cracking of cement treated base.

CHAPTER 4. RECOMMENDATIONS

- Pre-mix (pulverization of the roadway prior to spreading Portland cement) is an effective method to adjust moisture content of the parent material, produce a homogeneous mix, and locate soft spots or shallow utility lines. Sufficient moisture addition is one of the most important factors in FDR construction to ensure desired layer strength. Further, any large chunks of asphalt concrete can be removed during the pre-mix stage. A pre-mix should be required in GDOT specification 315 section 3.03.
- UCS values in the cement-treated layer vary significantly by location, which is likely due to the non-uniform contents of Portland cement. The design field compressive strength of 300psi was not met in many locations and a few core samples collapsed during extraction. To reduce the variability in strength, a stricter control on the cement spread rate and the treatment depth would be necessary.
 - The phenolphthalein indicator solution is an alternative way to verify the treatment depth. When the phenolphthalein solution encounters a higher pH material, the color turns a purplish pink indicating that the cement has been incorporated. This technique may allow a quick thickness check via a test hole in the compacted FDR base.
 - Deflection testing with either LWD or FWD should be performed to monitor the stiffness characteristics (or strength gains) of the FDR base during construction.

- Further research is needed to develop design input values, such as modulus of elasticity and modulus of rupture of FDR base, to perform pavement design with FDR using the AASHTOWare Pavement ME Design (PMED) software. Local calibration study is also required to determine NCHRP model (CTA fatigue) calibration coefficients.

ACKNOWLEDGEMENTS

The authors would like to thank Mr. David Jared and Mr. Brennan A. Roney of GDOT for their support and assistance during various stages of the project.

The authors would also like to thank Mr. Philip Snider and Mr. David Gibbs of GDOT for their assistance in conducting field evaluations of FDR sites.

Finally, the guidance and support of Dr. Peter Wu and Mr. Ian Rish of GDOT throughout this research project are sincerely acknowledged and appreciated.

REFERENCES

1. Lewis, D.E., Jared, D.M., Torres, H. and Mathews, M. (2006). *Georgia's Use of Cement- Stabilized Reclaimed Base in Full-Depth Reclamation*, Transportation Research Record vol. 1952, Issue 1, Transportation Research Board of the National Academies, Washington, D.C.
2. Amarh, E.A. (2017). *Evaluating the Mechanical Properties and Long-term Performance of Stabilized Full-Depth Reclamation Base Materials*, M.S. Thesis, Virginia Polytechnic Institute and State University, Blacksburg, Virginia.
3. Wirtgen. (2004). Cold Recycling Manual, 2 Ed. Windhagen: Wirtgen GmbH, Hohner Strasse 2.53578.
4. Georgia Department of Transportation. (2018). *Supplemental Standard Specification Section 315 Cement Stabilized Reclaimed Base Construction*, Georgia Department of Transportation. Atlanta, GA.
5. Caltrans Division of Maintenance. (2013). *Design Guide – Full Depth Reclamation Using Cement*, California Department of Transportation, Sacramento, CA.
6. Virginia Department of Transportation. (2018) *Manual of Instructions (MOI) Section 608.03*, Virginia Department of Transportation, Richmond, VA.
7. Syed, M. (2007). *Full-Depth Reclamation with Portland Cement: A Study of Long-Term Performance*, Portland Cement Association, Skokie, IL.
8. New York Department of Transportation. (2015). *Geotechnical Engineering Manual: Design and Construction Guideline for Full Depth Reclamation of Asphalt Pavement*, New York Department of Transportation, Albany, NY.

9. Ohio Department of Transportation. (2011). *Supplement 1120. Mixture Design for Chemically Stabilized Soils*, Ohio Department of Transportation, Columbus, OH.
10. Pennsylvania Department of Transportation. (2015). *Publication 242 Pavement Policy Manual, Appendix J*, Pennsylvania Department of Transportation, Harrisburg, PA.
11. South Carolina Department of Transportation. (2017). *Standard Method of Test for Sampling, Preparing and Testing of Cement Modified Recycled Base Compression Specimens in the Laboratory (SCT-26)*, South Carolina Department of Transportation, Columbia, SC.
12. Texas Department of Transportation. (2012). *Test Procedure for Preparing Soil and Flexible Base Materials for Testing (TEX-101-E)*, Texas Department of Transportation, Austin, TX.
13. Virginia Department of Transportation. (2016). *Special Provision for Full-Depth Reclamation (FDR) - SP315-000420-00*, Virginia Department of Transportation, Richmond, VA.
14. Portland Cement Association. (2017). *Guide to Full-Depth Reclamation (FDR) with Cement*, Portland Cement Association, Skokie, IL.
15. Texas Department of Transportation. (2019). *Standard Specifications Item 275 Cement Treatment (Road-Mixed)*, Texas Department of Transportation, Austin, TX.
16. Heather, M.J., Spencer, G.W., Rebecca, C.A. and Brad S. (2006). *Evaluation of Cement-Stabilized Full-Depth-Recycled Base Materials for Frost and Early Traffic Conditions*, TFH61-98-00095, Federal Highway Administration, McLean, VA.

17. Tolbert, J.C. (2014). *Effect of High Percentages of Reclaimed Asphalt Pavement on Mechanical Properties of Cement-Treated Base Material*, M.S. Thesis, Brigham Young University, Provo, UT.
18. California Department of Transportation. (2018). *Standard Specification 30-4 Full Depth Stabilization - Cement*, California Department of Transportation, Sacramento, CA.
19. Indiana Department of Transportation. (2017). *Standard Specification 307-R-657 Cement Stabilized Full Depth Reclamation, FDR*, Indiana Department of Transportation, Indianapolis, IN.
20. Ohio Department of Transportation. (2010). *Special Provision Full Depth Reclamation (FDR) Chemical Stabilization*, Ohio Department of Transportation, Columbus, OH.
21. Mississippi Department of Transportation Materials Division. (2010). *Inspection, Testing and Certification Manual MT-25 Design of Soil Cement Mixtures*, Mississippi Department of Transportation, Jackson, MS.
22. Howard, I.L., Sullivan, W.G., Anderson, B.K., Shannon J. and Cost, T. (2013). *Design and Construction Control Guidance for Chemically Stabilized Pavement Base Layers*, FHWA/MS-DOT-RD-13-20, Mississippi State University (MSU), Mississippi State, MS.
23. Texas Department of Transportation. (2013). *Test Procedure for Soil-Cement Testing, Tex-120-E*, Texas Department of Transportation, Austin, TX.
24. Louw, S. and Jones, D. (2015). *Pavement Recycling: Literature Review on Shrinkage Crack Mitigation in Cement-Stabilized Pavement Layers*, University of California Pavement Research Center, Davis, Berkeley.

25. South Carolina Department of Transportation. (2010). *Supplemental Specifications Section 306 Cement Modified Recycled Base*, South Carolina Department of Transportation, Columbia, SC.
26. Mississippi Department of Transportation. (2010). *Special Provision No. 907-499-1 Roadbed Reclamation with Portland Cement*, Mississippi Department of Transportation, Jackson, MS.
27. Morian, D.A., Solaimanian, M., Scheetz, B. and Jahan, S. (2012). *Developing Standards and Specifications for Full Depth Pavement Reclamation*, FHWA-PA-2012-004-090107, Pennsylvania Department of Transportation, Harrisburg, PA..
28. Scullion, T., Sebesta, S., Estakhri, C., Harris, P., Shon, C., Harvey, O. and Rose-Harvey, K. (2012). *Full-Depth Reclamation: New Test Procedures and Recommended Updates to Specifications*, FHWA/TX-11/0-6271-2, Texas Transportation Institute, The Texas A&M University System, College Station, Texas.
29. Abu-Farsakh, M. Alshibli, K., Nazzal M. and Seyman, E. (2004). *Assessment of In Situ Test Technology for Construction Control of Base Courses and Embankments*, FHWA/LA.04/389, Louisiana Transportation Research Center, Baton Rouge, LA.
30. German Institute for Standardization. (1993). *Determining the Deformation and Strength Characteristics of Soil by the Plate Loading Test*, E.V., DIN 18134, Deutsches Institut für Normung (German Institute for Standardization), Berlin, Germany.
31. Gosk, W. (2016). *Stiffness Estimation of the Soil Built-in Road Embankment on the Basis of Light Falling Weight Deflectometer Test*, Procedia Engineering vol. 143, Elsevier B.V., Amsterdam, Netherlands.

32. Missouri Department of Transportation. (1999). *Ground Penetrating Radar (GPR) for Pavement Thickness*, Research Investigation 96-011, Missouri Department of Transportation, Jefferson City, MO.
33. Morey, R. M. (1998). *Ground Penetrating Radar for Evaluating Subsurface Conditions for Transportation Facilities*, Synthesis of Highway Practice 255, National Cooperative Highway Research Program, Washington, D.C.
34. ASTM International. (2015). *ASTM D4748-10: Determining the Thickness of Bound Pavement Layers*, ASTM International, Conshohocken, PA.
35. Evans, R. (2009). *Optimising Ground Penetrating Radar (GPR) to Assess Pavements*, M.S. Thesis, Loughborough University, Loughborough, United Kingdom.
36. Scullion, T. (2005). *Implementing Ground Penetrating Radar Technology within TXDOT*, Texas Transportation Institute, The Texas A&M University System, College Station, Texas.
37. Bigman, D.P. (2018). *GPS Basics: A Handbook for GPR Users*, Bigman Geophysical, LLC, Suwanee, GA.
38. Ulaby, F., More R. and Fung, A.K. (1986). *From Theory to Applications. Appendix E. Microwave Dielectric Properties of Earth Materials*, Microwave Remote Sensing, vol. III, Artech House, Boston, MA.
39. Mayhan R. and Bailey, R. (1975). *An Indirect Measurement of the Effect Dielectric Constant Tangent of Typical Concrete Roadways*, IEEE Transaction on Antennas and Propagation, vol. 23, no. 4.
40. American Association of State Highway and Transportation Officials. (2018). *Standard Practice for Application of Ground Penetrating Radar (GPR) to Highways*,

AASHTO Designation: R 37-04, American Association of State Highway and Transportation Officials, Washington, D.C., 2018.

41. Diefenderfer B. and Apeageyi, A. (2011). *Analysis of Full-Depth Reclamation Trial Sections in Virginia*, VCTIR 11-R23, Virginia Center for Transportation Innovation and Research, Charlottesville, VA.
42. Chris, A., Jordan T. and Vincent, W.S. (2018). “*Georgia FDR Case: Batesville Road and Taylor Milton, GA*, 2018.” Presented at the Full-Depth Reclamation Symposium, Macon, GA.
43. Holzschuher, C., Lee, H. and Greene, J. (2007). *Accuracy and Repeatability of Ground Penetrating Radar for Surface Layer Thickness Estimation of Florida Roadways*, Research Report FL/DOT/SMO/07-505, Florida Department of Transportation, Gainesville, FL.
44. Maser, K. and Carmichael, A. (2015). *Ground Penetrating Radar Evaluation of New Pavement Density*, WSDOT Research Report WA-RD 839.1, Washington State Department of Transportation, Olympia, WA.
45. Saarenketo T. and Scullion, T. (1995). *Using Electrical Properties to Classify the Strength Properties of Base Course Aggregates*, Tech. Rep. 1341-2, Texas Transportation Institute, College Station, TX.
46. Saarenteko, T. (2006). *Electrical Properties of Road Materials and Subgrade Soils and The Use of Ground Penetrating Radar in Traffic Infrastructure Surveys*, Ph.D. Thesis, University of Oulu, Oulu, Finland.

47. Maser, K.R. (2002). *Use of Ground-Penetrating Radar Data for Rehabilitation of Composite Pavements*, Transportation Research Record vol. 1808, Transportation Research Board of the National Academies, Washington, D.C.
48. Thompson M. and Garg, N. (1998). *Mechanistic-Empirical Evaluation of the Mn/Road Low Volume Road Test Sections*, FHWA-IL-UI-262, University of Illinois, Urbana-Champaign.
49. Applied Research Associates. (2019). "AASHTOWare Pavement ME memorandum FY2019.4 - Backcalculated Modulus and FDR Inputs," Applied Research Associates, Champaign, IL.
50. American Association of State Highway and Transportation Officials. (1993). *Guide for Design of Pavement Structures*, AASHTO, Washington, D.C.
51. Burmister, D.M. (1943). *The Theory of Stresses and Displacements in Layered Systems and Application*, Highway Research Board, Washington, D.C.
52. Baker, W.R. and Gonzalex, C.R. (1991). *Pavement Design by Elastic Layer Theory*, Proceedings of the Airfield Pavement Committee, Air Transportation.
53. Applied Research Associates. (2004). *Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures, Appendix GG-1 Calibration of Permanent Deformation Models for Flexible Pavements*, National Cooperative Highway Research Program, Washington, D.C.
54. Kohn, S.D. (1989). *Development of a Thickness Design Procedure for Stabilized Layers under Rigid Airfield Pavements.*" Presented at the Fourth International Conference on Concrete Pavement Design and Rehabilitation, West Lafayette, IN.

55. George, K.P. (1991). *Characterization and Structural Design of Cement-Treated Base*, Transportation Research Record vol. 1288, Transportation Research Board of the National Academies, Washington, D.C.
56. Applied Research Associates. (2004). *Development of the 2002 Guide for the Design of New and Rehabilitated Pavement Structure, Final Report and Software (version 0.70)*, NCHRP, Washington, D.C.