GEORGIA DOT RESEARCH PROJECT 20-14

Final Report

GUIDELINES FOR INCORPORATION OF CEMENT STABILIZED RECLAIMED BASE (CSRB) IN PAVEMENT DESIGN

Georgia Department of Transportation

Office of Performance-based Management and Research
600 West Peachtree Street NW | Atlanta, GA 30308

February 2023

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Youngguk Seo (Co-PI), Ph. D.;</td><td>8. Performing Organization Report No.:
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Kennesaw State University
Civil and Environmental Engineering
655 Arntson Drive, Marietta, GA 30060
Phone: (470) 578-5080
E-mail: jkwon9@kennesaw.edu</td><td>10. Work Unit No.:
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GDOT is in the process of implementing an updated Mechanistic-Empirical Pavement Design Guide (MEPDG). To support this implementation, a research study was conducted to calibrate the national performance models for local conditions. However, reliable calibration coefficients could not be derived for semi-rigid pavements due to the lack of sufficient performance data. GDOT will therefore continue to utilize the current pavement design procedure (AASHTO 72/93) until appropriate MEPDG local calibration coefficients have been identified. This research project was undertaken to improve the reliability of the current GDOT pavement design procedure for CSRB and to provide recommendations regarding the steps required to verify and calibrate CSRB for use in MEPDG. A preliminary laboratory study of typical CSRB mix and field cores was conducted to characterize the CSRB materials and evaluate the accuracy of the relationship between elastic modulus and unconfined compressive strength, after which performance data was collected for samples from 4 different sites in Georgia. The FWD deflection data and UCS values of the field cores were then used to calculate the structural layer coefficient of the CSRB layer and a sensitivity analysis was performed to identify the input variables with the greatest influence on the performance predicted by the PMED. Two different pavement types were used to model the FDR pavement: flexible pavement and semi-rigid pavement. Finally, a data collection plan was developed to guide the collection of the data needed for local calibrations of the MEPDG for roads in Georgia.</td></tr>
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GDOT Research Project 20-14

Final Report

GUIDELINES FOR INCORPORATION OF CEMENT STABILIZED RECLAIMED BASE (CSRB) IN PAVEMENT DESIGN

By
Jayhyun Kwon, Ph.D., P.E.
Associate Professor – Department of Civil and Environmental Engineering

And

Youngguk Seo, Ph.D.
Associate Professor – Department of Civil and Environmental Engineering

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Contract with
Georgia Department of Transportation

In cooperation with
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Federal Highway Administration

February 2023

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)		
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
		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)		
^oC	Celsius	1.8C+32	Fahrenheit	^oF
		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)

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LIST OF ABBREVIATIONS

AADTT	Annual Average Daily Truck Traffic
AASHTO	American Association of State Highway and Transportation Officials
AC	Asphalt Concrete
ASTM	American Society for Testing and Materials
COV	Coefficient of Variation
CSRB	Cement Stabilized Reclaimed Base
CTB	Cement Treated Base
CSA	Cement Stabilized Aggregate
E	Elastic Modulus
FDR	Full Depth Reclamation
FHWA	Federal Highway Administration
FWD	Falling Weight Deflectometer
GAB	Graded Aggregate Base
GDOT	Georgia Department of Transportation
HMA	Hot Mix Asphalt
IRI	International Roughness Index
ISM	Impulse Stiffness Modulus
MAAT	Mean Annual Average Temperature
MOP	Mechanistic-Empirical Pavement Design Guide: A Manual of Practice
MoR	Modulus of Rupture
NCHRP	National Cooperative Highway Research Program
NDT	Nondestructive Testing
OMC	Optimum Moisture Content
PCA	Portland Cement Association
PM	Plastic Mold
PMED	AASHTOWare Pavement ME Design
SN	Structural Number
SR	State Route
UCS	Unconfined Compressive Strength

EXECUTIVE SUMMARY

Full Depth Reclamation (FDR) with cement stabilization is a cost-effective pavement rehabilitation strategy that improves structural capacity and reduces the cost of both materials and haulage. Pavement constructed with a cement stabilized layer is considered to be semi-rigid. However, although many state highway agencies across the U.S. have adopted the Mechanistic-Empirical Pavement Design Guide (MEPDG), not all have identified local calibration coefficients for their semi-rigid pavement.

This report provides a summary of the current practices related to the design of FDR projects in the US. A review of the State highway agencies' construction specifications and pavement design manuals revealed target UCS values for the cement stabilized reclaimed base (CSRB) materials utilized range from 200 psi to 800 psi, and the design structural layer coefficients of the CSRB itself range from 0.14 to 0.35. State highway agencies that use Pavement ME Design (PMED) model the cement stabilized aggregate base as a high stiffness unbound aggregate base layer, as the distress models in the PMED have been fully calibrated for a semi-rigid pavement. The modulus values assigned to the Cement Treated Aggregate (CTA) or Cement Stabilized Aggregate (CSA) varies from 25,000 psi to 1,000,000 psi. A resilient modulus is used to model CSRB in conventional flexible pavement, while an elastic modulus is used for CSRB in rigid pavement.

The laboratory study was conducted to characterize the CSRB in pulverized asphalt/base soil samples collected from four ongoing FDR projects in Georgia, extracting CSRB field cores from FDR pavement sections aged from 3 to 15 years. The results of the subsequent analysis revealed considerable variation in the Elastic Modulus (E) and

Unconfined Compressive Strength (UCS) of these field cores, likely due to the variation in reclaimed base mixture qualities such as gradation, mineralogy, and aggregate physical properties. A comparison between the measured data and existing E-UCS models of the chemically stabilized materials indicates that the common soil-cement model is inadequate for determining the elastic modulus of the CSRB.

The results of the Falling Weight Deflectometer (FWD) tests show the overall stiffness of the pavement sections decreases with age. Structural layer coefficients computed from the UCS correlations suggest a structural layer coefficient of 0.22 for CSRB.

A PMED sensitivity analysis was also conducted, where the pavement with CSRB was modeled as both conventional flexible pavement and semi-rigid pavement. The results revealed that the predicted distresses are largely insensitive to the CSRB properties and the interface bonding conditions between the Asphalt Concrete (AC) and CSRB. However, the cracking performance in the chemically stabilized layer was found to be closely linked to the magnitude of the CSRB modulus.

CHAPTER 1. LITERATURE REVIEW

INTRODUCTION

Full Depth Reclamation (FDR) refers to the in-situ pulverization of existing pavement that is then mixed with appropriate stabilizing agents such as cement, foamed asphalt or hydrated lime to create a new base layer. In Georgia, Portland cement has been widely used as the stabilizing agent for over 15 years and the state's in-service pavements constructed using Cement Stabilized Reclaimed Base (CSRB) have performed well to date. The CSRB mix design uses representative field samples to determine the amount of water and cement required to achieve the target Unconfined Compressive Strength (UCS) for its intended use.

Although the American Association of State Highway and Transportation Officials (AASHTO) has recently released a new update for its Mechanistic-Empirical Pavement Design Guide, several state and local highway agencies in Georgia are still using the method recommended in AASHTO72/93 when designing local roads and streets [1-4]. The UCS of the CSRB is an essential design input in both the empirical AASHTO72/93 design, where it is used to determine a layer coefficient, and the Mechanistic-Empirical Pavement Design, where it is used to determine the modulus of elasticity of the CSRB.

The Georgia Department of Transportation (GDOT), which currently uses the 1972 AASHTO Interim Guide when designing pavements, is in the process of implementing the methods recommended in AASHTO's Mechanistic-Empirical Pavement Design Guide: A Manual of Practice (MOP) [2, 3, 5, 6]. AASHTO has also released guidelines for the local calibration of MEPDG implementations based on a study by the National Cooperative Highway Research Program (NCHRP 1-40B) that recommends a series of steps for the calibration of MEPDG based on local conditions and materials [7]. As part of the implementation of the MEPDG, an earlier research study was conducted to calibrate the nationally calibrated performance models for local conditions [5,6].

Using the AASHTOWare Pavement ME Design (PMED), the CSRB can be modeled either as a Cement Treated Base (CTB) or as an unbound aggregate layer [1]. However, it is not possible to derive reliable calibration coefficients for semi-rigid pavement due to the lack of real-world performance data [5,6]. An extensive review of the available sources is thus required to develop an adequate MOP input and historical pavement performance database for calibration. In the meantime, GDOT is forced to continue with the current pavement design procedure until MOP local calibration coefficients become available. The purpose of this research is, therefore, to evaluate the mechanical properties of the CSRB and assess the suitability of the structural layer coefficients assigned to CSRB for local conditions.

CURRENT STATE OF PRACTICE

The State Department of Transportation construction specifications and pavement design manuals from other states were reviewed to gather information about their current practice related to the design of FDR projects. Many state transportation agencies are actively engaged in implementing the MEPDG or have already adopted it. However, Georgia is by no means alone in continuing to use AASHTO 72/93 methodology for the design of the state's pavement [2,6]. The main findings of this review are summarized in this section.

1972/1993 AASHTO Methodology

The 1972 Interim Guide and the final AASHTO 93 Design Guide use structural layer coefficients for the different materials in the design of pavement structures. These structural layer coefficients are regression coefficients developed from empirical models based on AASHO road tests. The AASHTO 93 Design Guide provides a correlation chart to determine the appropriate layer coefficient for a cement treated aggregate based on its UCS and modulus values, as shown in Figure 1.

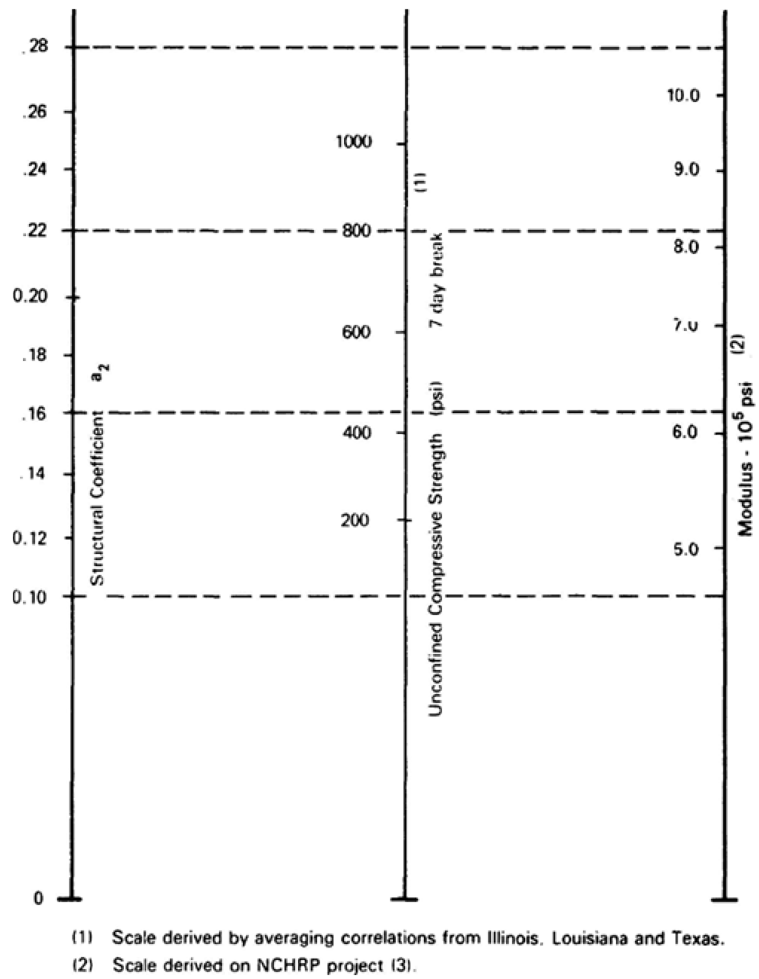


Figure 1. Chart. Correlations between structural layer coefficient and various strength and stiffness parameters for cement-treated granular bases [4].

State highway agencies that use FDR with cement must specify the minimum and maximum acceptable UCS values for the Cement Treated Aggregate (CTA) or Cement Stabilized Aggregate (CSA) they are using. These required UCS values range from 200 psi

(Ohio's minimum value) to 800 psi (New York's maximum value), as reported in the previous study [8]. Looking at the data presented in Figure 1, these typical UCS values correspond to structural layer coefficients ranging from 0.13 to 0.22. The structural layer coefficient or elastic moduli assigned to the chemically treated base course by various agencies are summarized in Table 1.

It is important to note that not all the state highway agencies follow the 1993 guidelines when determining the structural layer coefficients. The cement treated (no soil-cement) layer was first included in the 1972 Interim Guide, where three different layer coefficients were proposed for a cement treated layer, depending on the 7-day UCS value range of that layer. For example, a cement treated layer with a UCS value between 400 and 650 psi has a layer coefficient of 0.2, while a cement treated layer with a UCS of less than 400 psi is assigned a structural layer coefficient of 0.15. According to the correlation table provided in the AASHTO 93 Design Guide, however, the compressive strength of the cement treated base needs to be 650 psi to be assigned a layer coefficient of 0.2.

In Georgia, the mix design is required to achieve a field compressive strength of 300 psi for the cement stabilized reclaimed base; GDOT assigns its cement stabilized graded aggregate a layer coefficient of 0.22. In order to assign cement stabilized reclaimed base the same layer coefficient as cement stabilized graded aggregate, a 7-day UCS of around 800 psi is considered an appropriate target value for the mix design, based on the data presented in Figure 1.

Table 1. Structural layer coefficients and elastic moduli for the chemically stabilized layers

State	Structural layer coefficients	Modulus (psi)
AZ	Cement treated or Bituminous treated base: 0.28 Cement or lime treated subgrade 0.23	N/A
CO	N/A	Cement stabilized aggregate: 100,000 psi Soil cement: 25,000 psi
FL	Soil cement (UCS 500 psi): 0.20 Soil cement (UCS 300 psi): 0.15	N/A
GA	Cement stabilized graded aggregate: 0.22 Soil cement: 0.20	N/A
LA	Soil cement (stabilized - UCS 300 psi): 0.14 Soil cement (treated – UCS 150 psi): 0.10	N/A
MD	Soil cement: 0.15-0.25 (desired coefficient: 0.20)	Maximum modulus: 750,000 psi Minimum modulus: 150,000 psi
MI	Cement stabilized base: 0.26	Cement stabilized base: 1,000,000 psi
MS	Cement stabilized FDR, soil-cement base layers: 0.2	N/A
NC	Cement Treated Base Course (CTBC): 0.23	N/A
OH	Chemical stabilized aggregate base: 0.17	N/A
PA	FDR chemical stabilization: 0.32-0.35	N/A
SC	Cement stabilized earth base: 0.25 Cement modified recycled base: 0.26 Cement stabilized aggregate base: 0.34	N/A
TX	N/A	Cement treated base (minimum 7-day UCS of 300 psi): 80,000 - 150,000 psi
UT	Cement stabilized aggregate: 0.16 to 0.20	Cement stabilized aggregate (7-day UCS of between 400 psi and 500 psi): 1,000,000 psi
VA	N/A	Chemically stabilized layers like CTA and FDR shall be modelled as non-stabilized base layers. Modulus value used for FDR is 80,000 psi for flexible pavement
PCA	Moderately strong FDR layer: 0.26 Low strength FDR layer: 0.20	Moderately strong FDR layer (7-day UCS of 400 psi): 730,000 psi Low strength FDR layer (7-day UCS of 200 psi): 516,000 psi

A 7-day compressive strength of around 300 psi corresponds to a layer coefficient of 0.14. The Florida and Louisiana Departments of Transportation assign layer coefficients of 0.15 and 0.14, respectively, to a soil cement layer with a 7-day compressive strength of 300 psi. Prior to its implementation of PMED in 2018, the Virginia Department of Transportation used the AASHTO 93 Design Guide, assigning a layer coefficient of 0.20 to the cement treated aggregate base layer and requiring an equivalent 7-day compressive strength of more than 600 psi [9].

Hossain et al. reported that the strength of cement treated aggregate (CTA) depends on properties of the aggregate in the mix such as gradation, angularity, and surface texture [9]. As such, the structural layer coefficients of the cement stabilized (aggregate) base are generally higher than that of the soil cement, while the target UCS is also generally higher than that of the soil cement. Table 2 compares the gradation requirements for the graded aggregate, CSRB, and soil cement. CSRB has a coarser gradation than soil cement. A properly mixed CSRB would have similar mechanical properties to a cement stabilized graded aggregate.

Table 2. Particle size distributions in graded aggregate, CSRB, and soil cement

Sieve size	Percent passing by weight			
	Graded aggregate (Group 1)	Graded aggregate (Group 2)	CSRB	Soil cement
3 in. (75mm)			100	
2 in. (50 mm)	100	100		
1-1/2 in. (37.5 mm)	97-100	97-100		100
¾ in. (19.0 mm)	60-95	60-90		
No. 4 (4.75 mm)			55	80
No. 10 (2 mm)	25-50	25-45		
No. 60 (250 mm)	10-35	5-30		
No. 200 (75 mm)	7-15	4-11		
GDOT Specification	**Section 810**		**Section 315**	**Section 301**

The Mechanistic-Empirical Pavement Design Guide: A Manual of Practice

State highway agencies across the US have either already implemented PMED or are in the process of developing local calibrations for its implementation. In the PMED edition used in this study (version 2.6.0), the use of CSRB as a stabilized layer for flexible designs is not provided as an option and it must be modeled as a non-stabilized base material. For rigid and semi-rigid designs, the CSRB layer is modeled as a chemically stabilized layer; chemically stabilized materials with cement can be either cement treated aggregate or soil cement. Table 3, which is reproduced from the MEPDG MOP, shows the input properties with their test protocols and recommended input Level 2 and 3 parameters and values [1].

Table 3. Input properties and source of data for chemically stabilized materials

Material type	Property	Source of data		
		Test for Level 1 input	Estimate using Levels 2 and 3	
			Conversion	Typical value
Cement treated aggregate	Elastic modulus	ASTM C 469	$E = 57{,}000\ (f_c)^{0.5}$ AASHTO T22	1,000,000 psi
	Flexural strength	AASHTO T 97	Use 20% of the compressive strength	• Used as base: 750 psi • Used as subbase, select material, or subgrade: 250 psi
	Poisson's ratio	N/A		Typical value: 0.1 to 0.2
Soil cement	Elastic modulus	N/A	$E = 1200\ q_u$ ASTM D1633	500,000 psi
	Flexural strength	ASTM D 1635	Use 20% of the compressive strength	• Used as base: 750 psi • Used as subbase, select material, or subgrade: 250 psi
	Poisson's ratio	N/A		Typical value: 0.15 to 0.35

Note: Unconfined compressive strength (fc' or q_u) in psi

The compressive strength is a crucial value for pavement design as it determines the elastic modulus and flexural strength of the chemically stabilized materials incorporated in the structure. The modulus and compressive strength relationships found in the AASHTO 93 Design Guide and the MEPDG Manual of Practice are plotted together in Figure 2. Interestingly, for the same compressive strength, the modulus and compressive strength relationship for cement treated aggregate predicts a higher modulus than that for the soil cement.

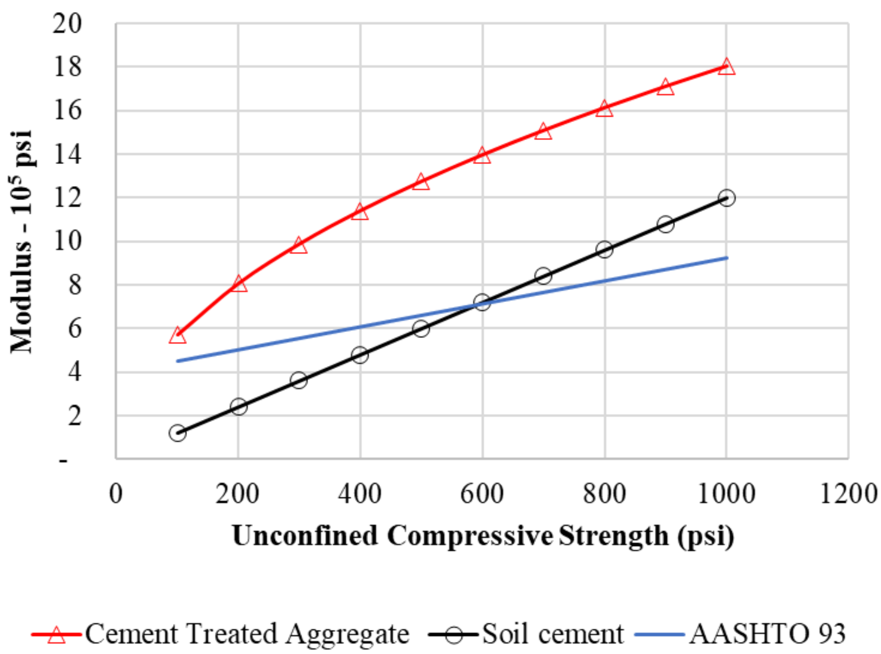


Figure 2. Graph. Comparison of the relationships between compressive strength and moduli from in the AASHTO 93 Design Guide and the MEPDG Manual of Practice.

A study examining the published modulus and compressive strength correlations for over 500 cores of chemically stabilized samples obtained from the Mississippi Department of Transportation highway network found that the modulus and UCS correlation equation for soil cement slightly underpredicted the modulus at low UCS, while the correlation equation for the cement treated aggregate meaningfully overpredicted its modulus [10]. However, the researchers concluded that the AASHTO relationship between the layer coefficient, modulus and UCS presented in Figure 1 provides reasonable modulus predictions for UCS values between 300 and 600 psi.

CHAPTER SUMMARY

- State highway agencies that use FDR with cement specify minimum and maximum UCS values for the CSRB. The required UCS values range from 200 psi to 800 psi.
- A minimum 7-day UCS of 300 psi is required for cement stabilized FDR in Georgia. This UCS value corresponds to a layer coefficient of 0.14 using the AASHTO 93 guideline correlation chart or a layer coefficient of 0.15 according to the AASHTO 72 Interim Guide.
- The Florida and Louisiana Departments of Transportation both require 7-day UCS values of 300 psi for the soil cement layer. Structural layer coefficients of 0.15 and 0.14 are assigned for the soil cement layer. The Virginia Department of Transportation assigns a layer coefficient of 0.20 to the cement treated aggregate base layer, with a minimum 7-day compressive strength of 600 psi.
- In general, cement stabilized aggregate has higher UCS and layer coefficients than those for soil cement. This could be due to the difference in strength between aggregate material and soil.

- The effect of parent materials in the mix is also shown in the UCS-modulus conversion equations included in the MEPDG Manual of Practice. Cement treated aggregate generally has a higher modulus than soil cement for the same UCS.
- The modulus value used for the cement stabilized layer by state highway agencies that use mechanistic pavement design procedures (either PMED or an agency specific pavement design method) varies from 25,000 psi for a soil cement layer in Colorado to 1,000,000 psi for cement treated aggregate in Michigan. In Virginia, a resilient modulus of 80,000 psi is assigned for cement treated aggregate if the pavement is modeled as conventional flexible pavement. If cement treated aggregate is used in rigid pavement as a chemically stabilized layer, an elastic modulus of 750,000 psi is used.

CHAPTER 2. LABORATORY EXPERIMENTS

To characterize the CSRB materials, a series of laboratory tests was conducted on specimens fabricated in the laboratory and cores extracted from various field sites across the state. The sources of the CSRB specimen materials selected are described in this chapter, along with the standard laboratory testing regime chosen to evaluate their fundamental engineering properties. The results of the analyses are presented and their implications for the validation and implementation of the design inputs currently used for CSRB in Georgia are considered.

FABRICATION OF LABORATORY SPECIMENS

For the laboratory specimens, pulverized asphalt/base soil samples were collected from two ongoing Full Depth Reclamation (FDR) projects in Georgia, the first located at Peachtree City (two separate road sections, designated PTC1 and PTC2) and the second at Winder (one section designated Winder). These representative raw samples are typical of the in-situ materials commonly used in the state that are graded, stabilized, and compacted to form CSRB. During the fabrication of the laboratory specimens, every effort was made to reproduce the construction steps used in the field as closely as possible. The pulverized samples were delivered to the laboratory at Kennesaw State University (KSU) and graded according to GDT 49 [11].

This process often involved the removal of large asphalt concrete chunks through screening. The graded samples were stabilized via mixing with Type 1 Portland cement and water. Four target cement contents were selected: 3, 5, 7, and 9% by dry weight of mix, equivalent to 30, 45, 65, and 83 pounds per square yard, respectively, for a layer depth of

10-inches. Water was added to achieve the optimum moisture content estimated for each cement content, as stated in the respective mix design reports. Table 4 presents the moisture content of each sample batch.

Table 4. Moisture content of the CSRB mixes

Cement content (by weight)	PTC1	PTC2	Winder
3%	-	10.82%	14.35%
5%	11.36%	11.20%	15.05%
7%	10.28%	10.43%	14.81%
9%	11.09%	10.09.%	14.83%

The final step was to mold and compact the processed CSRB mixes into cylindrical or prismatic specimens. Figure 3 shows the three different types of molds used to fabricate the specimens, two of which are cylindrical molds and one a rectangular beam mold. The standard mold shown in Figure 3a produces a 4" diameter cylinder that is 4.6" high. The Plastic Mold (PM) is designed to cast an 8” tall cylinder with the same diameter, 4", which is unconventional for CSRB materials. Having two types of cylinders to test and compare enabled us to verify the utility and practicality of PM for CSRB materials and examine the effect of size on material properties. The third mold type is a rectangular steel prism that forms a 14" long prismatic specimen with a 4" square cross-section. All the prismatic specimens were loaded as beams during the laboratory testing. After fabrication, most of the specimens were placed in freezer bags or wrapped with plastic and cured at room temperature in covered water baths for 7 days; some of the PTC2 beams were cured for 28 days for comparison purposes.

(a) (b) (c)

Figure 3. Photos. Sample fabrication with three mold types: (a) standard steel proctor mold (GDT65); (b) Plastic Mold (PM), (AASHTO PP 92-19); and (c) rectangular beam mold (ASTM C78) [12-14].

FIELD CSRB CORES

Field cores are essential for understanding the level and distribution of quality in CSRB materials, supplementing the findings from laboratory specimens. A total of 26 six-inch diameter cores were extracted from four existing FDR pavement sections that have been in service for some years (Table 5). Samples with ages varying from 3 to 15 years were collected, with an equal number of cores being taken from each carriageway direction (northbound or southbound). A condition assessment of these cores revealed that the CSRB layer was at least twice as thick as the surface asphalt concrete layers apart from in four of the field cores from the Smiley Cross Rd. that contained no CSRB, possibly due to a layer separation due to coring or severe moisture-induced erosion experienced over time. As these core samples were not testable, they were removed from the testing program. All the eligible cores were cut and trimmed at the top and bottom of the CSRB layer using a masonry saw to ensure a uniform contact between the loading blocks and the specimen surfaces during testing. Photographic images of the cores are provided in Appendix A.

Table 5. Field cores from FDR pavement sections

Road name	Construction	Core No.	Direction	Thickness (in.)		Notes
				AC	*Base*	
SR 70 – Fulton County	2018	1	Southbound	5	15	
		2	Southbound	6	11.5	
		3	Southbound	5.75	10.5	significant voids in AC
		4	Northbound	5.25	13.5	
		5	Northbound	5	14.75	
		6	Northbound	6.25	13.5	
			Average	**5.5**	**13.1**	
Metcalf Road – Thomas County	2015	1	Southbound	3	10	
		2	Southbound	3.25	10	AC separated from Base
		3	Southbound	3.75	9	
		4	Northbound	3.75	10.5	
		5	Northbound	3.5	10.25	AC separated from Base
		6	Northbound	3.75	12	AC separated from Base
			Average	**3.5**	**10.3**	
Senoia Road – Fayette County	2015	1	Southbound	3.5	10	
		2	Southbound	7	N/A	No CSRB
		3	Southbound	4.5	11.75	
		4	Northbound	4.5	11	
		5	Northbound	4.25	10	
		6	Northbound	3.5	13	
			Average	**4.5**	**11.2**	
Moody Bridge Road (now Smiley Cross Road) – Long County	2004	1	Southbound	1.75	8.25	
		2	Southbound	1.5	8.5	
		3	Southbound	4	0	No CSRB
		1	Northbound	3.5	0	No CSRB
		2	Northbound	1.5	7.5	
		3	Northbound	7.5	0	No CSRB
		4	Northbound	1.5	8.5	
		5	Northbound	3.75	0	No CSRB
			Average	**3.1**	**4.1**	

LABORATORY TESTING

Laboratory tests were performed to investigate the three fundamental engineering properties of the CSRB material collected: Unconfined Compressive Strength (UCS), Modulus of Elasticity (E), and Flexural Strength (or Modulus of Rupture, MoR). Table 6 lists the in-house experiments used, the number of samples examined for each, and the standards and guides referenced. Three replicate specimens (six for the PM specimens) were tested for each core sample to determine the variability of the experimental measurements. A compression machine (Humboldt, 300,000-pound loading capacity) was loaded at a speed of 0.05 in/min, with an additional loading jig used where necessary to capture the strain in the center of a cylinder for E or to redirect the vertical movement of the crosshead to bend a beam for MoR. The temperature was not continuously monitored and controlled but can reasonably be assumed to be 23±2.0 ºC throughout.

Table 6. Testing program implemented for laboratory specimens and field cores

Test	**Cement %**				**No. of field cores**	**Sample preparation references**
	3%	*5%*	*7%*	*9%*		
Unconfined Compressive Strength (UCS)	9	9	9	9	27	GDT 65
Modulus of Elasticity (E)	3	3	3	3	9	AASHTO PP 92*
Flexural Strength (Modulus of Rupture, MoR)	-	-	3	3	-	ASTM C78

* UCS tests on 8" tall cylinders

The UCS is widely accepted for the design of many transportation materials. In particular, the 7-day UCS (the UCS for 7-day old specimens) appears in many of the prediction models used for chemically stabilized materials. In this study, the 7-day UCS was obtained for two types of cylinders and four different cement contents (3, 5, 7, and 9 %) to characterize the CSRB materials, as per ASTM D1633 [15]. Each specimen was subjected to uniaxial compression loading until fracture, after which its UCS was calculated by dividing the maximum load by the cross-sectional area of the cylinder. The same test setup and data analysis method were applied when testing the UCS of the field cores.

The modulus of elasticity (E) measures the stiffness of a cylinder within the elastic region of a stress/strain curve in accordance with ASTM C469 [16]. To ensure the tests were performed within the elastic response zone for each cylinder, the applied peak stress was limited to 40% of the average UCS measured for each cement content and each field location. Each cylinder was mounted in a loading jig and the pressure raised in increments of 100 psi until the peak load was reached. At each load level, the strain was calculated based on readings from digital gauges attached to the loading jig. Once the stress-strain curve had been developed for each sample, a secant modulus - a straight line slope connecting two points on the curve - was selected as E.

A flexural strength test was performed to determine the Modulus of Rupture (MoR) – an indicator of flexural strength - for the beams utilizing a third point bending measurement. Only two high cement content samples could be tested (7 and 9%) because the beams mixed with lower cement contents (3 and 5%) were too weak to sustain their own weight, even after an extended curing period.

TEST RESULTS AND ANALYSIS

This section reports the main test results obtained for the laboratory specimens and field cores. Focusing on the relationships between E and UCS and MoR and UCS, simple regression analyses were performed and the results compared with those obtained using existing model predictions for E and MoR. Table 7 presents the average and coefficient of variance (CoV) for UCS, MoR, and E for each set of samples; the raw data is provided in Appendix B.

Table 7. Summary of test results for the lab specimens

Sample	Cement content (%)	7-day UCS, (Standard)		7-day UCS, (PM)		7 or 28-day MoR [1]		7-day Elastic Modulus (E)	
		Mean (psi)	*CoV (%)*	*Mean (psi)*	*CoV (%)*	*Mean (psi)*	*CoV (%)*	*Mean (psi)*	*CoV (%)*
Winder	3	70	2.9	41	21.2	-	-	633	28.0
	5	129	2.8	113	5.4	-	-	2,499	51.8
	7	279	5.6	234	8.2	69	7.6	3,117	18.9
	9	433	5.3	417	2.0	101	2.6	3,090	38.1
PTC1	3	142	29.7	326	24.4	-	-	2,771	14.6
	5	245	4.13	346	13.1	-	-	1,993	25.0
	7	409	3.7	477	9.8	81	15.0	2,928	43.4
	9	539	1.1	791	10.3	104	12.3	3,046	15.8
PTC2	3	128	4.0	121	3.5	-	-	1,241	28.4
	5	261	15.1	265	1.9	-	-	2,808	39.3
	7	448	1.4	429	5.1	136	10.6	4,320	27.1
	9	682	5.6	678	7.8	176	4.9	7,120	22.3

[1] 28-day MoR is measured for the PTC2 samples only

Here, CoV is defined as the percentage ratio of the average obtained to the standard deviation of the data, an indicator of the degree of data reproducibility. As a higher CoV signifies poor data repeatability, any data with a CoV of more than 30% - a statistical threshold – would usually receive little credence [17]. The results indicate that a cement content of at least 7% is needed to achieve the minimum UCS requirement (300 psi at 7-day). Comparisons between the results for the two different types of cylinders reveal no significant size effect of the type typically observed in cement mortar or concrete samples. Interestingly, the values of E are much lower than those normally found for soft layer materials. This may be caused by the lack of compatibility of the RAP within the composite, which negatively affects the compaction and, possibly, the curing process. Further, some RAP nuggets crumble easily under loading, making specimens less stiff. The boundary conditions of the cylinder could be another factor contributing to the small E values. However further testing would be needed to confirm this speculation.

These results indicate that all the properties benefit from a higher cement content. Within the range tested, UCS increased by an average of 92 psi/%, 150 psi/%, and 155 psi/% for Winder, PTC1, and PTC2, respectively. Regarding the size effect, the standard cylinders yielded a higher UCS for Winder and PTC2 than for the PM specimens, but this pattern was reversed for PTC. This lack of a size effect on UCS suggests there may be no significant compaction constraints imposed by utilizing different mold materials (in this case, steel versus polyethylene). The 7- and 28-day MoR data show that the curing period does indeed play a role in the development of the flexural strength of CSRB, however. Although the positive effect of cement content on E is confirmed by the results obtained for UCS and MoR, some data sets were not repeatable (those with a CoV of over 30%), possibly due to sample-to-sample variability combined with the inherent sensitivity of the loading jig.

Table 8 presents the UCS and E results for the field cores. The intrinsic randomness of in-situ CSRB materials is clearly captured by their generally higher CoVs. Despite the expected gap in UCS values between laboratory specimens and field cores, the difference in the UCS gains between the two groups (laboratory versus field) was not especially significant. Although identifying the underlying cause of this disparity is beyond the scope of this study, it is fair to assume that as both cement hydration and curing make CSRB stiffer over time, this will amplify the performance of FDR pavement. The raw data is presented in Appendix B.

Table 8. Summary of test results from field cores

Road name	Age (years)	No of sample	UCS		Elastic Modulus (E)	
			Mean (psi)	*CoV (%)*	*Mean (psi)*	*CoV (%)*
SR-70	+3	6	687	33.5	178,110	36.5
Metcalf	+5	6	643	20.7	28,611	51.8
Senoia	+5	5	381	46.9	214,461	51.3
Smiley Cross	+15	4	186	16.5	-	-

The E-UCS Relationship

This section examines how the relationship between E and UCS evolves for PM specimens and field cores and considers how the trends relate to the E prediction models recommended for other chemically stabilized pavement materials. Figure 4 shows the relationships between the experimentally determined E and UCS values recorded for the PM specimens. A simple linear regression function was found to be sufficient to define each correlation, with all the R^2 values being over 0.80. PTC2 achieved the highest correlation for the 7-day specimens.

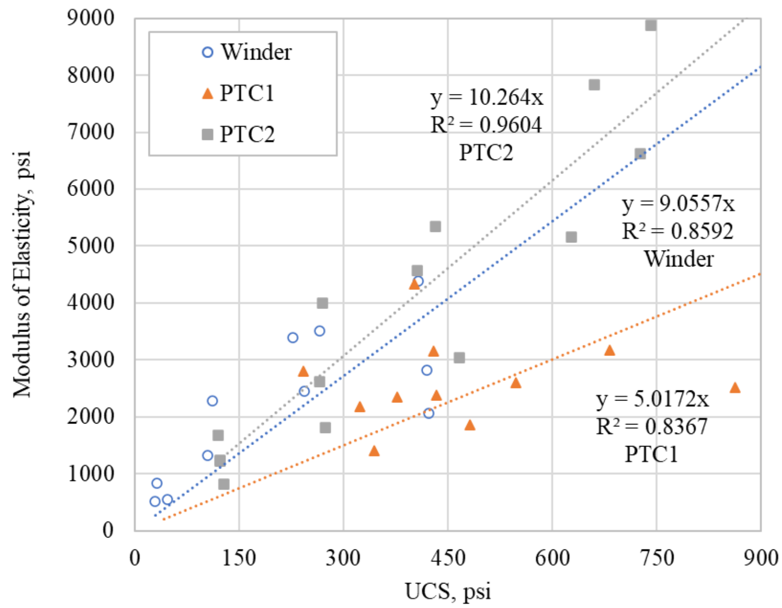


Figure 4. Graph. E versus UCS relationships for three different FDR sections.

Analyzing the same data with similar linear functions for the various cement contents (Figure 5) revealed that despite a reasonably good correlation for each individual cement content, the cement content had no apparent effect on the E-UCS relationships for 7-day old specimens. In fact, this finding agrees with those reported in a recent study examining the relationships between UCS and a range of other mechanical properties [18]. Figure 6 shows the E-UCS relationships developed for field cores aged between 3 and 5 years. Note that the samples from Smiley Cross Road are not included due to the lack of E data. There is no clear influence of the service life (or an extended curing period) on the growth of E, which may instead arise due to differences in the design and/or construction conditions.

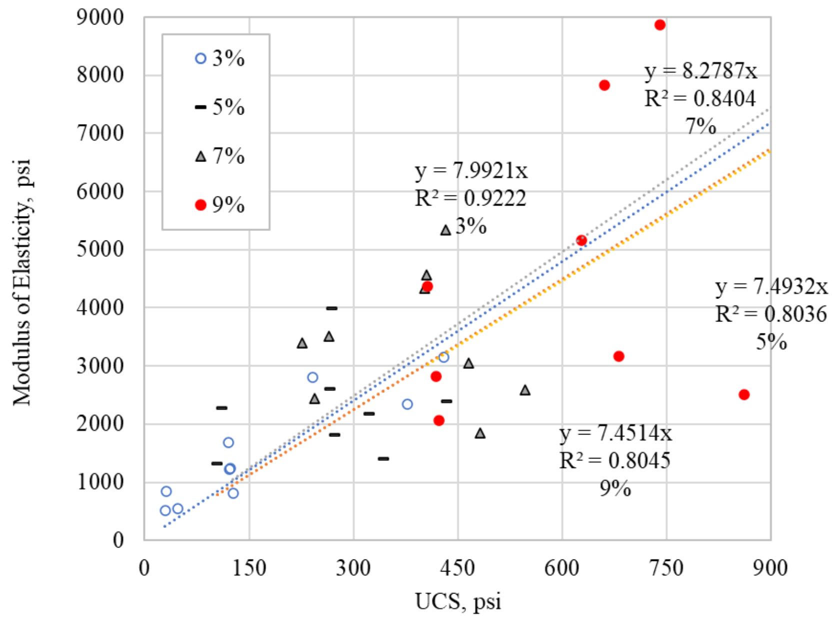


Figure 5. Graph. E versus UCS relationships for four cement contents.

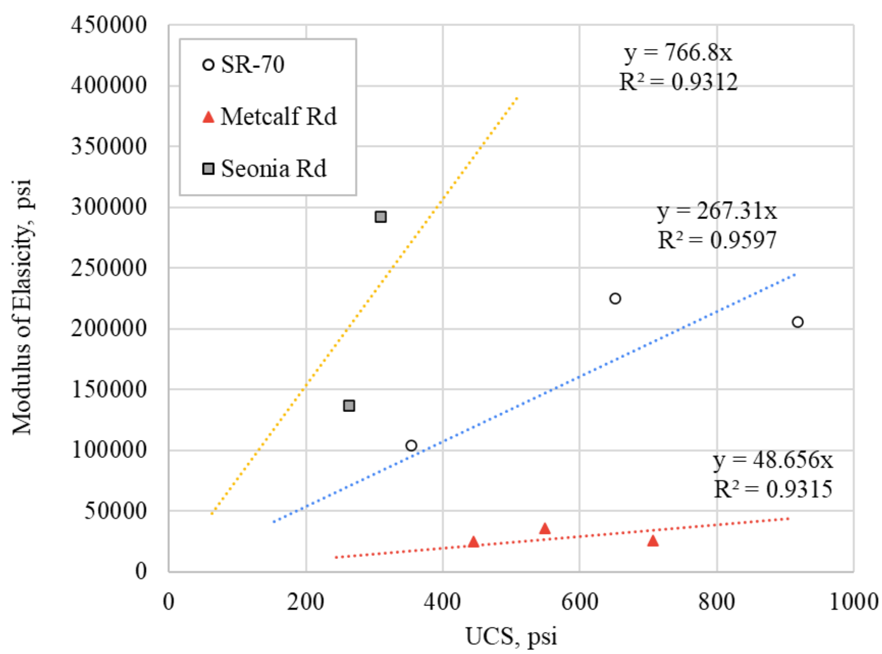
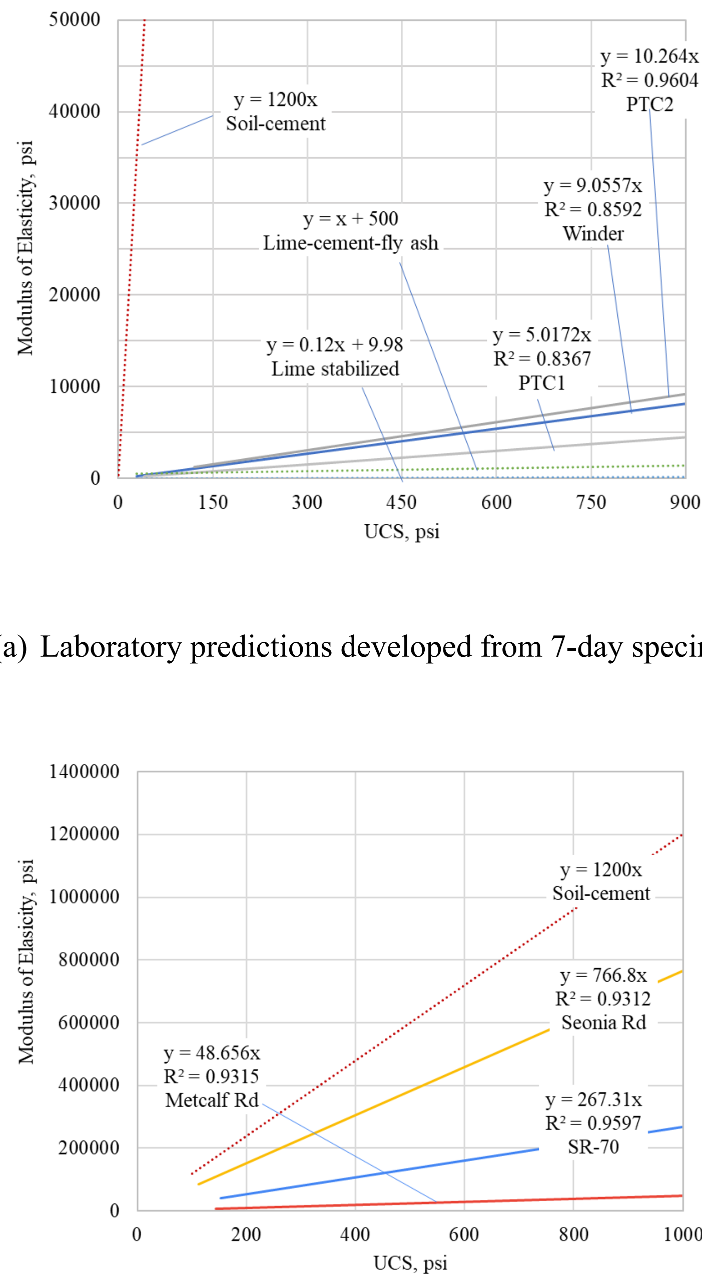


Figure 6. Graph. E versus UCS relationships developed for field cores.

As demonstrated by the data presented in Tables 7 and 8, the level of E observed in the field samples is much higher than that observed in the laboratory specimens. Differences in the curing periods, compaction methods, and boundary conditions, among other factors, may be responsible for this variance in E.

To date, several UCS-based E prediction models (E models) have been recommended and used for a variety of chemically stabilized pavement materials. These models offer reasonable pavement design inputs (Levels 2 and 3). In the absence of reliable E models for CSRB materials, however, pavement designers often find it challenging to choose an appropriate alternative E model for CSRB. It is not the intent of this work to develop E models for CSRB. Modeling requires a comprehensive test regime that extends across a wide spectrum of material variables and includes a rigorous statistical analysis. Instead, this study focusses on the possibility of positioning an E model for CSRB within the existing models. We therefore compared the regression functions for CSRB materials with three common substitute E models, namely those for soil-cement, lime stabilized, and lime-cement-fly ash, at different curing times.

Figure 7a compares the proposed regression functions with the predictions of the existing models for laboratory samples that have undergone a seven-day curing period. In the figure, the proposed functions are the same as those shown in Figure 4, with the data points omitted for clarity. The results suggest that although the soil-cement model should not be suitable for CSRB materials, the lime-cement-fly ash and lime stabilized models deliver results that are comparable to those for the proposed models. However, when the field core models are examined (Figure 7b), the proposed functions become closer to the soil-cement model after CSRB materials have been cured for an extended period, probably beyond 28 days.



(a) Laboratory predictions developed from 7-day specimens

(b) Field core predictions and soil cement model

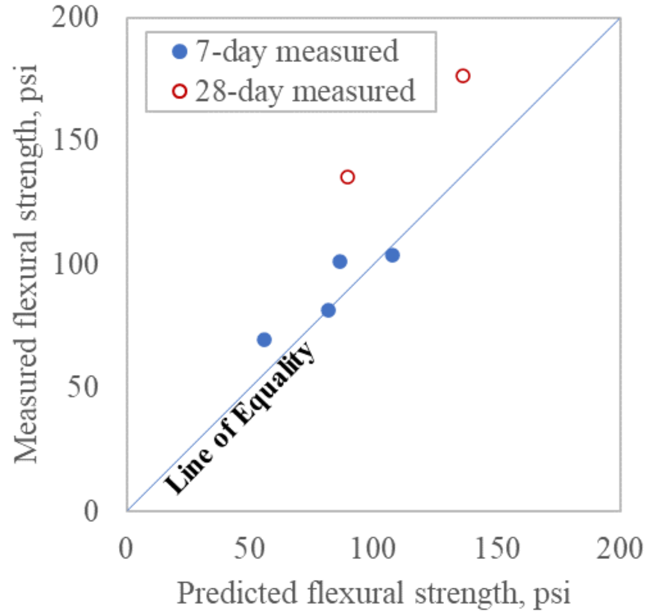
Figure 7. Graphs. Comparison of E prediction models for different sample types.

This observation may indicate the dominant effect of curing age on E models for CSRB materials, although once again, more study is required to confirm the validity of the proposed regression functions. These results suggest that none of the existing models are appropriate when it comes to modulus prediction for CSRB layers. A separate laboratory test on an appropriate set of specimens is therefore required to determine the layer moduli for pavement design.

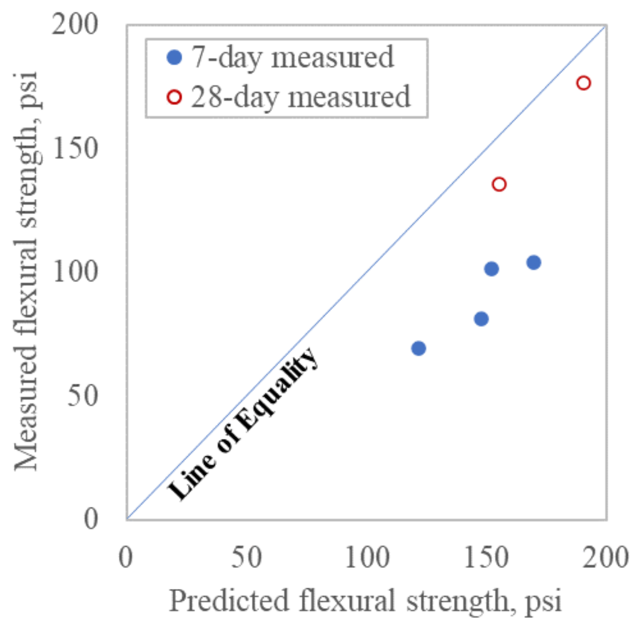
The MoR-UCS Relationships

The Modulus of Rupture (MoR) is one of the key input parameters for the Level 2 fatigue cracking predictions in the AASHTO MOP. For conventional cement-treated layer materials, the MoR, which is generally measured at 28-days, can be obtained from the MoR-UCS relationship models proposed by either AASHTO or the Portland Cement Association (PCA). Note that both models predict the 28-day MoR based on the 7-Day UCS of cement-treated aggregate and have not been validated for CSRB materials.

Figure 8 compares the actual results measured for the CSRB materials in this study with their predicted MoR values. Each data point represents the average MoR for three replicate standard specimens and two curing periods (7 and 28 days). The predictions from the AASHTO model (MoR = 0.2UCS) appear to be closer to the actual 7-day MoR than those for the 28-day MoR, although the PCA model (MoR = 7.30 $(\mathrm{UCS})^{0.5}$) delivers slightly better predictions for the 28-day MoR than the 7-day MoR. Overall, we found that the AASHTO model underestimates the 28-day MoR, while the PCA model overestimates it slightly for the CSRB materials tested here.



(a) AASHTO model



(b) PCA model

Figure 8. Graphs. MoR values predicted by the AASHTO and PCA models.

CHAPTER SUMMARY

A series of laboratory tests was conducted on laboratory specimens and field cores made with CSRB materials. The test results improve our understanding of the main material properties, while at the same time raising questions that need to be answered. To summarize our findings:

- Comparing the standard and PM specimens, the effect of size on UCS is insignificant, making the use of PM appropriate for properly graded CSRB materials.
- The cement content plays a crucial role in achieving the target strength, as evidenced by the increase in UCS with cement content observed in the laboratory specimens. This is also the case for the development of stiffness in CSRB, even after the shortest curing times.
- As CSRB materials age, E increases much faster than the UCS rises, offering strong support for the resistance to vertical deflection of surfaces and the development of wearing courses in FDR pavement.
- The E-UCS relationships proposed here and the comparison of their performance with those of the existing E prediction models reveal that the soil-cement model is inadequate as an alternative model for CSRB materials. Thus, it is recommended that an independent laboratory test be run on a set of representative specimens to determine the actual layer moduli for pavement design.
- The PCA model slightly overestimates the MoR of CSRB materials.

CHAPTER 3. FIELD PERFORMANCE EVALUATION AND PMED ANALYSIS

Falling Weight Deflectometer (FWD) tests were conducted on site at each test location to characterize the CSRB layer. The four test sites, which were located on SR70 (Franklin County), Metcalf Road (Thomas County), Senoia Road (Fayette County) and Smiley Cross Road (Long County), all met the GDOT laboratory strength requirement of 450 psi at 7-days. Fast Falling Weight Deflectometer data was collected at 100-ft intervals and at three load levels at each site. The deflection data were then analyzed in accordance with the guidelines provided in the *AASHTO 1993 Guide for Design of Pavement Structures* to estimate the subgrade resilient modulus, effective structural number value, and structural layer coefficient of the CSRB layer at each location.

The Impulse Stiffness Modulus (ISM) is a normalization of the applied load by the resulting load plate deflection. Figure 9 shows the average ISM values at the study locations. The SR70 data from a previous study is also included to show the effect of the curing period on the ISM. Note that a 4-inch asphalt concrete surface was installed two weeks after the FDR process was completed, hence the ISM value of the SR 70 section at 3 years includes the stiffness from the asphalt surface. In general, the stiffness modulus increases with time. However, the stiffness moduli of the pavement sections are generally lower than that for SR70 in the two pavement sections that are more than 5 years old (Metcalf and Senoia Roads). The very low ISM value for the Smiley Road pavement section could indicate a significant stiffness loss in its CSRB base after more than 15 years of service. It is important to bear in mind that historical data such as the CSRB mix design, amount of traffic load, and maintenance/rehabilitation activities are only intermittently available over the long periods of time these older sections have been in service.

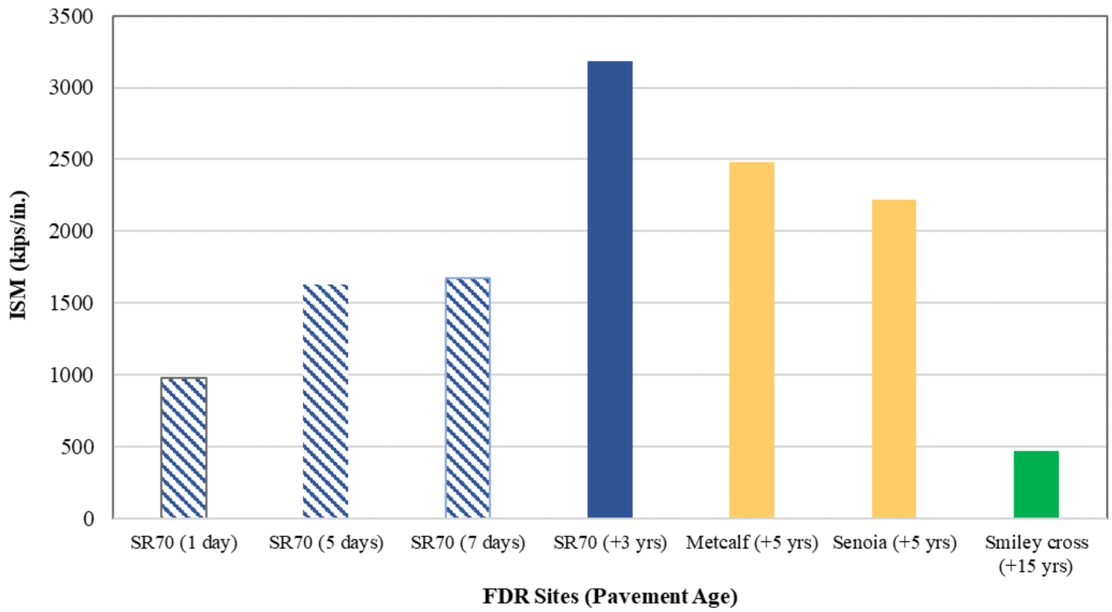


Figure 9. Chart. Average Impulse Stiffness Modulus.

STRUCTURAL LAYER COEFFICIENT FOR CSRB

The effective structural numbers for the FDR sections were calculated following the procedure described in the AASHTO 1993 Design Guide using the values for the center plate deflection and the total pavement thickness obtained from the cores [4]. The effective structural numbers and design subgrade resilient modulus were then used to calculate the structural layer coefficient for the CSRB layer. The pavement sections are modeled as an idealized two-layer structure composed of HMA and a CSRB layer. GDOT's current pavement design manual uses structural layer coefficients for HMA that range from 0.3 (old) to 0.44 (new). An HMA layer coefficient of 0.42 was used here for the SR70 section calculations as the pavement section is relatively new; an HMA layer coefficient of 0.3 was used for the other sections. The drainage coefficient is assumed to be 1.

The effective structural number was calculated using the equations shown in Figure 10:

$$SN_{eff} = 0.0045\ D\ \sqrt[3]{E_p}$$

where, D = Total thickness of all pavement layers above the subgrade, in inches

Ep = Effective modulus of pavement layers above the subgrade, in psi

$$d_o = 1.5\ p\ a \left\{ \frac{1}{M_R \sqrt{1 + \left(\frac{D}{a} \sqrt[3]{\frac{E_P}{M_R}} \right)^2}} + \frac{\left[1 - \frac{1}{\sqrt{1 + \left(\frac{D}{a} \right)^2}} \right]}{E_P} \right\}$$

where, do = Deflection measured at the center of the load plate, inches

p = Nondestructive testing (NDT) load plate pressure, in psi

a = NDT load plate radius, in inches

M_R = Subgrade resilient modulus, in psi

And

$$M_R = \frac{0.24\ P}{d_r\ r}$$

where, P = Applied load, in pounds

d_r = Deflection at a distance r from the center of the load, in inches

r = Distance from center of load, in inches

Figure 10. Equations. Effective structural number.

The effective structural numbers from the calculations are presented in Table 9. The largest effective structural numbers (6.91 – 9.29) were found for the SR70 section. The Smiley Road had the smallest effective structural numbers (4.51 – 6.04), indicating a significant loss in the pavement stiffness after 15 years of service. It is beyond the scope of this study to identify the main causes for the stiffness degradation, but it appears that as pavement sections age, the pavement stiffness decreases. The effective structural numbers were then used to determine the layer coefficients for the CSRB. Figure 11 shows the average layer coefficients of all the CSRB sections, ranging from 0.42 (SR 70) to 0.25 (the Smiley Road).

Based on the correlation chart shown in Figure 1, the layer coefficients can be calculated from the UCS values of the field core specimens and from the backcalculated modulus obtained from the FWD deflection measurement data. The results are shown in Table 10 and Figure 11. The deflection data were analyzed using the AASHTOWare Backcalculation Tool (BcT) software package. The structural layer coefficients decrease as pavement age increases, indicating a reduction in CSRB strength over time.

Significant differences were observed between the layer coefficients determined from the effective structural numbers and those from the UCS values. The layer coefficients based on the effective structural numbers are nearly double those calculated from the field core UCS values. In general, the layer coefficients from the UCS values are in better agreement with the structural layer coefficient suggested for the existing chemically treated base by AASHTO [6].

Table 9. Calculated Layer Coefficient for CSRB – Effective SN

	Station	FWD load		FWD deflection		Layer thickness (in.)			Calculated Subgrade M_R (psi)	E_P (ksi)	SN-eff	CSRB - a_i
		p (psi)	P (lb)	d_0 (mils)	d_{36} (mils)	HMA	CSRB	Total D				
SR70	SB-204	86.6	9,503	2.32	1.52	5.00	15	20.00	41,678	1,100	9.29	**0.48**
	SB-2000	84.4	9,261	3.93	2.69	6.00	11.5	17.50	22,952	830	7.40	**0.42**
	SB-4011	83.8	9,195	2	0.95	5.75	10.5	16.25	64,529	1,150	7.66	**0.50**
	NB-208	84.8	9,305	3.31	2.15	5.25	13.5	18.75	28,853	820	7.90	**0.42**
	NB-2010	84.6	9,283	3.59	2.69	5	14.75	19.75	23,007	870	8.48	**0.43**
	NB-3997	84.3	9,250	2.3	1.41	6.25	13.5	19.75	43,736	470	6.91	**0.32**
Metcalf	SB-208	86.8	9,525	4.13	1.56	3	10	13.00	40,703	620	4.99	**0.41**
	SB-2010	85.6	9,393	4.67	2.33	3.25	10	13.25	26,875	770	5.47	**0.45**
	SB-3997	85	9,327	4.65	2.26	3.75	9	12.75	27,513	790	5.30	**0.46**
	NB-206	84.7	9,294	3.4	1.87	3.75	10.5	14.25	33,134	1050	6.52	**0.51**
	NB-2013	84	9,217	3.56	1.71	3.5	10.25	13.75	35,935	700	5.49	**0.43**
	NB-4009	83.9	9,206	2.77	1.17	3.75	12	15.75	52,458	400	5.22	**0.34**
Senoia	SB-200	85	9,327	5.95	2.29	3.5	10	13.50	27,153	410	4.51	**0.35**
	SB-4003	85.1	9,338	5.55	2.06	4.5	11.75	16.25	30,220	330	5.05	**0.32**
	NB-202	84.7	9,294	4.3	2.2	4.5	11	15.50	28,164	650	6.04	**0.43**
	NB-2008	84.7	9,294	4.37	1.76	4.25	10	14.25	35,205	550	5.25	**0.40**
	NB-4003	84.4	9,261	3.52	1.39	3.5	13	16.50	44,418	540	6.05	**0.38**
Smiley	SB-0	81.4	8,932	8.28	2.71	1.75	8.25	10.00	21,973	380	3.26	**0.33**
	SB-2692	81.1	8,899	11.6	2.54	1.5	8.5	10.00	23,357	150	2.39	**0.23**
	NB-394	82	8,998	8.43	2.28	1.5	7.5	9.00	26,309	340	2.83	**0.32**
	NB-3301	80.9	8,877	16.08	1.81	1.5	8.5	10.00	32,697	56	1.72	**0.12**

A sample calculation for calculating layer coefficients of CSRB is shown in Appendix C.

Table 10. Calculated Layer Coefficient for CSRB – UCS of field core specimens and FWD back calculation.

Site Age MAAT*	Station	Existing thickness (in.)		UCS Field core (psi)	Layer coefficient from UCS [1]	Back calculated CSRB modulus (psi) [2]	Layer coefficient from back calculated modulus [1]
		AC	BASE				
SR70 +3 yrs. 61.3 °F	SB 204	5	15	816	**0.22**	792,260	**0.22**
	SB 2000	6	11.5	488	**0.17**	787,860	**0.21**
	SB 4011	5.75	10.5	895	**0.24**	802,500	**0.22**
	NB 208	5.25	13.5	353	**0.15**	710,400	**0.19**
	NB 2010	5	14.75	651	**0.20**	799,450	**0.22**
	NB 3997	6.25	13.5	918	**0.24**	642,800	**0.17**
Metcalf +5 yrs. 66.6 °F	SB 213	3	10	620	**0.20**	334,050	**0.03**
	SB 2007	3.25	10	812	**0.22**	345,450	**0.04**
	SB 3998	3.75	9	729	**0.21**	359,610	**0.04**
	NB 206	3.75	10.5	549	**0.18**	542,910	**0.13**
	NB 2013	3.5	10.25	706	**0.21**	361,460	**0.05**
	NB 4009	3.75	12	445	**0.17**	489,250	**0.11**
Senoia +5 yrs. 61.4 °F	SB 200	3.5	10	406	**0.16**	85,380	**0.00**
	SB 4003	4.5	11.75	245	**0.13**	91,750	**0.00**
	NB 202	4.5	11	310	**0.14**	330,530	**0.03**
	NB 2008	4.25	10	680	**0.21**	431,100	**0.08**
	NB 4003	3.5	13	262	**0.13**	255,580	**0.00**
Smiley Cross +15 yrs. 67.4 °F	SB 0	1.75	8.25	158	**0.11**	107,510	**0.00**
	SB 2692	1.5	8.5	180	**0.11**	31,150	**0.00**
	NB 394	1.5	7.5	229	**0.12**	132,560	**0.00**
	NB 3301	1.5	8.5	175	**0.11**	11,350	**0.00**

[1] Based on the correlations shown in Figure 1

[2] The backcalculated modulus is a direct output of the AASHTOWare Backcalculation Tool (BcT)

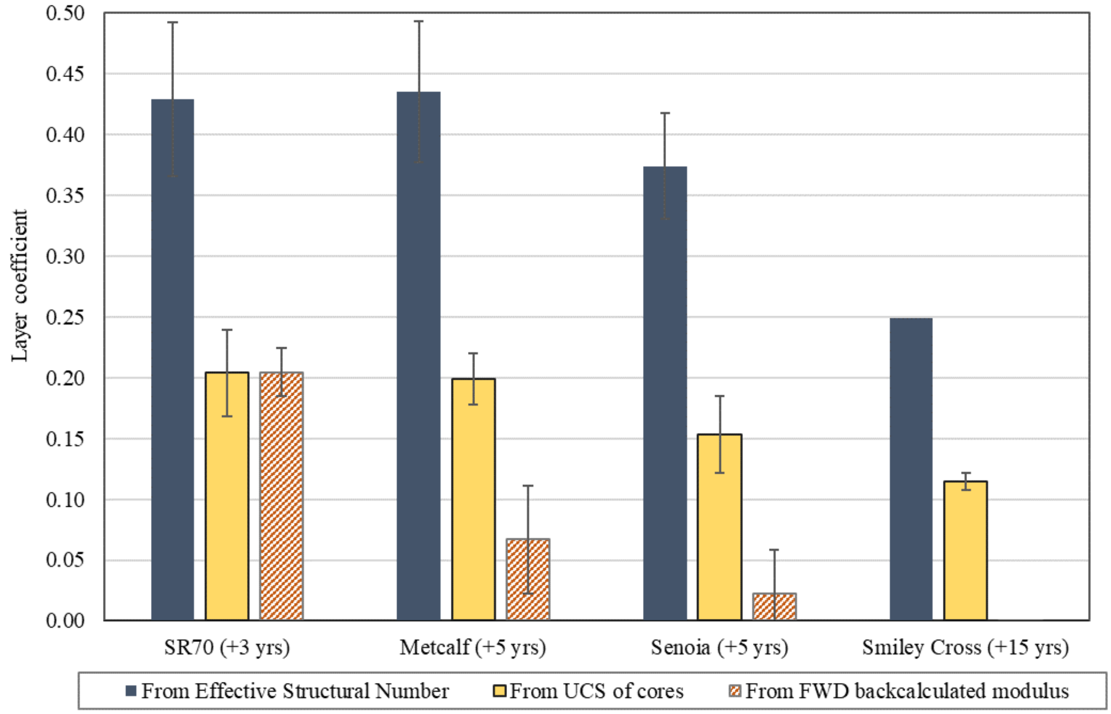


Figure 11. Chart. Calculated structural layer coefficients for CSRB.

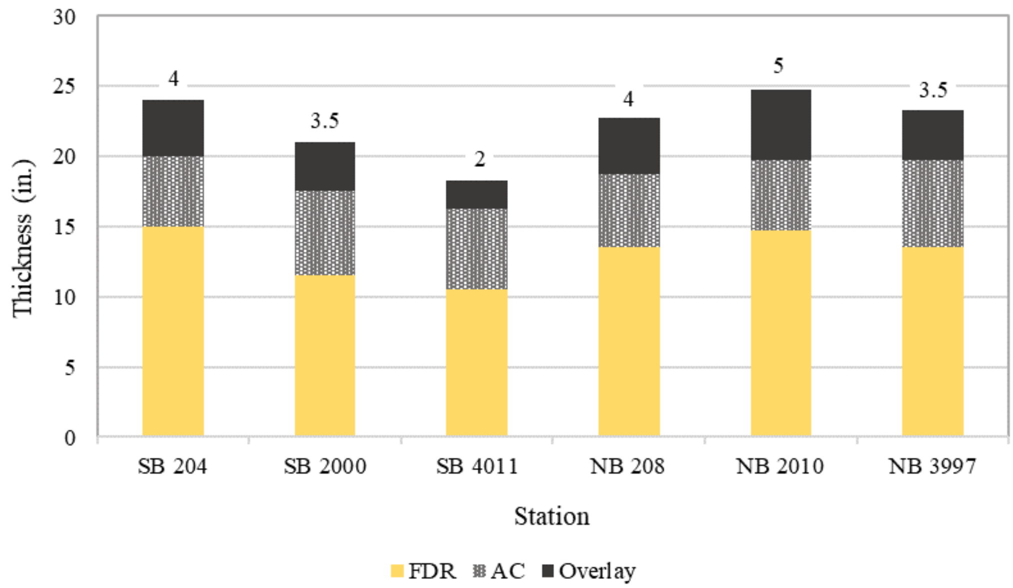
FDR IN AASHTOWARE PAVEMENT MDE DESIGN (PMED)

Overlay Designs

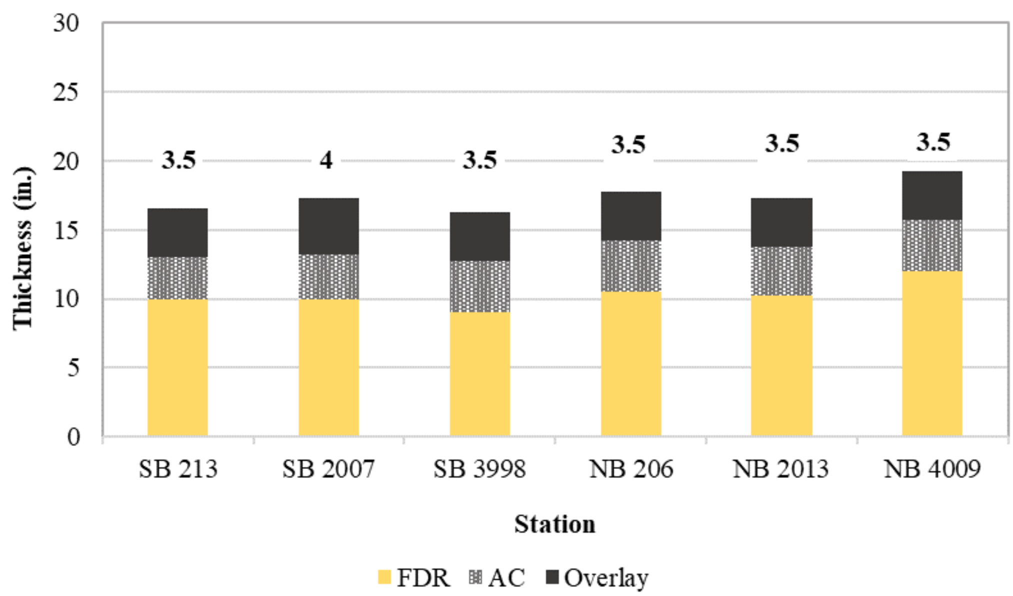
The overlay designs were developed utilizing the AASHTOWare Pavement ME Design (PMED) software, version 2.60, based on the current pavement condition estimated using the FWD test results. Although a flexible pavement section with CSRB is considered to be a semi-rigid pavement, as yet there is no semi-rigid pavement simulation available in PMED as no reliable calibration coefficients for semi-rigid pavements have yet been derived. At present, the recommendation approach is to model a semi-rigid pavement as a conventional flexible pavement with a high stiffness Graded Aggregate Base (GAB) layer [5,6]. The two-way average annual daily truck traffic (AADTT) for local roads is assumed to be equal to 3,000 trucks, which takes into account a predicted increase in future traffic volumes.

The performance limits used in the PMED analyses were based on the GDOT Pavement ME Design User Input Guide. Top-down cracking has not been included among the performance indicators as it is not recommended that top-down cracking should be included in Georgia pavement design until the transfer function has been calibrated [6, 19]. Correction factors of 0.6 and 0.5 were used for the NDT modulus of conversion for the non-stabilized bases and subgrades, respectively. The required overlay thicknesses are compared in Figure 12.

The existing pavement sections at the Metcalf Road and Senoia Road sites are broadly comparable, although the average AC and base thickness of the pavement sections at the Senoia Road site is slightly thicker than those for the Metcalf Road sections. The difference in the predicted overlay thicknesses between these two sections is mainly due to differences in their local climates: the Mean Annual Average Temperature (MAAT) at the Senoia Road site is 61.4 ^{o}F, while that at the Metcalf Road site is 66.6 ^{o}F. To verify the sensitivity of the climate to predicted distresses, a further analysis was performed for a variety of different overlay thicknesses and the results are provided in Table 11. Metcalf north bound station 2013 and Senoia south bound station 200 were selected for this comparison; their pavement structures and material properties were kept the same for the purposes of this comparison. The optimized overlay thicknesses for the Senoia Road site show the sensitivity of the NDT modulus of the existing AC to the predicted AC reflective cracking and AC permanent deformation. The existing AC modulus has no effect on any of the other predicted distresses.

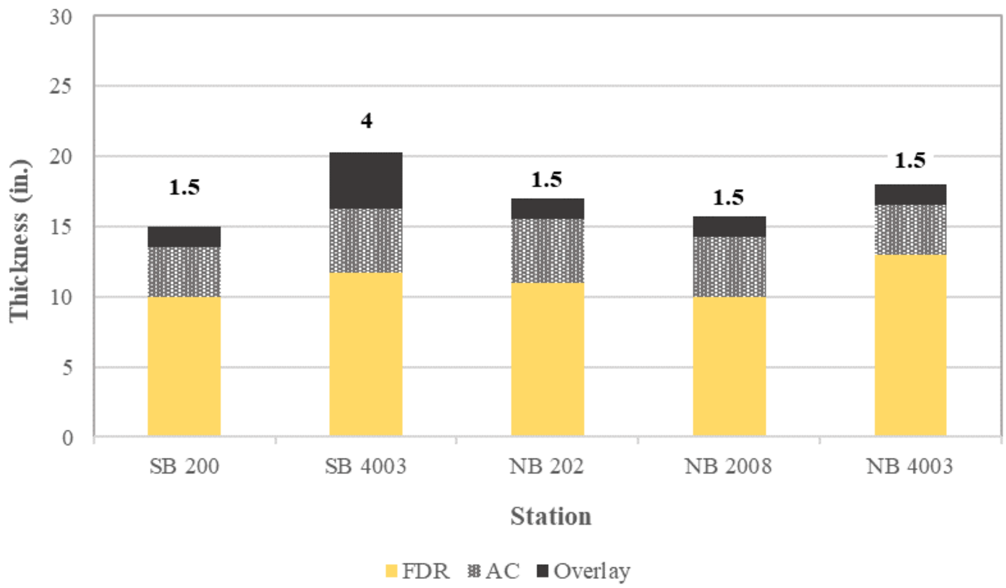


(a) SR 70

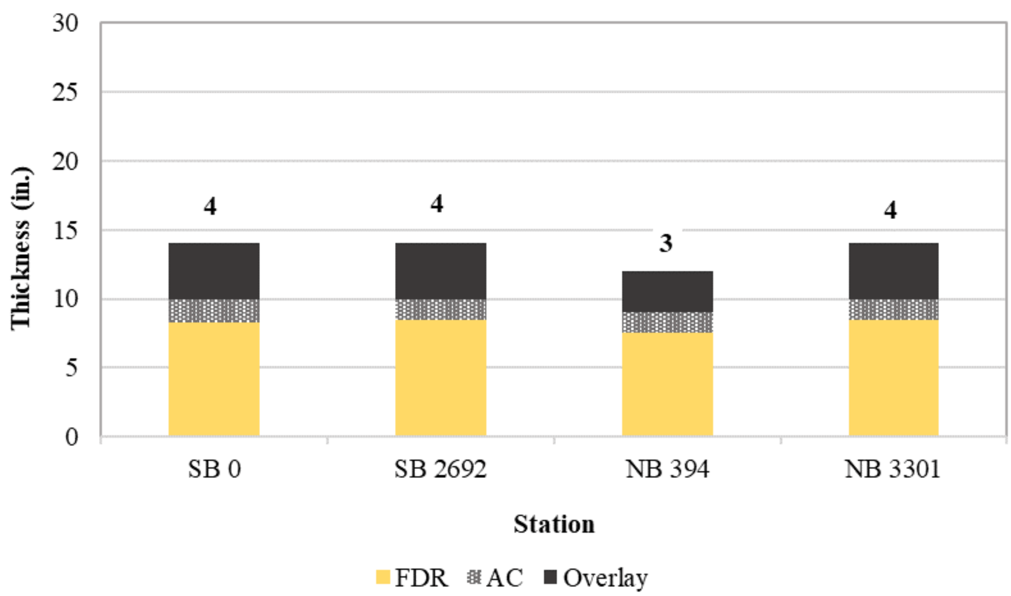


(b) Metcalf Road

Figure 12. Charts. Required overlay thickness.



(c) Senoia Road



(d) Smiley Cross Road

Figure 12. Charts. Required overlay thickness (Continued).

Table 11. Distress prediction summary for different climate zones

Distress Type	Target Distress @ Specified Reliability	Target Reliability (%)	W/ 1.5 in overlay		W/ 3.5 in overlay	
			MAAT 61.4 °F	MAAT 66.6 °F	MAAT 61.4 °F	MAAT 66.6 °F
Terminal IRI (in/mile)	220	75	157.78	163.32	151.97	153.21
Permanent deformation - total pavement (in)	0.4	75	0.15	0.14	0.13	0.12
AC total fatigue cracking: bottom up + reflective (% lane area)	25	50	23.60	27.13	22.61	25.00
AC total transverse cracking: thermal + reflective (ft/mile)	2500	50	698.00	1532.24	214.35	624.08
Permanent deformation - AC only (in)	0.4	75	0.04	0.04	0.03	0.03
AC thermal cracking (ft/mile)	1500	50	623.04	1457.28	139.39	549.12

Note that the overlay thicknesses required for the SR 70 site were necessarily optimized to deliver higher target reliability and cope with greater traffic volumes than those required for the local roads. The predicted overlay thicknesses for SR 70 road sections are thus generally thicker than those for the Senoia Road sections, even though the two project sites are located in the same climate zone. The predicted overlay thicknesses become similar after adjusting for the reliability targets and layer thicknesses.

The Smiley Cross Road has the oldest pavement among the 4 cases. The existing layer thickness measured in the core samples for this location indicate that the pavement section at this site has relatively thin asphalt concrete and CSRB layers, hence a thicker overlay than that for the other sites is predicted.

Sensitivity Analysis

A sensitivity analysis was performed to identify the input variables that exert the most influence on the performance, as predicted by the PMED. Two different pavement types were used to model the FDR pavement: flexible pavement and semi-rigid pavement. Although the semi-rigid pavement simulation is not directly applicable in PMED, a sensitivity analysis was performed to investigate the influence of this input parameter.

Analysis 1. Stabilized base modeled as high stiffness GAB

A conventional flexible pavement with a 4-inch asphalt concrete over a 10-inch CSRB was used for the sensitivity analysis; the CSRB thickness in Georgia is typically 8 to 12 inches. The non-stabilized base modulus range was selected based on the typical mean resilient modulus of the granular aggregate base materials used in Georgia and the 28-day elastic modulus of the cement aggregate base mixture [6, 19]. The sensitivity analysis inputs are summarized in Table 12.

Tables 13 and 14 provide summaries of the predicted distress values for conventional flexible pavement that satisfy the target performance criteria. In general, although the predicted distresses decrease with increasing base modulus and thickness, these are relatively insensitive to variations in the modulus and layer thickness unless AC bottom-up fatigue cracking is present, at which point the predictions become highly sensitive to the base moduli. These results indicate that to limit bottom-up cracking to below 2 % per lane area, the CSRB moduli should be greater than 60,000 psi. As the modulus of the cement aggregate base is typically greater than 60,000 psi, bottom-up cracking is not expected to be an issue for a pavement with a CSRB. However, the cement aggregate base may develop shrinkage cracks, and these could be reflected in the asphalt surface. The reflective cracking potential in such cases will be explored in detail in Analysis 3.

Table 12. PMED inputs for sensitivity analysis 1

Pavement structure: 4.0 in. Asphalt Concrete 10.0 in. Non-stabilized Base (A-1-a) Subgrade resilient modulus: 8000 psi
Traffic (Two-way AADTT): State Route: 4,000 Local road: 3,000
Climate: Fulton, GA
Sensitivity variables: Asphalt concrete thickness (in.): 2 to 6 in. Non-stabilized base thickness (in.): 8 to 12 in. Non-stabilized base modulus (psi): 25,000 psi to 100,000 psi.

Table 13. Sensitivity analysis summary (Flexible pavement - State Route)

Sensitivity - AC thickness

Distress type	Target distress	Reliability (%)	Predicted distress		
			AC thickness (in.)		
			2 in.	4 in.	6 in.
Terminal IRI (in/mile)	220	90	141.14	138.34	136.66
Permanent deformation - total pavement (in)	0.4	90	0.19	0.17	0.15
AC bottom-up fatigue cracking (% lane area)	25	90	1.45	1.45	1.45
AC thermal cracking (ft/mile)	1500	50	166.85	97.36	39.49
Permanent deformation - AC only (in)	0.4	90	0.02	0.03	0.03

Sensitivity - Base thickness

Distress type	Target distress	Reliability (%)	Predicted distress		
			Base thickness (in.)		
			8 in.	10 in.	12 in.
Terminal IRI (in/mile)	220	90	139.09	138.34	137.81
Permanent deformation - total pavement (in)	0.4	90	0.18	0.17	0.16
AC bottom-up fatigue cracking (% lane area)	25	90	1.45	1.45	1.45
AC thermal cracking (ft/mile)	1500	50	93.03	97.36	103.7
Permanent deformation - AC only (in)	0.4	90	0.03	0.03	0.03

Sensitivity - Base modulus

Distress type	Target distress	Reliability (%)	Predicted distress		
			Base modulus (psi)		
			25,000	62,500	100,000
Terminal IRI (in/mile)	220	90	142.74	139.2	138.34
Permanent deformation - total pavement (in)	0.4	90	0.23	0.19	0.17
AC bottom-up fatigue cracking (% lane area)	25	90	18.17	1.53	1.45
AC thermal cracking (ft/mile)	1500	50	97.36	97.36	97.36
Permanent deformation - AC only (in)	0.4	90	0.04	0.03	0.03

Table 14. Sensitivity analysis summary (Flexible pavement - local road)

Sensitivity - AC thickness

Distress type	Target distress	Reliability (%)	Predicted distress AC thickness (in.) 2 in.	4 in.	6 in.
Terminal IRI (in/mile)	220	75	146.05	143.67	142.38
Permanent deformation - total pavement (in)	0.4	75	0.16	0.14	0.13
AC bottom-up fatigue cracking (% lane area)	25	75	0.76	0.76	0.76
AC thermal cracking (ft/mile)	1500	50	149.95	77.93	29.57
Permanent deformation - AC only (in)	0.4	75	0.02	0.02	0.02

Sensitivity - Base thickness

Distress type	Target distress	Reliability (%)	Predicted distress Base thickness (in.) 8 in.	10 in.	12 in.
Terminal IRI (in/mile)	220	75	144.32	143.67	143.23
Permanent deformation - total pavement (in)	0.4	75	0.15	0.14	0.14
AC bottom-up fatigue cracking (% lane area)	25	75	0.76	0.76	0.76
AC thermal cracking (ft/mile)	1500	50	80.89	77.93	80.36
Permanent deformation - AC only (in)	0.4	75	0.02	0.02	0.02

Sensitivity - Base modulus

Distress type	Target distress	Reliability (%)	Predicted distress Base modulus (psi) 25,000	62,500	100,000
Terminal IRI (in/mile)	220	75	147.09	144.38	143.67
Permanent deformation - total pavement (in)	0.4	75	0.19	0.16	0.14
AC bottom-up fatigue cracking (% lane area)	25	75	9.44	0.77	0.76
AC thermal cracking (ft/mile)	1500	50	77.93	77.93	77.93
Permanent deformation - AC only (in)	0.4	75	0.03	0.02	0.02

Analysis 2. Interface debonding between AC and CSRB

De-bonding at the interface between the AC and the CSRB were observed at a few locations during the field coring operation at the Metcalf Road site. As a result, the effect of interface friction on the predicted performance of a pavement section was also investigated using PMED. The design inputs and target performance for the State Route section were used in this analysis. The results indicate that the predicted distresses are insensitive to the interface bonding condition (see Table 15).

Table 15. Sensitivity analysis 2 - interface bonding

Distress type	Target distress	Reliability (%)	Predicted distress	
			Layer interface friction	
			Full slip (0)	Full bonding (1)
Terminal IRI (in/mile)	220	75	145.7	143.67
Permanent deformation - total pavement (in)	0.4	75	0.19	0.14
AC bottom-up fatigue cracking (% lane area)	25	75	9.18	0.76
AC thermal cracking (ft/mile)	1500	50	77.93	77.93
Permanent deformation - AC only (in)	0.4	75	0.04	0.02

Analysis 3. Stabilized base modeled as Chemically Stabilized Layer (Soil-cement)

The value ranges utilized for the stabilized base modulus and modulus of rupture are based on the minimum and maximum values required in the PMED software. The sensitivity analysis inputs are listed in Table 16 and the results are shown in Tables 17 and 18. The results show that although the predicted bottom up fatigue cracking, reflective cracking, and fatigue fractures in the chemically stabilized layer are all sensitive to the modulus of rupture of the CSRB, the predicted distresses are all below the threshold values.

Table 16. PMED inputs for sensitivity analysis 3

Pavement structure: 4.0 in. Asphalt Concrete 10.0 in. Chemically-stabilized base (soil cement) 15.0 in. Subgrade 1 resilient modulus: 8,000 psi Subgrade 2 resilient modulus: 14,000 psi
Traffic (Two-way AADTT): 3,000
Climate: Thomas, GA
Sensitivity variables: Chemically-stabilized base thickness (in.): 8 to 12 in. Chemically-stabilized base modulus (psi): 150,000 psi to 4,000,000 psi. Modulus of Rupture: 150 psi to 400 psi

Table 17. Sensitivity analysis – modulus of rupture

Distress type	Target distress	Reliability (%)	Predicted distress		
			Modulus of Rupture (psi)		
			150	250	400
Terminal IRI (in/mile)	172	90	161.60	161.59	161.59
Permanent deformation - total pavement (in)	0.75	90	0.35	0.35	0.35
AC total fatigue cracking: bottom up + reflective (% lane area)	25	90	3.64	1.31	1.05
AC total transverse cracking: thermal + reflective (ft/mile)	2500	90	2952.79	2952.79	2952.79
AC bottom-up fatigue cracking (% lane area)	25	50	0.00	0.00	0.00
AC thermal cracking (ft/mile)	1000	50	596.64	596.64	596.64
Permanent deformation - AC only (in)	0.4	90	0.14	0.14	0.14
Chemically stabilized layer - fatigue fracture (% lane area)	25	50	2.88	0.54	0.29

Table 18. Sensitivity analysis summary (semi-rigid)

Sensitivity – Chemically Stabilized Base thickness

Distress type	Target distress	Reliability (%)	Predicted distress		
			Base thickness (in.)		
			8 in.	10 in.	12 in.
Terminal IRI (in/mile)	172	90	165.07	161.60	160.87
Permanent deformation - total pavement (in)	0.75	90	0.37	0.35	0.33
AC total fatigue cracking: bottom up + reflective (% lane area)	25	90	13.34	3.64	1.99
AC total transverse cracking: thermal + reflective (ft/mile)	2500	90	2943.92	2952.79	3004.05
AC bottom-up fatigue cracking (% lane area)	25	50	0.00	0.00	0.00
AC thermal cracking (ft/mile)	1000	50	588.19	596.64	631.49
Permanent deformation - AC only (in)	0.4	90	0.14	0.14	0.15
Chemically stabilized layer - fatigue fracture (% lane area)	25	50	12.50	2.88	1.27

Sensitivity - Chemically Stabilized Base modulus

Distress type	Target distress	Reliability (%)	Predicted distress		
			Base modulus (psi)		
			150,000	2,075,000	4,000,000
Terminal IRI (in/mile)	172	90	161.60	193.48	195.09
Permanent deformation - total pavement (in)	0.75	90	0.35	0.35	0.35
AC total fatigue cracking: bottom up + reflective (% lane area)	25	90	3.64	72.46	75.76
AC total transverse cracking: thermal + reflective (ft/mile)	2500	90	2952.79	2952.79	2952.79
AC bottom-up fatigue cracking (% lane area)	25	50	0.00	0.00	0.00
AC thermal cracking (ft/mile)	1000	50	596.64	596.64	596.64
Permanent deformation - AC only (in)	0.4	90	0.14	0.14	0.14
Chemically stabilized layer - fatigue fracture (% lane area)	25	50	2.88	71.70	75.00

The predicted distresses generally decrease with increasing thickness of the stabilized layer. These results also indicate that the modulus of the CSRB should be less than 150,000 psi to control asphalt concrete bottom-up fatigue cracking and reflective cracking, as well as fatigue fractures in the chemically stabilized layer.

CHAPTER SUMMARY

The FWD data of four different FDR pavement sections in Georgia were analyzed to determine their respective Impulsive Stiffness Modulus values. The structural layer coefficients of the CSRB were then backcalculated and a pavement overlay design performed with PMED using the FWD deflection data. Sensitivity analyses were also performed with PMED. The FDR pavement sections were modeled as conventional flexible pavements and semi-rigid pavements with a chemically stabilized base. A summary of this chapter's findings are as follows:

- The Impulsive Stiffness Modulus (ISM) decreased with time. A very low ISM value was estimated for the oldest pavement section, which had over 15 years in service. This could indicate a significant loss of stiffness in the FDR base.
- The back-calculated layer coefficient of the CSRB also decreased with time. Similar trends were observed for the UCS values of the FDR field cores and the correlated layer coefficients. It should be noted that the estimated layer coefficients varied with the correlation methods. The effective structural number approach yielded average layer coefficients ranging from 0.44 (Metcalf) to 0.25 (Smiley). Much lower ranges - 0.20 (SR 70) to 0.15 (Senoia) – were estimated from the field core UCS values, but these are still higher than the design layer coefficient of 0.14,

which is based on the design UCS value of 300 psi. The AASHTO 93 guideline recommends estimating the structural layer coefficient based on engineering judgement. The guide clearly states that the layer coefficient determined from the analysis of nondestructive testing is an approximation of the true structural capacity of the pavement [6].

- The PMED sensitivity analysis confirmed that the predicted distresses are largely insensitive to the CSRB properties. Among the material properties examined, the CSRB modulus was identified as being the critical property that has the greatest impact on AC bottom-up fatigue cracking/reflective cracking and fatigue cracking in the chemically stabilized layer. The GDOT specifies low strength for CSRB to minimize shrinkage/reflective cracking potential. Since the required strength of CSRB is well below the minimum modulus of the chemically stabilized layer in the PMED, the CSRB layer should be modeled as a high stiffness Graded Aggregate Base (GAB) layer in the PMED.

CHAPTER 4. CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

The objectives of this project were to improve the reliability of the current GDOT pavement design procedure for CSRB and to provide recommendations regarding the steps required for the verification and calibration of CSRB into MEPDG. The practices currently utilized by various State Departments of Transportation for the design of FDR projects were identified and reviewed and experiments performed to both analyze the properties of pulverized asphalt/base soil samples collected from ongoing FDR projects and characterize CSRB in the laboratory. In situ FWD tests were conducted for four FDR pavement sections aged from 3 to 15 years as part of the performance evaluation process, and CSRB field cores were extracted for forensic studies, focusing particularly on their Elastic modulus and UCS. A series of sensitivity analysis was run with PMED to identify the predominant design inputs affecting the performance of FDR pavements.

Based on the findings from the laboratory and field experiments and the PMED simulations, the following conclusions were drawn:

- The correlation chart provided in the 1993 AASHTO Guide and recommendations in the 1973 AASHTO 73 Interim Guide indicate that the minimum 7-day UCS of 300 psi corresponds to a layer coefficient range of 0.14 - 0.15 for CSRB.
- The layer coefficient of cement stabilized aggregate is higher than that of soil cement due to the difference in strength between the two materials, namely aggregate and soil. GDOT section 315 requires the reclaimed base mixture to have a similar gradation to that of the graded aggregate.

- The effect of sample size on the UCS and Elastic Modulus (E) values is not markedly different for samples created using two mold types: a 4.6-inch-tall standard steel mold and a 8-inch tall Plastic Mold (PM). This suggests that PM is indeed suitable for use in the proposed CSRB characterization process.
- Field cores exhibit high variations in E and UCS, which likely stem from the randomness of the reclaimed base mixture as this is inevitably affected by the gradation, mineralogy, and aggregate physical properties of the material utilized.
- The comparison between the proposed E-UCS models and the existing AASHTO models reveals that the common soil-cement model is likely to be inadequate for CSRB, especially at early ages.
- The PCA model overestimates the MoR of CSRB slightly more than the AASHTO model does.
- The results of the FWD tests indicate that the overall stiffness of the pavement section increases in the short term (up to 3 years) and then gradually decreases, as seen in the pavement sections over 5 years old (Metcalf and Senoia Road). A significant loss of CSRB stiffness was observed after 15 years of service (Smiley Cross Road). This means the structural layer coefficients computed from FWD deflections and UCS correlations show a decreasing trend with age. As CSRB ages, the prediction gaps for the structural layer coefficients widen.
- The sensitivity analysis with PMED shows that apart from reflective cracking, the predicted pavement distresses appear to be insensitive to changes in E and the layer thickness.

- Most of the cracking observed (AC bottom-up fatigue cracking/reflective cracking and fatigue cracking) in the chemically stabilized layer is closely linked to the magnitude of the CSRB modulus.

RECOMMENDATIONS

- Based on the calculated average layer coefficient from UCS of the field cores, the study recommends the design layer coefficient of 0.22 for the CSRB layer until additional field data supports higher layer coefficient.
- The CSRB should be modeled as a high stiffness Graded Aggregate Base (GAB) layer, as the required strength of CSRB is well outside the minimum and maximum modulus of the chemically stabilized layer in the PMED. As reported in Chapter 3, a minimum resilient modulus of 60,000 psi is recommended for the GAB layer when designing the CSRB layer with PMED.
- The success of the calibration of the PMED depends on the quality and volume of data on the pavement structure, materials, performance, traffic, and maintenance/rehabilitation records. GDOT should include pavement sections with CSRB in the MEPDG calibration sites to collect the necessary performance data. A previous study provides a useful reference to guide the development of a data collection plan for local calibrations of the MEPDG for use in Georgia [20]. GDOT's Office of Materials and Testing (OMAT) should continue to assess the performance of the CSRB sections and collect a wide range of performance data to support PMED calibration. Table 19 presents a list of data suggested for a sight length of 500 ft [20, 21] as an example.

Table 19. Data for PMED Calibration

Data	Data collection frequency
Pavement distress [1] • International Roughness Index (in/mile) • Permanent deflection - total pavement (in) • AC total fatigue cracking: bottom up + reflective (% lane area) • AC total transverse cracking: thermal + reflective (ft/mile) • AC bottom-up fatigue cracking (% lane area) • AC thermal cracking (ft/mile) • Permanent deformation depth- AC only (in) • Chemically stabilized layer - fatigue fracture (% lane area)	Every 2 years
Traffic	Annually
Material properties and layer thickness • 28-day modulus of elasticity • 7-day Unconfined Compressive Strength	Once after construction
Pavement structure capacity • Falling Weight Deflectometer	Every 2 years

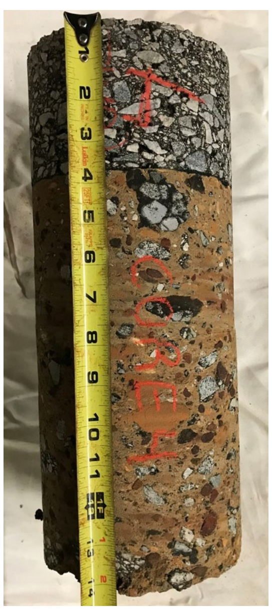
[1] Collect according to the methods recommended in the LTPP distress protocol [20]

APPENDIX A. CORE PHOTOS

Core 1

Core 2

Core 3



Core 4

Core 5

Core 6

Figure 13. Photos. Field cores from Metcalf Road pavement section

Core 1

Core 2

Core 3

Core 4

Core 5

Core 6

Figure 14. Photos. Field cores from Senoia Road pavement section

Core 1

Core 2

Core 3

Core 4

Core 5

Core 6

Core 7

Core 8

Figure 15. Photos. Field cores from Smiley Cross Road pavement section

APPENDIX B. LABORATORY TEST RESULTS

Table 20. USC and Modulus raw data (Laboratory specimens)

Cement Content %	Location: Peachtree City 1			Location: Winder			Location: Peachtree City 2		
	Sample ID	UCS psi	E psi	Sample ID	UCS psi	E psi	Sample ID	UCS psi	E psi
3	P-1	359.7		W-1	49.3		P2-1	113.8	
	P-2	321.5		W-2	47.0		P2-2	117.8	
	P-3	226.0		W-3	43.8		P2-3	121.0	
	P-4	429.7	3158.3	W-4	47.0	544.5	P2-4	120.2	1682.1
	P-5	377.2	2349.7	W-5	31.0	837.6	P2-5	122.5	1244.5
	P-6	241.9	2805.3	W-6	29.4	529.5	P2-6	121.8	1219
							P2-extra	127.3	819.2
5	P-7	341.4		W-7	119.4		P2-7	263.4	
	P-8	327.9		W-8	117.8		P2-8	262.6	
	P-9	304.8		W-9	112.2		P2-9	258.6	
	P-10	343.8	1406.8	W-10	104.2	1328.3	P2-10	269.0	3995.9
	P-11	323.1	2182.5	W-11		3888.8	P2-11	273.0	1811.3
	P-12	433.7	2389.1	W-12	110.6	2279.9	P2-12	265.8	2815.3
7	P-13	475.1		W-13	207.7		P2-13	427.3	
	P-14	463.1		W-14	226.8		P2-14	434.5	
	P-15	491.0		W-15	232.4		P2-15	408.2	
	P-16	546.7	2597	W-16	265.0	3515	P2-16	432.1	5342.8
	P-17	401.9	4331.3	W-17	243.5	2441.5	P2-17	405.0	4571.4
	P-18	481.4	1854.3	W-18	226.8	3392.8	P2-18	465.5	3044.5
9	P-19	740.9		W-19	428.9		P2-19	719.4	
	P-20	796.6		W-20	413.0		P2-20	600.0	
	P-21	875.4		W-21	411.4		P2-21	678.0	
	P-22	861.8	2515.1	W-22	422.6	2069.8	P2-22	660.5	7825.9
	P-23	N/A	3456.1	W-23	406.6	4378.3	P2-23	627.1	5166
	P-24	682.0	3165.6	W-24	419.4	2821.9	P2-24	740.9	8868.4
							P2-extra	725.7	6620.3

Table 21. Flexural bending test raw data

Location: Peachtree City 1				
Cement content, %	Sample No	Peak load, lb	Average load, lb	Stdev
7	1	500	433.3	65.1
	2	430		
	3	370		
9	1	530	553.3	68.1
	2	500		
	3	630		
Location: Winder				
Sample No	Cement content, %	Peak load, lb	Average load, lb	Stdev
7	1	350	370	28.3
	2	390		
	3			
9	1	530	540	14.1
	2	550		
	3			
Location: Peachtree City 1				
Sample No	Cement content, %	Peak load, lb	Average load, lb	Stdev
7	1	740	723.3	76.4
	2	640		
	3	790		
9	1	950	940	45.8
	2	980		
	3	890		

Table 22. USC and Modulus raw data (Field core specimens)

Section	Sample	UCS (psi)	Average UCS (psi)	Stdev	E (psi)	Average E (psi)	Stdev
SR 70	1	815.6	687	230.3	N/A	199,715	64,588.8
	2	488.4			N/A		
	3	895.2			N/A		
	4	353.3			129,118		
	5	651.5			255,840		
	6	917.8			214,188		
Metcalf	1	619.6	643	133.2	N/A	25,170	11,048.2
	2	812.0			N/A		
	3	728.6			N/A		
	4	548.6			37,833		
	5	706.3			20,183		
	6	444.6			17,495		
Senoia	1	406.4	380.8	178.6	N/A	239,954	117,189.5
	3	245.5			N/A		
	4	309.8			322819		
	5	680.1			*9021*		
	6	262.4			157088		
	Sample 5 is an outlier and excluded from modulus calculation						
Smiley	2	229.2	185.5	30.6	Unable to measure elastic modulus due to specimen size		
	4B	174.7					
	6	158.1					
	7	180.0					

APPENDIX C. CSRB LAYER COEFFICIENT CALCULATION

FWD location: SR70 SB-204

Step 1. Subgrade resilient modulus, M_R

The subgrade resilient modulus can be calculated as follows

$$M_R = \left(\frac{0.24\ P}{d_r\ r}\right) = \left(\frac{0.24\ \times\ 9{,}503}{1.52 \times 0.001 \times 36}\right) = 41{,}678\ \text{psi}$$

where, P = Applied load (P = 9,503 lb)

d_r = Deflection at a distance r from the center of the load (d_r = 0.00151 in.)

r = Distance from center of load (r = 36 in.)

Step 2. Effective modulus of pavement layers above the subgrade, E_P

The effective modulus of the pavement layers is computed using the following equations.

$$d_o = 1.5\ p\ a \left\{ \frac{1}{M_R \sqrt{1 + \left(\frac{D}{a} \sqrt[3]{\frac{E_P}{M_R}}\right)^2}} + \frac{\left[1 - \frac{1}{\sqrt{1 + \left(\frac{D}{a}\right)^2}}\right]}{E_P} \right\}$$

$$0.00232 = 1.5 \times 86.6 \times 5.91 \left\{ \frac{1}{41{,}678 \sqrt{1 + \left(\frac{20}{5.91} \sqrt[3]{\frac{E_P}{41{,}678}}\right)^2}} + \frac{\left[1 - \frac{1}{\sqrt{1 + \left(\frac{20}{5.91}\right)^2}}\right]}{E_P} \right\}$$

The effective modulus of the pavement layers is calculated using Excel's built-in What-If Analysis tool, Goal Seek,

$$E_P = 1{,}100{,}000 \text{ psi}$$

where, do = Deflection measured at the center of the load plate (do = 2.32 mils)

p = NDT load plate pressure (p = 86.6 psi)

a = NDT load plate radius (a = 5.91 in.)

M_R = Subgrade resilient modulus (M_R = 41,678 psi)

Step 3. Effective structural number, SN_{eff}

The effective structural number is computed using the following equation.

$$SN_{eff} = 0.0045\, D \sqrt[3]{E_p} = 0.0045 \times 20 \sqrt[3]{1{,}100{,}000} = 9.29$$

where, D = Total thickness of all pavement layers above the subgrade (D = 20 in.)

E_p = Effective modulus of pavement layers above the subgrade (E_p = 1,100 ksi)

Step 4. Structural layer coefficient of the CSRB

The structural capacity for an idealized two-layer pavement structure (HMA and CSRB) can be determined as follows

$$SN_{eff} = a_1D_1 + a_2D_2$$

Where, a_i = layer coefficient for layer i

D_i = layer thickness for layer i

The layer coefficient for CSRB is calculated by rearranging the above equation as follows

$$a_2 = \frac{SN_{eff} - a_1 D_1}{D_2} = \frac{9.29 - 0.42 \times 5}{15} = 0.48$$

where, a_1 = Layer coefficient of HMA (0.42 for the SR70 and 0.3 for the other sections)

D_i = layer thickness for layer i (D_1 = 5 in. and D_2 = 15 in.)

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REFERENCES

1. AASHTO (2020). *Mechanistic-Empirical Pavement Design Guide: A Manual of Practice: Third Edition,* American Association of State Highway and Transportation Officials, Washington, DC.

2. GDOT (2022). *Pavement Design Manual Revision 4.0*, Georgia Department of Transportation. Atlanta, GA.

3. AASHTO (1972). *AASHTO Interim Guide for Design of Pavement Structures*, American Association of State Highway and Transportation Officials, Washington, DC.

4. AASHTO (1993). *AASHTO Guide for Design of Pavement Structures*, American Association of State Highway and Transportation Officials, Washington, DC.

5. Von Quintus, H. L., Darter, M. I., Bhattacharya, B. B., and Titus-Glover, L. (2016). *Implementation and Calibration of the MEPDG in Georgia*, FHWA/GA-014-11-17, Applied Research Associates, Inc., Federal Highway Administration, Washington, DC.

6. Von Quintus, H. L., Darter, M. I., Bhattacharya, B. B., and Sadasivam, S. (2015). *Implementation of the Mechanistic-Empirical Pavement Design Guide in Georgia: Georgia DOT Pavement ME Design User Input Guide*, FHWA/GA-014-11-17, Applied Research Associates, Inc., Federal Highway Administration, Washington, DC.

7. AASHTO (2010). *Guide for the Local Calibration of the Mechanistic-Empirical Pavement Design Guide*, American Association of State Highway and Transportation Officials, Washington, DC.

8. Kwon, J., Seo, Y., Kaplan, A., and Yang, J. (2020). *Full Depth Pavement Reclamation: Performance Assessment and Recommendations for Best Performance*, GDOT-GA-20-1822, Georgia Department of Transportation, Atlanta, GA.

9. Hossain, M. S., Nair, H., and Ozyildirim, H. C. (2017). *Determination of Mechanical Properties for Cement-Treated Aggregate Base*, FHWA/VTRC17-R21, Virginia Department of Transportation, Richmond, VA.

10. Carey, A. S., Cooley, L. A., Middleton, A., Sullivan, W. G., Ayers, L. E. W., and Howard, I., L. (2021). "Statewide Spectra of Chemically Stabilized Soil Strength and Modulus Properties Applicable to Mechanistic-Empirical Pavement Design.", *101st Annual Meeting of the Transportation Research Board*, Transportation Research Board of the National Academies, Washington, DC.

11. GDOT (2018). *GDT 49-Determining the Theoretical Maximum of Dry Density of Materials containing > 25% retained on the no. 10 sieve using a 10-pound rammer and an 18-inch drop*, Georgia Department of Transportation. Atlanta, GA.

12. GDOT (2018). *GDT 65- Laboratory Design of Soil-Cement and Cement Stabilized Graded Aggregate*, Georgia Department of Transportation. Atlanta, GA.

13. AASHTO (2019). *AASHTO PP 92 Preparation of Test Specimens Using the Plastic Mold Compaction Device*, American Association of State Highway and Transportation Officials, Washington, DC.

14. ASTM (2022). *C78 Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)*, ASTM International, West Conshohocken, PA.

15. ASTM (2017). *D1633 Standard Test Methods for Compressive Strength of Molded Soil-Cement Cylinders*, ASTM International, West Conshohocken, PA.

16. ASTM (2002). *C469 Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression*, ASTM International, West Conshohocken, PA.

17. Brown, C. E. (1998). *Applied Multivariate Statistics in Geohydrology and Related Sciences*, Springer-Verlag, New York.

18. Lee, S.I., Faruk, A.N.M, Hu, X., Haggerty, B., and Walubita, L.F. (2017). "Alternative Laboratory Test Method and Correlations to Estimate Modulus of Rupture of Cement Treated Based Materials.", *96th Annual Meeting of the Transportation Research Board*, Transportation Research Board of the National Academies, Washington, DC.

19. Kim, S. S., Worthey, H., Brink, W., Von Quintus, H. L., Durham, S.A., and Chorzepa, M. (2020). *Innovative Training Modules for Rapid and Continuous Deployment of MEPDG.* FHWA-GA-20-1718, Georgia Department of Transportation, Atlanta, GA.

20. Tsai, Y., Wu, Y., and Salameh, R. (2020). *Georgia's Long-term Pavement Performance Monitoring and Management Plan: Detail Design of Plan.* FHWA-GA-19-1730, Georgia Department of Transportation, Atlanta, GA.

21. FHWA (2003). *Distress Identification Manual for the Long-Term Pavement Performance*, FHWA-RD-03-03, Federal Highway Administration, Washington, DC.