GEORGIA DOT RESEARCH PROJECT 14-13

FINAL REPORT

Evaluation of NCHRP 747 Pavement Forensic Guide for GDOT's Adoption

Georgia Department of Transportation

OFFICE OF PERFORMANCE-BASED MANAGEMENT AND RESEARCH
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16. Abstract: This report presents the recommendation for whether the Georgia Department of Transportation (GDOT) should adopt the National Cooperative Highway Research Program (NCHRP) Report 747 (Guide for Conducting Forensic Investigations of Highway Pavements) as a guide to conduct forensic investigations in Georgia. This report documents the evaluation of three pavement types using the NCHRP 747 guide: Jointed Plain Concrete (JPC), Continuously Reinforced Concrete (CRC), and Hot Mix Asphalt (HMA). Each pavement type consisted of an evaluation of two sites, one in "good/fair" condition and the other in "poor" condition. Non-destructive testing was performed using Ground Penetration Radar (GPR) and Falling Weight Deflectometer (FWD). Destructive testing (coring) and on-site field testing was performed consistent with the recommendations of the forensic guide. Laboratory tests were conducted to determine material properties of the existing pavement. These results were combined with traffic data and weather information to form conclusions about the cause(s) of pavement distress in sections of "poor" condition. Based on the successful investigations, it is recommended to adopt the NCHRP Report 747 for use in Georgia.			
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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)

GDOT Research Project No. 14-13

Final Report

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EXECUTIVE SUMMARY

State agencies and civil engineers are responsible for the safe travel of millions of vehicle passengers. The roads traveled must be structurally sound and provide a smooth ride. Surely newer roads satisfy these requirements, but throughout the years, roads worsen and need to be either resurfaced, rehabilitated, or reconstructed. Additionally, transportation agencies are starting to discover that some pavements stay in good condition over time, while others have deteriorated significantly. The University of Georgia (UGA) and the Georgia Department of Transportation (GDOT) have collaborated to investigate the cause of this phenomenon.

To discover the cause of these pavement conditions, the National Cooperative Highway Research Program (NCHRP) Report 747 (Guide for Conducting Forensic Investigations of Highway Pavements) was used as a guide. In the absence of an official document for conducting forensic investigation in Georgia, GDOT desires to evaluate and review this latest document for compatibility with current GDOT practices. If discrepancies exist, modifications will need to be developed and presented to GDOT for acceptance.

This report is composed of the evaluations of three types of pavements using NCHRP Report 747: Jointed Plain Concrete Pavement (JPCP), Continuously Reinforced Concrete Pavement (CRCP), and Hot Mix Asphalt (HMA) Pavement. Each pavement was investigated through a site investigation that contained visual inspection and non-destructive/destructive testing. Non-destructive methods include a visual inspection, Ground Penetration Radar (GPR), and Falling Weight Deflectometer (FWD). Destructive testing includes coring samples and laboratory tests performed on the respective samples. Cored specimens were tested to measure the material's physical and strength properties. A series of tests for concrete cores includes the measurement of coefficient of thermal expansion, rapid chloride permeability, compressive strength, modulus of elasticity, alkali-silica reactivity, and carbonation reactivity. Asphalt cores were also tested to determine susceptibility to rutting and stripping, maximum specific gravity, bulk specific gravity, air void content, and binder content.

Based on the experience of conducting a forensic investigation using the NCHRP 747 Report through this study, it is recommended for GDOT's adoption of the NCHRP 747 Report as the Forensic Pavement Guide for Georgia, with the following additions/recommendations:

- A comprehensive forensic investigation is very extensive, expensive, and time consuming. Precautions should be exercised to determine whether a full investigation is

needed. It is recommended to determine the level of forensic analysis based on the "Phased Approach to Forensic Investigations" diagram in the NCHRP 747 Guideline (Appendix A).

- Rather than using NCHRP visual condition survey form, it is recommended to use the GDOT's visual inspection forms that have been used for PACES update. However, development of new methodology to assess PACES rating for CRCP is strongly recommended as current methodology does not reflect the functional condition evaluation of CRCP properly. It is believed that new methodology to assess PACES rating for CRCP will be developed through RP 15-02, "Developing a Comprehensive Pavement Condition Evaluation System for Rigid Pavements in Georgia".
- Based on this RP 14-13 study, flow charts for pavement forensic investigations were developed (Appendices D, E, and F). The flow charts will provide the GDOT engineers with a systematic procedure when pavement forensic investigations are deemednecessary.
- Traffic information along with pavement service life has large impact on pavement design and performance. To accurately investigate pavement performance, it is recommended that traffic information is efficiently archived and easily accessible. This includes traffic volumes, traffic loads/load spectra, traffic growth, seasonal trends, load restrictions, and any related traffic information during entire pavement service life.
- It is recommended that all construction documents be efficiently archived and easily accessible when forensic investigations are started. This includes all construction drawings, rehabilitation history, mix design, and other construction information.

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ABBREVIATIONS LIST

Abbreviation

AASHTO	American Association of State Highway and Transportation Officials
AC	Asphaltic Content
AGBS	Aggregate Base
BCO	Bonded Concrete Overlay
Gmb	Bulk Specific Gravity
Caltrans	California Department of Transportation
CDOT	Colorado Department of Transportation
CRC	Continuously Reinforced Concrete
DCP	Dynamic Cone Penetrometer
DOT	Department of Transportation
DT	Destructive Testing
FDR	Full Depth Repair
FHWA	Federal Highway Administration
FWD	Falling Weight Deflectometer
GPR	Ground Penetration Radar
HMA	Hot Mix Asphalt
HWT	Hamburg Wheel Tracking
ISM	Impulsive Stiffness Modulus
JOR	Joint Repair
JPC	Jointed Plain Concrete
NCHRP	National Cooperative Highway Research Program
NDT	Non-Destructive Testing
PDR	Partial Depth Repair
RCP	Rapid Chloride Permeability
RDD	Rolling Dynamic Deflectometer
RWD	Rolling Wheel Deflectometer
SP	Superpave
SSD	Saturated Surface Dry
STI	Stitching
Gmm	Theoretical Maximum Specific Gravity
TRB	Transportation Research Board
UCO	Unbonded Concrete Overlay

1. INTRODUCTION

1.1 Problem Statement

The National Cooperative Highway Research Program (NCHRP) Report 747 (Guide for Conducting Forensic Investigations of Highway Pavements) was released in 2013. The report explored the process for conducting forensic investigation of pavements to help understand the reasons behind premature failures or exceptionally good performance. The report recommended performing both functional and structural evaluation of pavements for forensic study. It provides general guidance on the organization and planning of the forensic investigation, sampling and testing requirements, analysis of results, and the decision making process. In the absence of a guide for conducting forensic investigation in Georgia, the Georgia Department of Transportation (GDOT) desires to evaluate and review this latest document for compatibility with current GDOT practices. If discrepancies exist, modifications will need to be developed and presented to GDOT for acceptance.

The Michigan Department of Transportation (DOT) conducted a similar research study by evaluating six concrete pavement systems with materials-related distress (Sutter et al. 2010). Factors contributing to concrete pavement distress included extensive alkali-silica reactivity and freeze-thaw deterioration related to poor entrained air-void parameters. The Ohio DOT performed a forensic investigation in 2006 to determine the reasons for differences in performance of ten flexible pavements (Qin et al. 2013). Despite numerous research projects in this field, state transportation agencies seldom provide a formally written forensic pavement investigation guide.

Current forensic pavement investigation techniques consist of non-destructive and destructive tests applicable to both rigid and flexible pavement systems. These techniques are necessary to determine the strength and serviceability of a pavement system. Without on-going and well-structured forensic pavement investigation programs to detect current issues and prevent damage, unwanted downtime and loss of money will be inevitable to mitigate neglected problems (Rens et al. 1997).

1.2 Objectives

The primary goal of this research study is to evaluate the NCHRP 747 for GDOT's adoption by performing functional and structural evaluations of existing asphalt/concrete pavements. Specific objectives for this study include: (1) Conduct functional and structural evaluation to identify causes of distress on asphalt and concrete pavements based on the NCHRP 747 Forensic Guide; (2) Provide a recommendation whether the Georgia guide is warranted based on the functional and structural evaluation in accordance with the Forensic guide; (3)If warranted, develop a GDOT version of the Pavement Forensic Guide by considering GDOT practices, the unique characteristics of pavements, materials, and weather conditions in Georgia.

2. LITERATURE REVIEWS

2.1 GDOT Nationwide Survey

2.1.1 Introduction and Motivation

Before conducting pavement forensic investigations, a national survey was conducted to help the research team understand and possibly enhance forensic evaluation methods. The survey was distributed to each state DOT in North America. Questions were specifically engineered to inquire whether the participant used a pavement forensic guide to examine the functional and structural condition of existing rigid and/or flexible pavement. The survey also asked what specialized tests were used for pavement forensic analysis (Falling Weight Deflectometer (FWD), Ground Penetration Radar (GPR), Rapid Chloride Permeability (RCP) tests, etc.) and inquired about methods for maintenance and/or rehabilitation such as Full Depth Repair (FDR), Partial Depth Repair (PDR), Bonded Concrete Overlay (BCO), Unbonded Concrete Overlay (UCO) (FHWA 2003) and Joint Repair (JOR). The survey participants were provided an opportunity to attach supporting documentation and/or give additional comments. In total, responses were received from 32 state DOT's as shown in Figure 1. Four responses were received from provinces in Canada.

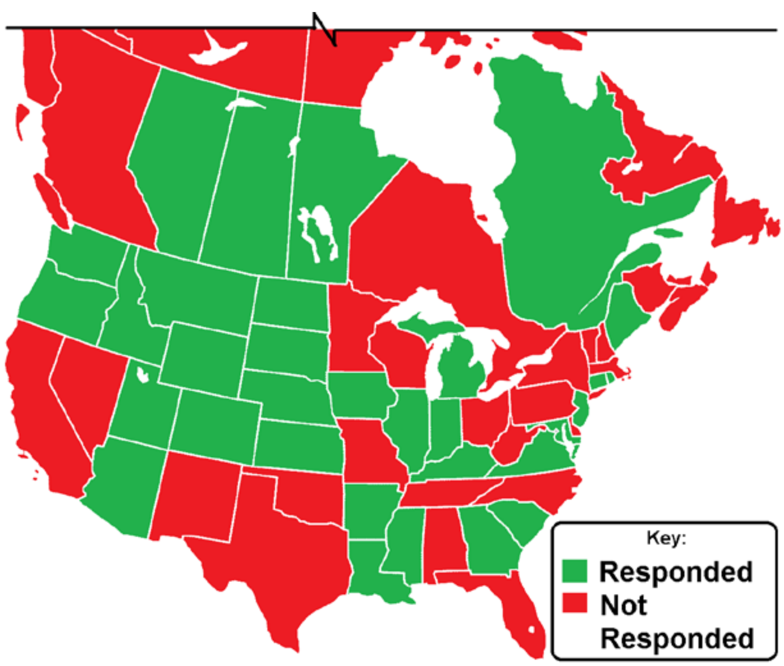


Figure 1 - Survey Responses in North America.

The following sections present survey results of state practices and a review of available forensic pavement investigation techniques. The primary purpose of the survey was to research the most commonly used forensic investigation methods that other state DOTs use and study the causes of pavement distresses and/or failure contributing to differences in performance (average, below average, above average) among rigid and flexible pavement systems in other states. Therefore, it should be noted that this survey focuses on structural distress types and possible causes in pavement systems. Functional distresses such as riding quality and skid resistance can vary widely from state-to-state, by climate, and pavement types. Thus, they are not intended to be a part of this survey although participants of this survey have shared a few functional evaluation methods.

2.1.2 Adoption of Pavement Forensic Guide

When asked if the respondent's DOT uses a pavement forensic guide to examine the functional and structural condition of existing rigid and/or flexible pavement, only 4 participants (12.5%) responded that their DOT maintained a pavement forensic guide as shown in Table 1. However, 39% of DOTs without a guide are interested in considering one. For the DOTs that did have a forensic guide, they all displayed positive views of their respective guides. Additionally, 3 DOTs (Colorado, Wyoming, and Quebec) have adopted the NCHRP 747 into their pavement practices. Colorado has a neutral opinion of the guide. Wyoming felt they have not had a chance to fully evaluate the guide. Quebec has their own procedures to conduct forensic investigations, but are evaluating the guide to see if it contains any information that they can add to their procedures.

Table 1 - Survey Results: States and Canadian Provinces with a Forensic Pavement Investigation Guide

Response	Respondent	Percentage
Yes	5**	15.6 %
No	27	84.4 %
Total*	32	100.0 %

Note: * includes 28 US states and 4 Canadian provinces.
** States/Provinces with a guide - Colorado, Maryland, Montana, Saskatchewan, and Quebec.

2.1.3 Forensic Technologies

Subsequently, the participants were asked whether certain forensic pavement testing technologies were in use by their DOTs, as shown in Table 2. FWDs were the most widely used technology, with 91% of DOT's confirming their use and 96% stating they would recommend the technology. Other technologies include the GPR (Usage=59%, Recommendation=95%), RCP Test (Usage=22%, Recommendation=50%), Dynamic Cone Penetrometer (DCP) (Usage=50%, Recommendation=81%), and the Rolling Dynamic Deflectometer (RDD) (Usage=0%, Recommendation=40%). Although RDD is recommended by multiple state DOTs, no usage was reported in the survey. In addition, the respondents were allowed to add any additional testing that was not listed on the survey. 10 DOTs listed technologies such as Rolling Wheel Deflectometer (RWD), boring, skid resistance, pipe cameras, and base/subgrade samples. Six (6) DOTs specifically mentioned that they use coring.

2.1.4 Rehabilitation Methods

Subsequently, the participants were asked whether certain pavement rehabilitation technologies were in use by their DOT, which is shown in Table 3. The top three used rehabilitation methods are PDR (Usage=97%, Recommendation=97%), FDR (Usage=91%, Recommendation=100%), and JOR (Usage=84%, Recommendation=96%). Other rehabilitation technologies include: BCO (Usage=25%, Recommendation=63%), UCO (Usage=59%, Recommendation=80%), and Stitching (STI) (Usage=47%, Recommendation=76%). The respondents were allowed to add any additional method that was not listed on the survey. Nine DOTs mentioned rehabilitation methods such as Dowel Bar Retrofitting, Rubblization (TRB 2006), Asphalt-Concrete Overlay, Diamond Grinding (FHWA 2016d), and Pavement Preservation Treatments. Table 4 summarized these methods.

Table 2 - Survey Results: Non-destructive and Destructive Testing Methods Used

State/Province	FWD	GPR	RCP	DCP	RDD
Alberta	✓	✓	—	—	—
Arizona	✓	—	✓	✓	—
Arkansas	✓	✓	—	—	—
Colorado	✓	—	✓	—	—
Connecticut	—	—	—	—	—
Georgia	✓	—	—	—	—
Idaho	✓	✓	—	—	—
Illinois	✓	—	—	✓	—
Indiana	✓	✓	—	✓	—
Iowa	✓	✓	—	✓	—
Kansas	✓	—	✓	✓	—
Kentucky	—	✓	—	—	—
Louisiana	✓	✓	—	✓	—
Maine	✓	✓	—	✓	—
Manitoba	✓	✓	—	—	—
Maryland	✓	✓	—	—	—
Michigan	✓	✓	—	—	—
Mississippi	✓	—	—	—	—
Missouri	✓	✓	✓	✓	—
Montana	✓	✓	—	✓	—
Nebraska	✓	—	✓	—	—
New Jersey	✓	✓	—	✓	—
North Dakota	✓	—	—	—	—
Oregon	✓	✓	—	✓	—
Quebec	✓	—	—	✓	—
Rhode Island	✓	✓	—	—	—
Saskatchewan	✓	—	—	—	—
South Carolina	✓	—	✓	✓	—
South Dakota	✓	✓	—	✓	—
Utah	✓	✓	—	✓	—
Virginia	✓	✓	—	✓	—
Washington	—	—	—	—	—
Wyoming	✓	—	✓	—	—
Responses	**29**	**19**	**7**	**16**	**0**
Percentage	**91%**	**59%**	**22%**	**50%**	**0%**

Note: ✓ Yes; — No (or No Response)

Table 3– Survey Results: Rehabilitation Methods Used.

State/Province	FDR	PDR	BCO	UCO	STI	JOR
Alberta	✓	✓	–	–	✓	✓
Arizona	✓	✓	–	–	–	✓
Arkansas	–	✓	–	✓	–	✓
Colorado	✓	✓	–	✓	✓	✓
Connecticut	✓	✓	–	–	–	–
Georgia	✓	–	–	–	–	✓
Idaho	✓	✓	–	✓	✓	✓
Illinois	✓	✓	✓	✓	–	–
Indiana	✓	✓	✓	✓	✓	✓
Iowa	✓	✓	✓	✓	✓	✓
Kansas	–	✓	✓	✓	✓	✓
Kentucky	✓	✓	–	–	–	✓
Louisiana	✓	✓	✓	✓	–	✓
Maine	✓	✓	–	–	–	✓
Manitoba	✓	✓	–	–	✓	✓
Maryland	✓	✓	–	–	–	✓
Michigan	✓	✓	–	✓	–	✓
Mississippi	✓	✓	–	–	–	✓
Missouri	✓	✓	✓	✓	✓	✓
Montana	✓	✓	✓	–	✓	✓
Nebraska	✓	✓	–	✓	✓	✓
New Jersey	✓	✓	–	–	–	✓
North Dakota	✓	✓	–	✓	✓	✓
Oregon	✓	–	–	–	–	–
Quebec	✓	✓	✓	✓	✓	✓
Rhode Island	✓	✓	–	–	–	✓
Saskatchewan	✓	✓	–	–	–	✓
South Carolina	✓	✓	–	✓	–	–
South Dakota	✓	✓	–	✓	✓	✓
Utah	✓	✓	–	✓	✓	✓
Virginia	✓	✓	–	✓	–	✓
Washington	–	✓	–	✓	–	–
Wyoming	✓	✓	✓	✓	✓	✓
Responses	**29**	**31**	**8**	**19**	**15**	**27**
Percentage	**91%**	**97%**	**25%**	**59%**	**47%**	**84%**

Note: ✓ Yes; – No (or No Response)

Table 4 – Other Forensic Investigation/Rehabilitation Methods Provided during the Survey.

State/Province	Forensic technologies	Rehabilitation methods
Colorado	Hamburg and French Rut tests	Diamond Grinding
Illinois	Coring / lab testing	Asphalt Overlays
Indiana	Pavement Coring	Retrofit dowel bars & Retrofit underdrain crack & Seat & overlay Rubblize & overlay Preventive and functional overlay
Iowa	–	Diamond grinding
Kentucky	–	Dowel Bar Retrofit
Louisiana	Rolling Wheel Deflectometer, Laboratory testing of field acquired specimens, Component method outlined in 1993 AASHTO guide, and MEPDG.	Rubblization and AC overlay, and numerous pavement preservation treatments
Maryland	Cores and Borings	A whole host of other pavement preservation treatments
Michigan	Pipe cameras, HMA sampling/recovery, Concrete petrographic analysis, Coring	Joint resealing Crack sealing/filling HMA milling/resurfacing HMA overlay Chip seal Microsurface Dowel bar retrofit Diamond grinding Crush and shape/resurfacing Aggregate lift/resurfacing Fog seal Paver placed surface seal
Missouri	–	Dowel Bar Retrofit
New Jersey	Lab testing samples extracted from the project: Composition analysis, APA rut, Overlay test, binder testing, etc.	–
Oregon	Coring and Base/Subgrade Samples	Localized punch-out repairs, which are full-depth Near the end of the CRCP service life, overlay the CRCP with 2 to 6 inches of asphalt”.
Quebec	Skid Resistance	–
South Carolina	Visual observation of cores	–

Note: – No Response

2.1.5 Other Published Forensic Pavement Guides and Resources

Respondents were encouraged to upload resources or send a pavement forensic guide if they had one or were willing to share one. Table 5 contains the resources and guides provided during the survey. Nine DOTs responded and attached a guide or web links: Alberta (Canada), Colorado, Illinois, Louisiana, Maryland, Michigan, Montana, South Carolina, and Quebec (Canada). A few DOTs provided links to their DOT website. Alberta, Illinois, Quebec, and South Carolina uploaded copies of their supporting forensic pavement literature.

Table 5 – Resources shared by state DOTs during the survey.

State	**Resources shared** (based on the survey conducted between June 2015 and January 2016).
Alberta	GUIDELINES FOR ASSESSING PAVEMENT PRESERVATION TREATMENTS AND STRATEGIES (web link) http://www.transportation.alberta.ca/Content/docType233/Production/gappts.pdf
Colorado	PROCEDURES FOR FORENSIC STUDY OF DISTRESS OF HOT MIX ASPHALT AND PORTLAND CEMENT CONCRETE (web link) https://s.qualtrics.com/ControlPanel/File.php?F=F_2OGvlIdbj3iHZsg
Illinois	Chapter 53- PAVEMENT REHABILITATION, BUREAU OF DESIGN & ENVIRONMENT MANUAL. (web link) http://www.idot.illinois.gov/Assets/uploads/files/Doing-Business/Manuals-Split/Design-And-Environment/BDE-Manual/Chapter%2053%20Pavement%20Rehabilitation.pdf
Louisiana	Pavement research (web link) http://www.ltrc.lsu.edu/preview/research_pavement.html
Maryland	Pavement & Geotechnical Design Guide (web link) http://sha.md.gov/OMT/MDSHA-Pavement-Design-Guide.pdf
Michigan	Pavement Design and Selection Manual (web link) http://www.michigan.gov/mdot/0,1607,7-151-9622_11044_11367---,00.html
Montana	Methods of Sampling and Testing, MT 329-04 - Procedure for Evaluating Plant Mix Surfacing Failures (web link) http://www.mdt.mt.gov/other/webdata/external/materials//materials_manual/329.PDF
Quebec	Rigid Pavement Maintenance and Rehabilitation Guide & Rigid Pavement Distress Identification Manual (web link) http://www3.publicationsduquebec.gouv.qc.ca/produits/ouvrage_routier.fr.html
South Carolina	Pavement Design Guidelines (web link) http://www.scdot.org/doing/technicalPDFs/materialsResearch/PavementDesignGuide2008.pdf

2.1.6 Additional Comments Provided by Pavement Engineers

The most frequent comment was that other DOTs would like to see the survey results. Many DOTs are evaluating the NCHRP 747 report for adoption as a forensic pavement guide or are interested in making their own.

- Illinois DOT has attached Chapter 53 of their Bureau of Design and Environment Manual. Although it has not been adopted as a formal forensic pavement guide, it is used for pavement evaluation and rehabilitation.
- Indiana uses various treatments, depending on the project and “type of roadway”.
- Louisiana stated that they are very experienced in conducting forensic evaluations and regard NCHRP 747 as an “excellent resource” and recommend it for “new engineers”.
- Michigan is reviewing NCHRP 747 for possible use in their DOT practices, but is not considering creating a new forensic pavement guide.
- Mississippi utilizes “pavement cores and FWD data” to conduct forensic evaluations on flexible pavement, but they do not have a published forensic pavement guide.

- Nebraska commented that “Many of the principles outlined in the NCHRP forensic guide are part of the pavement design process and are documented in our Pavement Design Manual.”
- Oregon is primarily a Continuously Reinforced Concrete Pavement (CRCP) state, and therefore Oregon DOT has not had a need for the types of rehabilitation presented in Table 3. It conducts “localized punch-out repairs, which are full-depth”. Near the end of the CRCP service life, Oregon DOT traditionally will “overlay the CRCP with 2 to 6 inches of asphalt”.
- Virginia DOT does not have a published forensic pavement guide, but uses the “Materials Division's Manual of Practice” to conduct pavement investigations and rehabilitate pavement.
- Wyoming commented that they have “had limited success with Bonded Concrete overlays on concrete, but have been very successful with Bonded Concrete on plant mix pavement”.

2.1.7 Discussion

Because only four agencies use a forensic guide for pavement investigation, it is difficult to conclude which techniques recommended in the NCHRP 747 report are preferred for implementation for use in a pavement investigation guide. However, investigation techniques currently used by multiple highway agencies prevail. Although FWD and GPR methods are used by 29 and 19 states/provinces, respectively, these methods are generally considered practical. For pavement rehabilitation, it was discovered that PDR (FHWA 2016c) and UCO are more popular than FDR (USDOT 2015) and BCO, respectively. Furthermore, JOR is common which suggests that improved maintenance and design processes are necessary.

It was discovered that states including Texas, New Mexico, and California provide a pavement investigation guide or maintenance program although they did not participate in the survey. Texas DOT appears to have a well-organized forensic pavement assessment process as well as rigid and flexible pavement rehabilitation methods available on its website (TxDOT 2015). The NCHRP report includes the Caltrans’ guide as well (2003). Moreover, California provides a well-organized Pavement Management System or PaveM (Caltrans

2016) which includes an Automated Pavement Condition Survey (APCS). The survey consists of collecting surface pavement sensor and image based distress data and analyzing data in conformance with the Department's APCS Manual. In addition, GPR technology is used to collect continuous layer thickness data.

New Mexico DOT has illustrated the benefits to pavement design, maintenance, and management through the use of non-destructive pavement testing technology, namely FWD and GPR, rather than destructive coring. Furthermore, the agency's effort and interest was also found in a recently completed research report (Bandini et al. 2012) for improving New Mexico DOT's pavement distress survey methodology and developing correlations between FHWA's Highly Polymer Modified (HPM) pavement distress data and Pavement Management System (PMS) data and pavement assessment projects (NMDOT 2016).

2.2 Pavement Types Review

This section provides a brief explanation of each pavement type.

2.2.1 Jointed Plain Concrete Pavement (JPCP)

Jointed Plain Concrete Pavement (JPCP) is a concrete slab that contains contraction joints in the transverse and longitudinal direction. These contraction joints control where the pavement cracks occur at specific locations. JPCP is commonly used in roadway construction as an economical choice. The performance depends on the load transfer efficiency and design parameters such as slab thickness, concrete strength, and dowel/joint spacing. Additionally, the pavement is known to have potential issues such as cracking (corner, longitudinal, transverse), faulting, and joint failure (Rada, 2013). These distresses can be prevented through a combination of proper design, construction, and/or material choice.

2.2.2 Continuously Reinforced Concrete (CRC) Pavement

Continuously Reinforced Concrete Pavement (CRCP) sections contain longitudinal reinforcement throughout the entire section. CRCP is known to maintain performance under heavy traffic loadings and challenging environmental conditions, provided proper design and quality construction practices are utilized (FHWA, 2012). This pavement has intentional longitudinal cracks that are held together tightly by the extensive reinforcement. The cracks, no larger than 0.02 inches, prevent moisture from penetrating the slab and damaging the pavement (PI, 2008). The most frequently observed distress in CRC pavements are punchouts (Rada, 2013). Closely spaced transverse cracks can be observed, although they are not necessarily detrimental to Georgia pavements.

2.2.3. Hot-Mix Asphalt (HMA) HMA Pavement

Superior Performing Asphalt Pavement System (SuperPave) was created in 1993 by the Strategic Highway Research Program (SHRP) (FHWA, 2010). This pavement is regarded as superior because it combines “asphalt binder and aggregate selection into the mix design process, and considers traffic and climate as well” (PI, 2011). An HMA pavement is typically constructed from a subgrade soil, subbase course, base course, and surface course. As the impact loading strikes the surface course, the load is distributed through each layer of material. The HMA flexes under loading, giving it the classification “flexible pavement”. HMA Pavements are susceptible to distresses such as cracking (alligator, transverse, longitudinal, and block), rutting, potholes, and raveling (Rada, 2013).

3. TESTING METHODOLOGY

3.1 Visual Inspection/Observation

Visual observation identifies patterns that reveal pavement deficiencies. The pavement distress is generally organized in a report that details the severity of the damage and how far the damage extends across the pavement. As part of the NCHRP Report 747, a visual inspection form for Asphalt Concrete (AC) and Portland Cement Concrete (PCC) pavement is provided as a guideline. A copy of these visual inspection guidelines can be viewed in Appendix 2 and 3.

GDOT has developed their own visual inspection method, Pavement Condition Evaluation System (PACES). Depending on the type of distress, the cause can be attributed to a certain factor, such as environmental conditions, poor construction practices, or increased traffic loading. For example, alligator cracks on flexible pavement surfaces generally indicate a load-related failure whereas block cracks largely result from an environmental-related failure. Linear cracking and corner breaks normally result from a load-related failure in rigid pavement surfaces while durability cracking is mostly due to an environmental-related failure. Generally, the condition of the pavement surface has been visually inspected periodically by experienced engineers for the purpose of computing the PACES rating. The PACES rating gives a numerical indicator that rates the surface condition of the pavement from 100 (Excellent Condition, no distress) to 0 (The worst Condition). Although the evaluation process may vary widely by jurisdiction, it provides a measure of pavement conditions based on the distress observed on pavement surfaces, as well as a practical indication of functional pavement condition and structural integrity.

3.2 Review of Pavement Forensic Technologies – Non-destructive

Although many performance problems show on the surface of the pavement, the cause is often attributed to issues within the pavement structure. Non-destructive testing (NDT) allows these issues to be located with precision, resulting in efficient repair. Furthermore, NDT is responsible for identifying problems that have not appeared on the pavement surface. The most common NDT technologies are Ground Penetrating Radar (GPR), Falling Weight Deflectometer (FWD), friction testers, and profilometers (Rada 2013). GPR and FWD technologies deal more with internal problems such as structural capacity and material properties, whereas friction testers and profilometers deal with external problems such as ride quality and road safety for functional evaluation (Rada 2013). It is necessary to describe each forensic investigation method in this section, in order to properly discuss the state DOT survey results.

3.2.1 Falling Weight Deflectometer

The need for non-destructive testing will usually be decided based on a visual assessment. A Falling Weight Deflectometer (FWD) is a device that applies a load to a pavement section and measures the resulting deflections (FHWA 2006). Figure 2 shows an image of the FWD harnessed to the back of a van. FWD equipment can quantify structural issues by means of measuring deflections. These deflections are measured in at least 7 locations along the test section using sensory instrumentation according to the American Society for Testing and Materials (ASTM), “Standard Test Method for Deflections with a Falling-Weight-Type Impulse Load Device” (ASTM-D4694, 2003). The standard gives instructions on conducting FWD tests to assess AC pavements and their respective parameters such as deflection, structural number, and elastic modulus, etc. (Bilodeau 2012). For site investigations, the FWD test is typically performed in one lane, unless there are thickness variations between lanes. The sections are then interpreted through software to give material properties and the pavement bearing capacity. Unfortunately, test location and temperature can influence FWD measurements and must be accounted for when calibrating the equipment. The Impulse Stiffness modulus (ISM) of the pavement sections is defined as the applied load (in kN) divided by the maximum deflection of the loading plate (in millimeter) (USDOT FAA 2011). Impulse Stiffness Modulus (ISM) plots display the stiffness over the length of the pavement, providing a simplified way to check for the overall strength of the pavement section.

Figure 2 - Falling Weight Deflectometer

3.2.2 Ground Penetration Radar

A Ground Penetration Radar (GPR) uses an antenna to send energy waves through pavement and monitors the surface reflection, or dielectric (ASTM-D6432 2011). A picture of a GPR is shown in Figure 3. The GPR rapidly and effectively analyzes layer thickness and detects problems such as “debonding, presence of moisture, voids under concrete slabs, and other issues that are normally assessed through coring” (Rada 2013; Zhao et al. 2016). This technology has been used by many DOTs to discover problematic areas in pavement. Using a GPR is very effective in detecting moderate to severe stripping in hot-mix asphalt (HMA) (Chen 2003). The GPR energy waves can penetrate approximately three feet (one meter) and can be operated at highway speeds, when attached to a vehicle, making it a useful addition to non-destructive testing technologies (Chen 2008). Unfortunately, the data results decrease in quality as the highway speeds increase, which may require road closures to receive accurate results. In addition, interpreting GPR data requires a technician with special training (Rada 2013). The GPR is wheeled over the pavement sections and the results are collected in the form of images that use colors to distinguish the variations of dielectric signals that differentiate material properties (Morey 1998). These images are then compared with core samples to verify pavement thickness.

Figure 3 - Ground Penetration Radar

3.2.3 Other Non-destructive Testing Techniques

The NCHRP report recommends the following non-destructive tests to explain issues or functional distress types being investigated: Profilometer (Praticò and Vaiana 2015), Skid Resistance/Friction (Rezaei and Masad 2013), Tire-Pavement Noise at the Source (Porras 2015), Texture Meter, Permeameter (Huang and Huang 2014), and Magnetic Tomography Technology (Stryk et al. 2013; Hoegh et al. 2012).

3.3 Review of Pavement Forensic Technologies – Destructive

3.3.1 Coring

Destructive testing (DT) is utilized where NDT techniques indicate potential pavement failures. Coring is a process where a 102 or 152-mm diameter cylindrical section is extracted from the pavement section. A coring machine is shown in Figure 4. A core sample shows a wide range of pavement layers (e.g., Subbase, Base, Subgrade, Concrete, and Asphalt mixture) that can be analyzed. When the core is taken, a borescope can photograph problem areas, such as voids. Layer thickness and/or cause of distress can be measured from taking core samples. Laboratory testing is conducted on cored specimens to reveal and confirm problems. In terms of the FHWA, the most common testing methods for concrete specimens are "compressive strength, modulus of elasticity (MOE), rapid chloride permeability (RCP), Alkali-Silica Reaction (ASR), Carbonation, and alternating current loop impedance" (Mallela 2006; Salgado and Yoon 2003). Air-void content, dynamic modulus test, Hamburg wheel track test, binder content, aggregate grading and properties, and resilient modulus are common for cores from flexible pavement. To address specific problems associated with pavement, state DOTs conduct many laboratory tests on freshly cored pavement samples.

Figure 4 - Coring machine

3.3.2 Rolling Dynamic Deflectometer (RDD)

A Rolling Dynamic Deflectometer (RDD) was developed as a non-destructive method for determining continuous deflection profiles of pavements in Texas (Nam et al. 2013). Unlike other commonly used pavement testing methods, the RDD performs continuous rather than separate measurements. Due to the low speed of measurements (< 3 mph), however, the use of RDD is not common.

3.3.3 Dynamic Cone Penetrometer (DCP)

A Dynamic Cone Penetrometer is used to determine underlying soil strength by measuring the penetration of the device into the soil after each hammer blow (Mejias-Santiago et al. 2015). DCP testing has been used to measure the relative strengths of stabilized and unstabilized aggregate base layers, and evaluate existing pavement base and subgrade layer strength during rehabilitation evaluations (MnDOT 2016).

3.4 Laboratory Testing Methods for Concrete Pavements

3.4.2 Alkali Silica Reaction (ASR) and Coefficient of Thermal Expansion (CTE)

Other on-site chemical tests such as carbonation and ASR tests and laboratory tests including petrographic analysis are recommended in the NCHRP 747 report (Rada 2013). It is known nationwide that a high percentage of slab cracks in concrete pavement systems may be attributed to high coefficient of thermal expansion (CTE) while the contributing factor for map cracking is generally the ASR (Kim 2012). This reaction causes the formation of a swelling gel of calcium-silicate-hydrate (CSH) and can ultimately cause serious cracking in concrete pavement. This gel increases in volume with water, and applies an expansive pressure inside the cementitious material, causing spalling and loss of strength, resulting in its structural failure.

3.4.3 Carbonation

A carbonation reaction results in a densification of the paste. The product mineral, calcite, is relatively insoluble in pore solution and its presence results in a permanent reduction in the capillary porosity of the paste (FHWA 2016b). Consequently, in a carbonation test, a diluted epoxy dye will penetrate into these areas, and they will exhibit little to no fluorescence compared to the uncarbonated areas of the same concrete, which would show high fluorescence (FHWA 2016b). Carbonation damage is rarely seen in Georgia pavements.

3.4.4 Rapid Chloride Permeability (RCP)

The Rapid Chloride Permeability (RCP) test is performed by monitoring the amount of electrical current that passes through a sample, a slice of a pavement core, which is 50 mm thick by 100 mm in diameter. The standardized testing procedures are provided in ASTM C 1202 (2012) or AASHTO T 277 (2008). A 60V DC voltage is maintained across the ends of the sample throughout the test. One lead is immersed in a 3.0% NaCl (salt) solution and the other in a 0.3 Molar concentration NaOH (sodium hydroxide) solution. Based on the charge (coulombs) that passes through the concrete sample, a qualitative rating is made of the concrete's permeability: High (>4000), moderate (2000 to 4000), Low (1000 to 2000), Very low (100 to 1000), and negligible (<100). Generally, high levels of penetrability relate to a decrease in pavement quality. RCP testing is not commonly conducted on pavements in Georgia.

3.5 Laboratory Testing Methods for Hot Mix Asphalt Pavements

3.5.1 Bulk Specific Gravity (Gmb), Theoretical Maximum Specific Gravity (Gmm)

Gmb is the ratio of a dry specimen's weight to the weight of an equal volume of water (PI, 2011). The test is performed according to AASHTO Standard T 166 – "Bulk Specific Gravity of Compacted Asphalt Mixtures using Saturated Surface-Dry Specimens" (AASHTO T 166, 2015). The test involves weighing the sample to record a dry weight. Next, the sample is immersed in water for 4 minutes and weighed underwater. Lastly, the sample is quickly dried and then measured for SSD weight. These parameters are then used to calculate the Gmb using the following formula:

$$\text{Bulk Specific Gravity (Gmb)} = \frac{A}{B - C} \tag{1}$$

Where A = Weight in grams of the specimen in air
B = Weight in grams, surface dry
C = Weight in grams, in water

Theoretical Maximum Specific Gravity (Gmm) is the specific gravity of a sample when there are no air voids. This is possible by testing the asphalt sample in "rice form" (AASHTO T209, 2015). Each sample is weighed to record a dry weight. Next, the material is placed inside the container and filled with water to a point of approximately 1 inch above the sample. Next, the container is sealed and a vacuum pressure of 25 to 30 mm Hg is applied for 15 minutes. Every 2 minutes, the container is agitated with a hammer to release any air bubbles. After the 15 minutes, the pressure is released and the sample is left undisturbed for 10 minutes. Afterwards, the sample is weighed underwater. These parameters are then used to calculate the Gmm using the following formula:

Theoretical Maximum Specific Gravity (Gmm)= $\frac{\text{A}}{\text{A+D-E}}$ (2)

Where A = mass of oven-dry sample in air
D = mass of container filled with water at 77°F
E = mass of container filled with sample and water at 77°F

The air content of as sample is calculated from by the Gmb and Gmm values using the following formula:

Air Voids (V_a) = $(1 - \frac{G_{mb}}{G_{mm}}) \times 100$ (3)

3.5.2 Hamburg Wheel Tracking Test

A Hamburg Wheel Tracking Test measures rutting and stripping in asphalt pavements by continuously rolling a steel wheel over the pavement surface (CDOT, 2014). During testing, the sample is submerged in 50°C water to evaluate moisture susceptibility. After testing, the rut depth of the sample is compared with the amount of wheel passes before failure (20,000 max). The result is used to determine the rate of pavement deformation, by approximating the stripping inflection point (SIP). The SIP is known as the point where "moisture damage starts to dominate performance" (FHWA, 2007). This value is formed by the intersection of the creep slope and the stripping slope. The creep slope refers to the slope of the graph before SIP, whereas the stripping slope is the slope after SIP has occurred (Izzo, 1999). A Hamburg Wheel Tracking Test machine can be viewed in Figure 5.

Figure 5 - Hamburg Wheel Tracking Machine at UGA

3.5.3 Binder Content

To determine the binder content of an asphalt specimen, the sample is heated to a high temperature (538^{o}C) using an ignition furnace, where all binder will burn away. The difference between the starting and ending weights is used to determine the binder content. Before ignition, samples are heated to a temperature of 230 ± 9 ° F for a minimum time 25 minutes. To determine the binder content of asphalt for this study, an NCAT Asphalt Content Furnace was used to conduct binder content tests in accordance with the AASHTO T 308 “Determining the Asphalt Binder Content of Hot Mix Asphalt (HMA) by the Ignition Method”. The furnace has an internal scale that automatically calculates binder content as the sample is burning. After the test is finished, the sample is removed and cooled. Proper safety precautions were strictly enforced throughout this entire process.

4. JOINTED PLAIN CONCRETE PAVEMENT

4.1 Introduction

Jointed Plain Concrete Pavement (JPCP) is characterized by its concrete slabs that contain steel dowels to efficiently transfer load from traffic. However, transportation agencies find that some JPCPs stay in good condition over time, while others have deteriorated significantly. According to the NCHRP 747 guidelines, JPCP is known to be susceptible to distresses such as transverse cracking, joint faulting, and spalling (Rada, 2013).

In this study, two JPCP sites have been selected in consultation with GDOT as shown in Figure 6: SR-22 in 'good' condition and I-75 in 'poor' condition. The field investigations were performed at the JPCP sites in two phases: non-destructive and destructive investigations. The non-destructive site investigation involved a visual inspection, Ground Penetration Radar (GPR) scanning and Falling Weight Deflectometer (FWD) testing. Destructive field testing involved collecting pavement cores from the sites and conducting laboratory tests on the cored specimens.

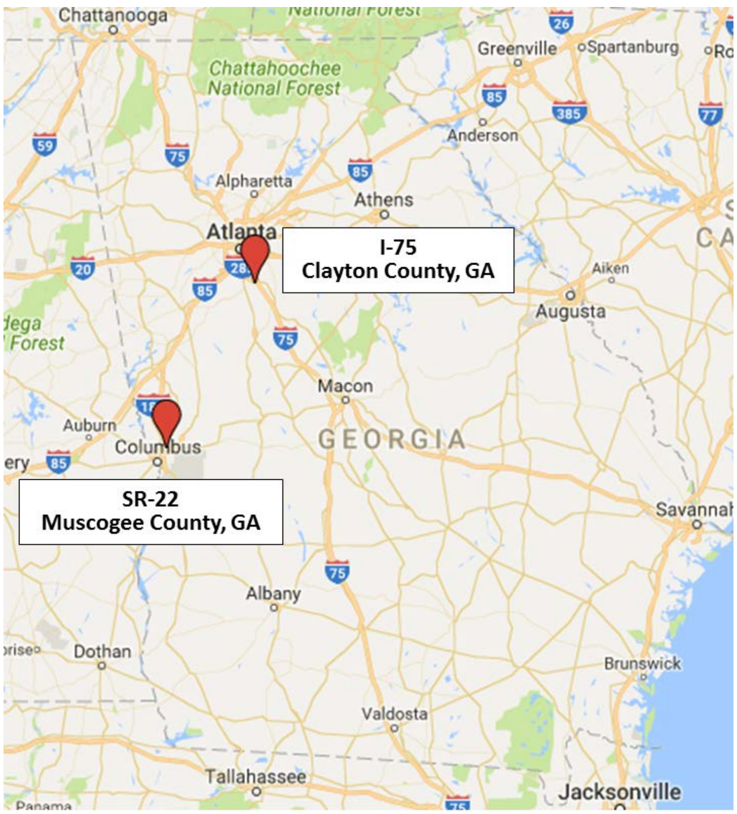


Figure 6 – JPCP Site Locations

4.2 Test Site and Field Setup

A mile-long section from each site was investigated. The first test site is located on State Route (SR) 22 (Westbound) in Muscogee County, Georgia. The road from milepost 8 to 7 is reported to be in a 'good' condition, which shows no visible deficiencies. The second site is located on Interstate (I) 75 (Southbound) in Clayton County, Georgia. The road from milepost 226 to 228 is in 'poor' condition, with multiple deficiencies observed on its surface. A visual comparison of both sites is shown in Figure 7. Table 6 shows a comparison of site conditions and pavement profile/construction parameters in the two JPCP sections. In Georgia, roadway sections are rated by district offices and are given a PACES rating (taken in 2015). Ratings of 70 or below generally warrant rehabilitations. The JPCP sections, SR-22 and I-75, have a PACES rating of 100 and 40, respectively, as summarized in Table 6. Deductions from I-75 are from Linear Cracking, Ruptured Slabs, and Joint Spalling.

SR-22 was constructed in 2008 with a design speed of 60 miles per hour. The section is composed of 9 inches of Portland Cement Concrete (PCC) and 8 inches of Graded Aggregate Base (GAB) (see Table 6). SR-22 is composed of two lanes in one direction. The Average Annual Daily Traffic (AADT) in 2013 for the site is 26,630 vehicles with 985 trucks (3.7%). The slab has skewed joints. The test section between MP 8 to MP 7 was selected because it shows relatively good concrete pavement condition in both the fast and slow lanes.

I-75 is believed to have been constructed in 1968 with the earliest documented rehabilitation occurring in 1989. This section is composed of an existing road that was widened from 2 lanes to 3 lanes in one direction with a design speed of 55 miles per hour. The outside lane (lane #3) is designed to have 10.5 inches of Portland Cement Concrete (PCC) over a layer of 6.5 inches of unbonded concrete with GAB underneath. The inside lane (lane #1) is composed of 10 inches of PCC over an Asphalt Concrete (AC) layer of 4 inches with GAB underneath. The inside lane consists of a different concrete composition than the outside lane. The AADT for the site is 115,000 vehicles with about 8,050 trucks (7%). During visual inspection, the inside lane was observed to have fewer signs of failure. The inside lane also experienced a lower volume of trucks.

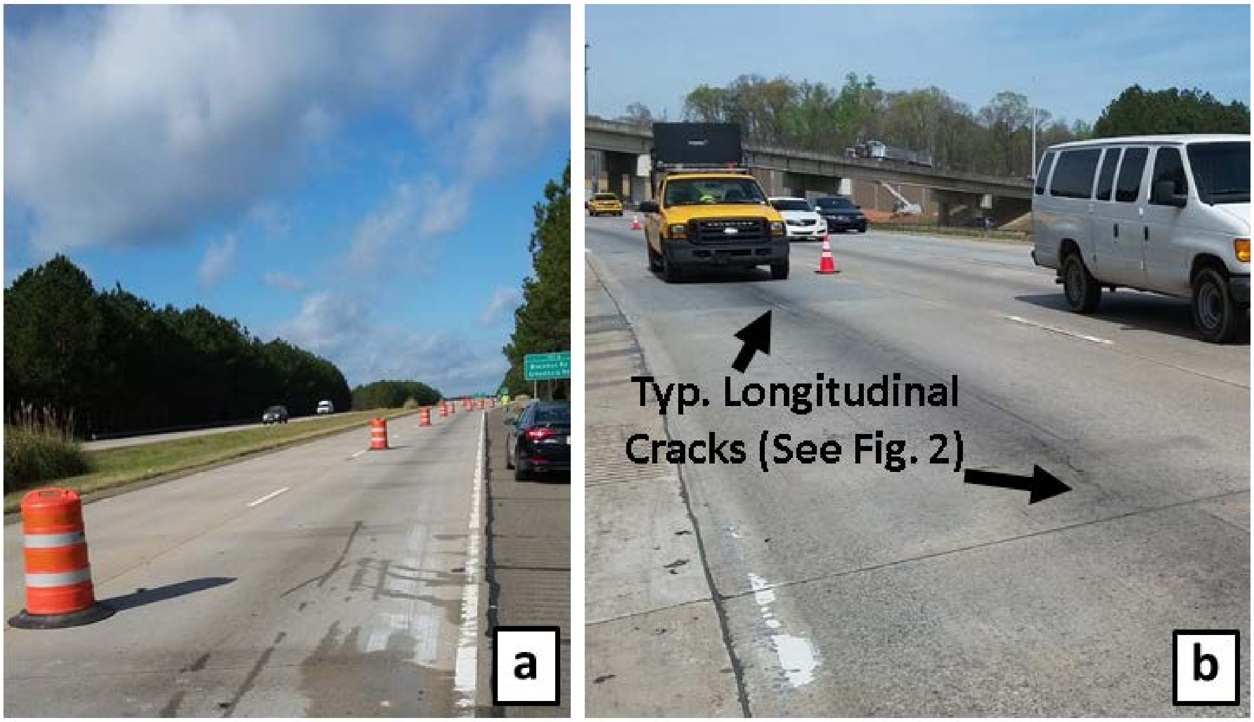


Figure 7 - General Site Conditions: (a) SR 22 (good condition); (b) I-75 (poor condition)

Table 6 - JPCP Conditions.

	Parameters	SR 22		I-75	
		Outside	Inside	Outside	Inside
Condition & Profile	Condition	Good		Poor	Fair
	Current Condition Rating (PACES) (2014-2015)	100		40	
	Visual Distress Observed	None		Primary Distress: Longitudinal Cracking Secondary Distress: Transverse Cracking, Punchouts, Joint Spalling, and Shattered Slabs	
	Age (years)	48 (1968)		26 (1990)	
	Pavement Structure (in.)	9” PCC/ 8”GAB	9” PCC/ 8”GAB	10.5”PCC/6.5”PCC/10”GAB	10”PCC/4”AC/10”GAB
	AADT (% Trucks) (taken in2013)	26,630 (3.7% trucks)		115,000 (7% trucks)	

In Georgia, roadway sections are rated by district offices and are given a PACES rating (taken in 2015). Ratings of 70 or below generally warrant rehabilitations. The JPCP sections, SR-22 and I-75, have a PACES rating of 100 and 40, respectively, as summarized in Table 6. Deductions for I-75 are from Linear Cracking, Ruptured Slabs, and Joint Spalling. During the visual inspection of I-75, many deficiencies were noted of which spalling, transverse cracks, and longitudinal cracks were most common (and can be seen in Figure 8). In the outside lane, longitudinal cracks running parallel to the wheel paths are the most frequently observed distress type. Small aggregate delamination was occasionally observed, as well as spalling.

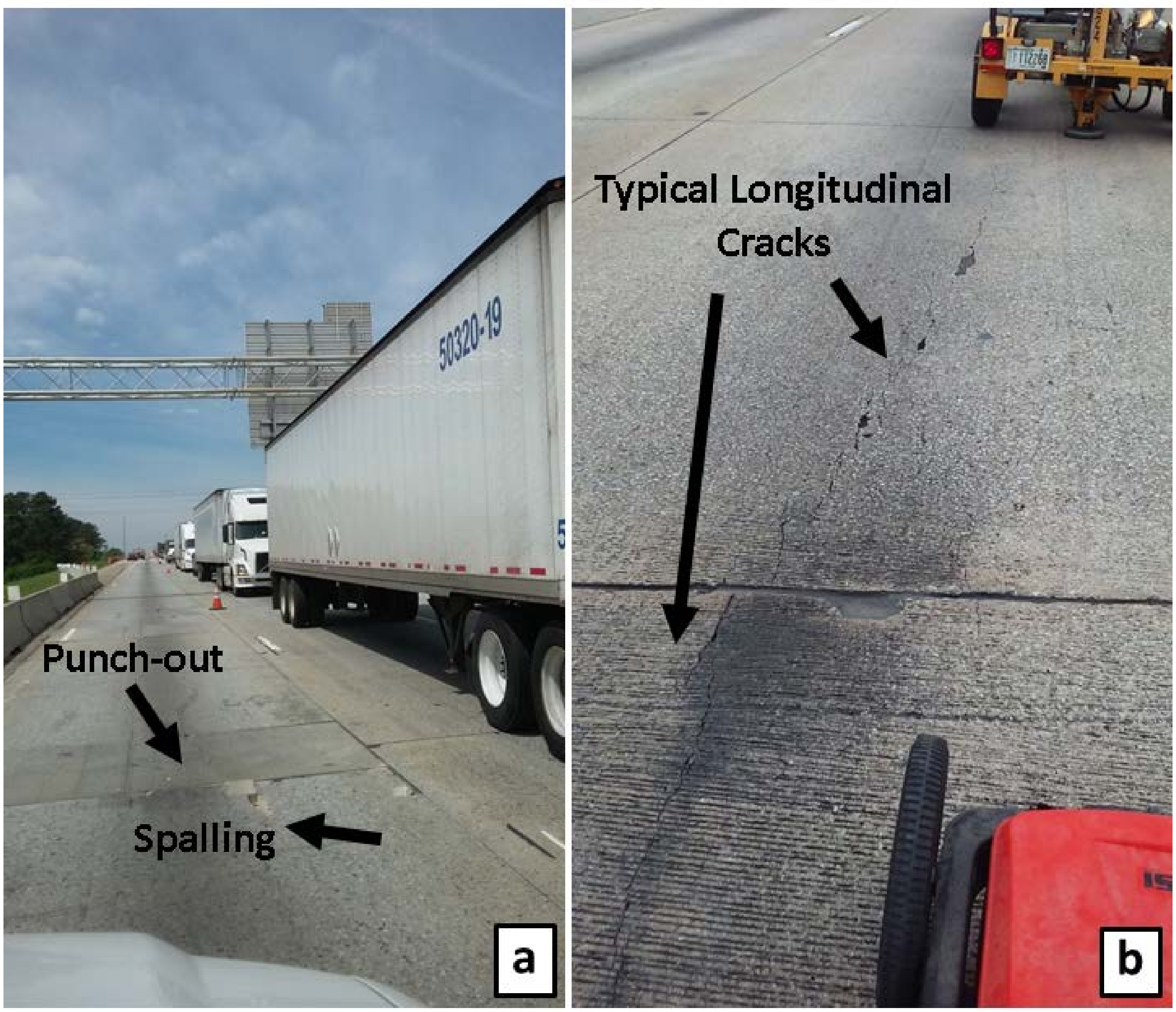


Figure 8 - I-75 Typical Distress (Poor Condition)

(a) Spalling Repairs and Longitudinal Cracking; (b) Longitudinal Cracking through joints.

4.3 Non-Destructive Testing

As described in previous section, nondestructive testing (NDT) methods, such as FWD and GPR, are widely used to evaluate in-situ material characteristics of in-service pavements. More information on these technologies can be found in Section 3.2 - Review of Pavement Forensic Technologies – Non-destructive.

Table 7 shows a summary of pavement structure determined from the GPR scans. This Table also includes saw cut depth, clear cover depth, rebar size, dowel spacing, and slab aspect ratio. The GPR results for both SR-22 and I-75 are fairly consistent, with a pavement thickness that is representative of their design data (Figure 9). Both sites have consistent compaction, as shown by their consistent layers in the GPR scan. The dowel bars are shown towards the center of the graph. It should be noted that dowels in the outside lane of the I-75 were placed slightly below the centerline.

Table 7 - JPCP NDT Results and Design Parameters.

	Parameters	**SR 22 (Good)**		**I-75 (Poor)**	
		Outside	Inside	Outside	Inside
FWD	Joint Efficiency (%)	92	90	92	85
	Average ISM (kip/in)	2000	2000	2500	3000
	Back-calculated subgrade reaction (pci)	105	115	138	162
JPCP Design Parameters	Surface Texture	Transverse Tining		Transverse Tining (Worn)	Transverse Tining (Worn)
	Saw Cut Depth (in.)	2.5		2	2
	Dowel Depth (Clear Cover) (in.)	4.25 (5 from core)		6.5	3
	Actual Dowel Diameter	1.125" (#9)		1.25" (#10)	1.25" (#10)
	Epoxy Coated Rebar	Yes		Yes	Yes
	Dowel Spacing (in. on center)	12		12	12
	Dowel Length (in.)	18		18	18
	Joint Spacing (ft)	20		15	15
	Slab Dimensions - Length by Width (ft)	20 by 12		15 by 12	15 by 12
	Slab Aspect (L/W) Ratio	1.67		1.25	1.25
	Slab Length-to-Thickness Ratio	26		17	18

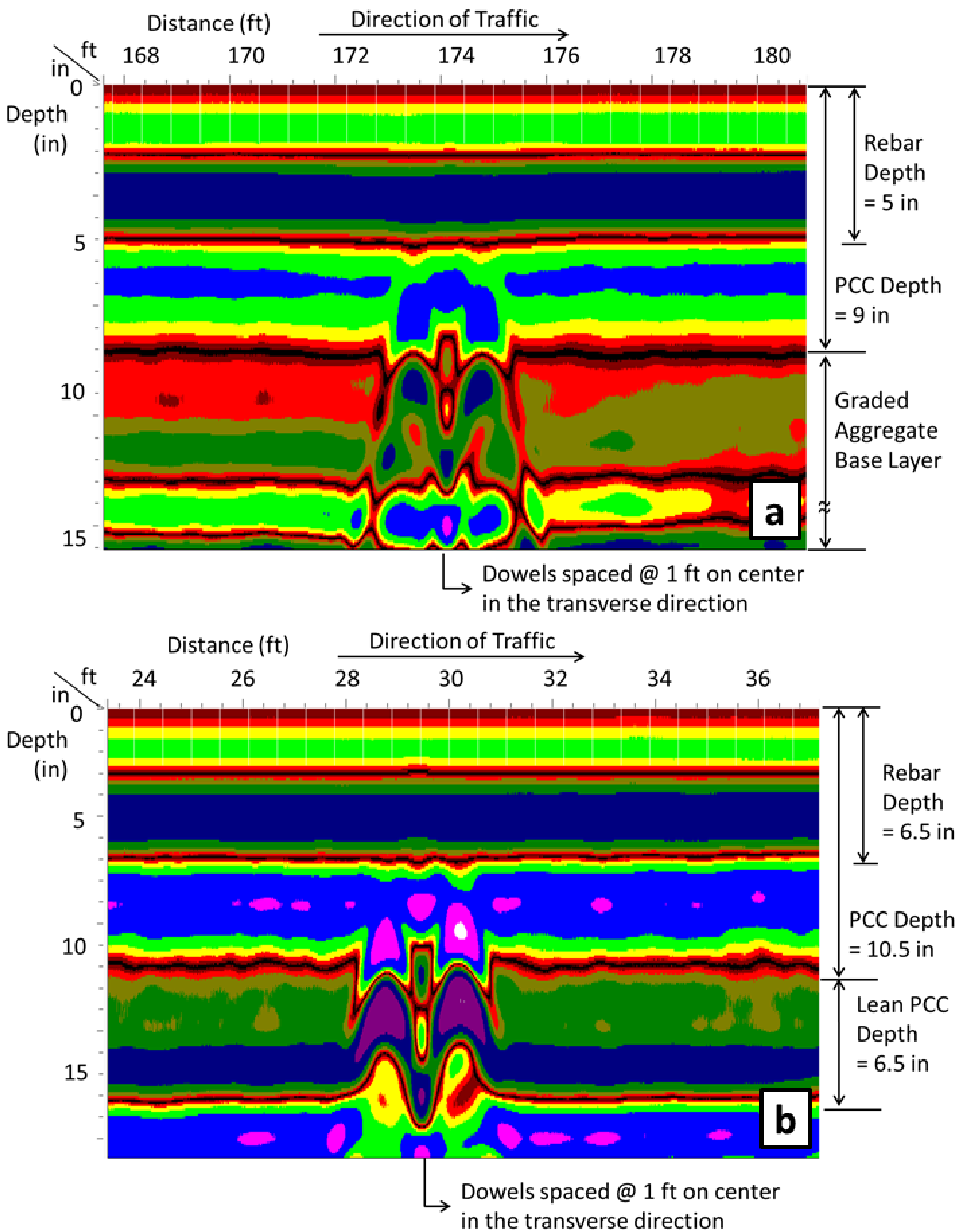
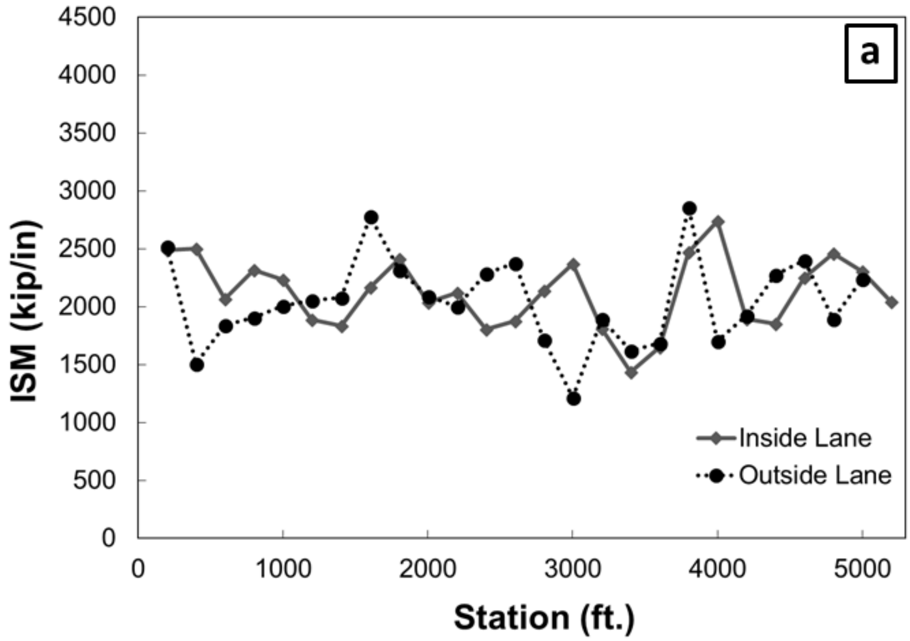


Figure 9 - Typical GPR scans showing single joint
(a) JPCP scan of SR 22 (good condition); (b) JPCP scan of I-75 (poor condition) in the direction of traffic.

The ISM plots created from the FWD testing are shown in Figure 10. Compared to the ISM plot in SR-22, the ISM plot from I-75 shows a certain degree of variation along with distance, that might be interpreted as a construction variability. Using the FWD data, a modulus of subgrade reaction, *k* was back-calculated based on the 1993 AASHTO design guide (AASHTO, 1993) and is summarized in Table 7.



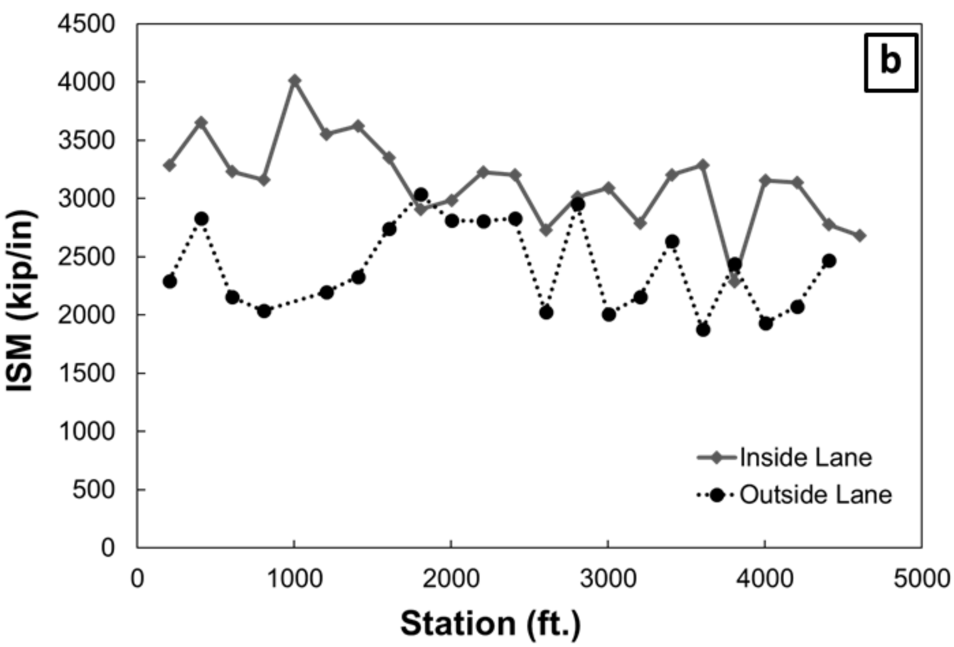


Figure 10 - Selected ISM Plots determined from FWD tests
(a) SR-22 (good condition); (b) I-75 (poor condition)

4.4 Destructive Testing – Coring and Field Testing

Typical cores and crack depths at joint locations, as well as photos of cores retrieved from joint locations are shown in Figure 11. Figures 12 and 13 illustrate coring locations for the two JPCP sections. Pavement cores were extracted in order to confirm the existing pavement thickness, dowel size, joint design, and crack depth. Based on the recommendations in the NCHRP 747 report, the cores were taken on the centerline of the slab, wheel paths in slow and fast lanes, and cracks to document the crack depth. Further, the non-destructive test data and visual inspection were reviewed to determine the coring locations for both sites as shown in Figures 12 and 13. For I-75, a 4-inch core drill was used for laboratory tests, although a 6-inch core bit was utilized to observe dowel locations. For SR-22, a 4-inch core drill was used for all extractions. Photos of all cores extracted are shown in Figure 14. As shown in Figure 14, cores show consistent compaction and few voids.

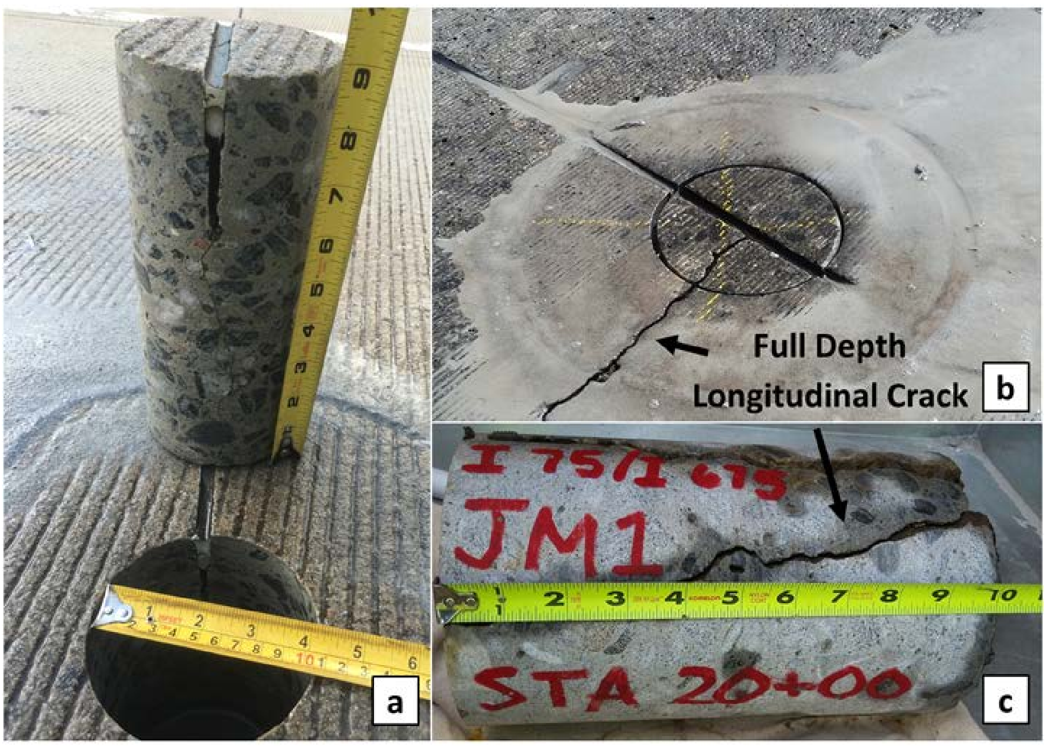


Figure 11 - Typical Cores at Joints

(a) SR 22 core at the joint (good condition); (b) I-75 core (poor condition); (c) I-75 core (poor condition) showing a full-depth longitudinal crack.

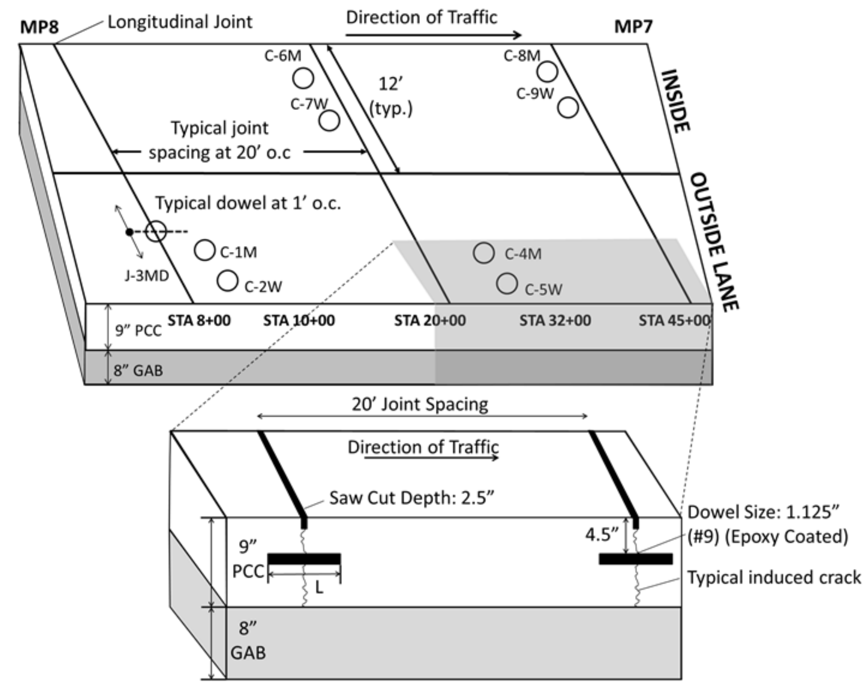


Figure 12- 3D View of Coring Locations and JPCP details for SR 22 (good condition).

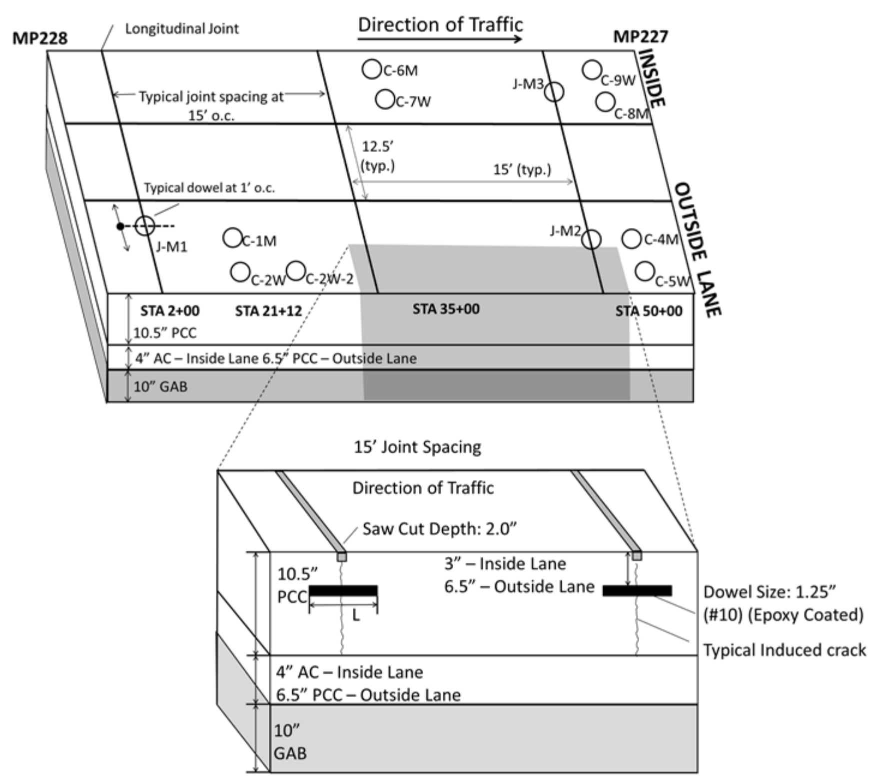


Figure 13- 3D View of Coring Locations and JPCP details for I-75 (poor condition).

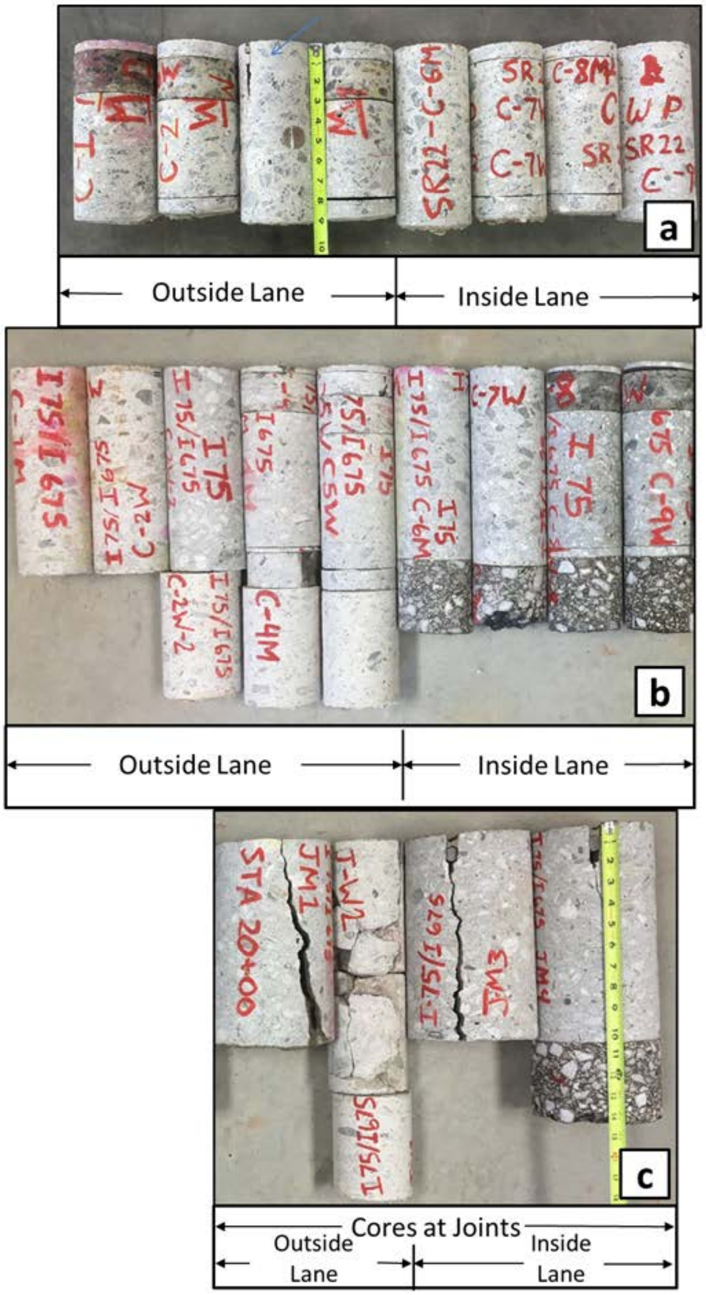


Figure 14 - All cores retrieved from JPCP sections

(a) SR 22 cores (good condition); (b) I-75 cores (poor condition) (c) I-75 cores at joints (poor condition). Note: Core samples shown in (a) and (b) are re-assembled after conducting the laboratory tests for this Figure.

4.4.1 SR-22 Section Coring and On-site Testing

The core (J-3MD) was extracted from a joint at the SR 22 location and tested for carbonation using a chemical testing kit. The test result showed a negative reaction, which means no carbonation was observed. The saw cut on the sample measured 2.5 inches (See Figure 14 (a)). The remaining cores maintained a consistent measurement of 9 inch PCC with GAB underneath. The dowel began at a clear-cover depth of 5 inches below the pavement surface. In this section, the asphalt concrete shoulders were sealed to prevent moisture from entering.

4.4.2 I-75 Section Coring and On-site Testing

At the I-75 site, the inside lane showed fewer signs of joint failure, as opposed to the outside lane which showed many deficiencies. The two lanes also show a visible difference in mix design, which strongly indicates that they were constructed at different times. As indicated in Figure 14 (c), the saw cut on the sample measured 2 inches. The second core sample taken in the middle of the slow lane (C-1M) was tested for an Alkali-Silica Reaction (ASR). There was a bright yellow reaction around aggregates as shown in Figure 15 (a), and thus on site testing revealed that there was evidence of ASR. This core sample was selected for a petrographic analysis. A core sample (C-2W-2) was taken from a rehabilitation patch and 6.5 inches of existing PCC was discovered underneath the new PCC. Sample J-W2 was tested for carbonation, which indicated positive on the surface of cracks. As shown in Figure 14 (b), cores taken from the fast lane were discovered to have approximately 4 inches of an asphalt concrete layer beneath the PCC layer. The reinforcement began at a clear-cover depth of approximately 3.5 in. below the pavement surface. Small particle delamination was observed on the site. Sample J-M4 contained a dowel and was subsequently tested for carbonation, which was not found and epoxy coated dowels were in good condition.

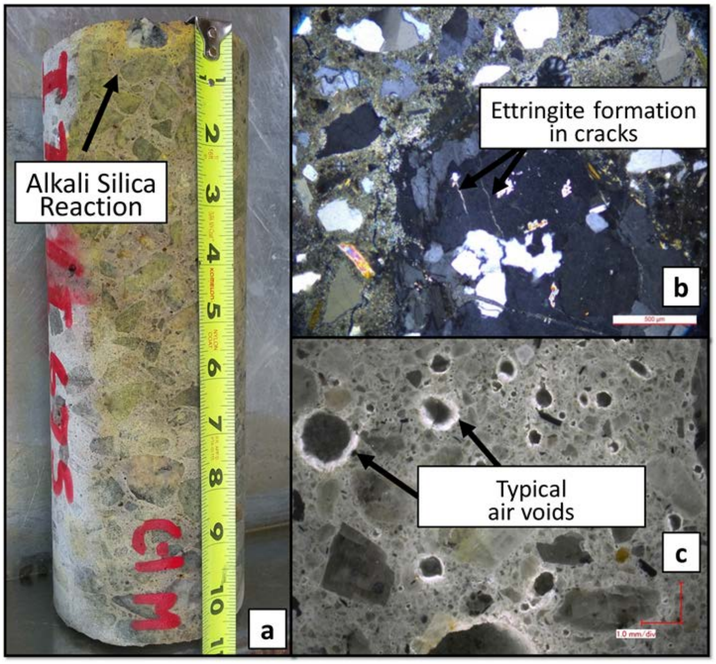


Figure 15- ASR damage found in I-75 section (poor JPCP)

(a) On-site field test; (b) Photomicrograph of ASR gel and a crack filled with ettringite in thin section; (c) Photomicrograph of the typical air void structure from a core.

4.5 Destructive Testing – Laboratory Testing

A summary of laboratory test results is described in this section. More information on these specific technologies can be found in Section 3.4 – Laboratory Testing Methods. The CTE tests were conducted using cored specimens in accordance with AASHTO T 336. The CTE of Portland cement concrete (PCC) generally ranges between 4.4 and 5.5 microstrains/°F (AASHTO, 2011). The measured CTEs are within an acceptable range for pavement design in Georgia (Kim, 2012). The measured CTE values for the two test sections are shown in Table 8.

The RCP tests were performed according to ASTM C1202-12: Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration (ASTM, 2012). As shown in Table 8, the RCP values in the SR-22 and I-75 were determined to be low and high, respectively.

The Modulus of Elasticity (MOE) measures the elastic relationship of stress to strain for a given material. More specifically, it measures material stiffness. The MOE tests for the JPCP specimens were conducted in accordance with the ASTM C469: Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression (ASTM, 2003). The MOE for I-75, particularly for the slow lane, was relatively low, although the average compressive strength was comparable to values determined from SR-22, as shown in Table 8. Both I-75 and SR-22 meet the requirements for minimum compressive strength.

4.6 Petrographic Examination

Two cores (one from each site) were selected for petrographic analysis. The petrographic analysis of selected cores (C9W from SR-22 and CM-1 from I-75) was performed by TEC Services, Inc. located in Lawrenceville, Georgia. This test involves taking an in-depth examination of selected cores to determine multiple construction and material parameters that are not available otherwise. The analysis of the sample from SR-22 indicated that the material makeup of the section consisted of a nominal maximum aggregate size (NMAS) of 3/4 inch crushed granite as coarse aggregate and natural quartz for fine aggregate. The water-to-cement ratio ranged between 0.45 and 0.49, and no fly ash/slag was included. The concrete is air-entrained, resulting in a 3%-6% air content. In reference to quality acceptance criteria from GDOT, the water-to-cement ratio is acceptable. The design air content range is between 4.0 to 5.5, so the concrete from SR-22 may meet the design requirements (GDOT 430.3.06, 2013).

The analysis of the sample from I-75 revealed that the material makeup of the section consisted of a NMAS of 5/8 inch crushed granite as coarse aggregate. Manufactured sand was used for fine aggregate. The paste was identified as good quality with a water-to-cement ratio of 0.4 and contained no slag or fly ash. The paste was described as well-hydrated, but mottled and unevenly distributed, which can be attributed to poor construction practices. An air-entraining agent was not found, and a low air content (2%) was found. The air was entrapped as shown in Fig. 15(c), which is indicative of a poor quality mixture. In reference to quality acceptance criteria from GDOT, the water-to-cement ratio is acceptable. The design air content range is between 4.0 to 5.5, so the concrete from I-75 does not meet the design requirements (GDOT 430.3.06, 2013).

Due to the presence of ASR, the concrete used in I-75 is unlikely to be durable for freeze-thaw cycles. Several micro-cracks were visible, and few contained alkali-silica reactivity (ASR) gel. Any deterioration of concrete by ASR or freeze-thaw action accelerates the rate at which ettringite leaves its original location and recrystallize in larger spaces such as voids or cracks (Suksawang, 2014). Although little ASR gel was found in the core sample from I-75, the ettringite formation at voids/cracks is indicative of ASR. This was identified as a major concern, as many of the cracks traveled through the aggregate as shown in Fig. 15(b). These micro-cracks and ettringite formation may be associated with heat of hydration damage during concrete placement or temperature-gradient related damage (PCA, 2001).

Table 8 - Summary of Core Test Results.

	Parameters	SR 22 (Good)		I-75 (Poor)	
		Outside	Inside	Outside	Inside
Condition	Good/Fair/Poor	Good	Good	Poor	Fair
On-site Field Testing	ASR	No	No	Low/Moderate	Low
	Carbonation	No	No	No*	No
Laboratory Testing	MOE (ksi)	4189	N/A	3755	4712
	f'c (psi)	9,500	7,700	7,600	5,600
	RCP (Coulomb)	2,845	2865	5997	6328
	CTE (in/in/°F)	5.10	5.10	4.65	4.47
Petrographic Analysis	Coarse Aggregate	Crushed Granite		Crushed Granite	
	Maximum Aggregate Size	3/4" The aggregate is well distributed and well graded.		5/8"; Well gradation; no segregation	
	Fine Aggregate	Natural quartz; The maximum sand particle size is 1/8"		Manufactured Sand	
	W/C ratio	0.45-0.49		0.4	
	Fly ash	No fly ash or slag		No fly ash or slag	
	Paste	The paste is of good quality.		The paste is of good quality.	
	Air entrained	Air entrained		No Air entrained	
	Air content	The air varies from 3-6% and is not evenly distributed. The majority of the air is of good quality.		Approximately 2% and many void are entrapped. The air is of poor quality.	
	Cracks	There are occasional cracks in the aggregate that do not appear to be significant.		Many microcracks visible; These cracks contain ASR gel and ettringite.	
	Other distresses to note	Concrete is well hydrated and the paste is hard.		Microcracks visible in thin section, often filled with ettringite (see Fig. 15(b)). Concrete is unlikely to resist freeze thaw cycles in a saturated condition.	

Notes: * Carbonation was discovered within the crack surfaces of the concrete sample.

4.7 Analysis of Testing Results

A low/medium level of ASR, as noted in Table 8, was observed at the surface of I-75. Neither site experienced carbonation, although surfaces of cracks exposed to air were carbonated.

The MOE results were unexpectedly lower in I-75 compared to SR-22. The RCP test results show that values for I-75 ranged from 4100 to 7600, with an average of 6000. This indicates very high chloride ion penetrability. The values changed sporadically throughout the section. In contrast, values for SR-22 ranged from 1800 to 3600, with an average value of approximately 3000 throughout the section. This indicates low or moderate chloride ion penetrability. The compressive strength for both sections ranged between 5600 and 7600 psi for I-75, whereas it ranged between 7700 and 9500 psi for SR-22. Relative to SR-22, I-75 has a thicker concrete slab with comparable compressive strength. However, it is concluded from the field and laboratory test results that the concrete in I-75 is depicted by poor material composition including microcracks, ettringite, poor air-entrainment, and ASR damage.

The NCHRP 747 guide prescribes possible causes of longitudinal and corner cracking, similar to the distress observed in I-75. Longitudinal cracking may be caused by low PCC strength, high CTE, thermal deformation due to warping and curling, and poor load transfer to tied shoulder. The causes of failure are fairly consistent with the Caltrans' JPCP design guide (Caltrans, 2008) in that longitudinal cracks occur parallel to the centerline of the pavement and are often caused by a combination of heavy load repetitions on pavement with unsatisfactory roadbed support, thermal curling, faulting, shrinkage, and moisture induced warping stress. Furthermore, the linear cracks running along the centerline of panels (see Figure 8) can develop due to a range of factors (Pavement Interactive, 2012). These include overloading, thermal expansion and contraction, moisture stresses, slab curling, and loss of support underneath the slab. When a combination of distress factors are involved, traffic loads are known to exacerbate these problems.

In comparison to each other, SR-22 and I-75 have very different amounts of traffic. The AADT (2013) for SR-22 is 26,630 vehicles with 985 trucks (3.7%). SR-22 has two lanes in each direction. The AADT (2013) for I-75 is 115,000 vehicles with about 8,050 trucks (7%). I-75 has 3 lanes in one direction. Even when lane distribution is taken into account, I-75 experiences a much higher amount of traffic. During visual inspection, the outside lane also experienced a higher volume of trucks.

The field and laboratory tests indicated that the concrete in I-75 is not likely to be durable, despite a reasonable compressive strength. Therefore, in addition to the distresses found in the concrete materials, another distress factor is suspected when non-destructive and destructive test results indicated no major deficiency in the concrete performance (strength and stiffness) of the sections. It can be concluded from the previous traffic analysis that the distress may be attributed to increased AADT. There are national guidelines available to evaluate JPCP design options such as the AASHTO 1993 design guide (AASHTO, 1993) and Pavement ME (Mu, 2015; ARA, 2004; Pierce, 2014). However, it was not possible to consider a combination of factors, such as thermal deformation and traffic loads, while providing a diagnosis of the full-depth longitudinal cracks found in I-75. Therefore, a nonlinear finite element analysis (FEA) model was constructed to simulate the cracking mechanism observed in I-75, which is discussed in the following section.

4.8 Finite Element Analysis of the Distress

A finite element analysis (FEA) model was constructed using the ANSYS v18.2 software with the objective of simulating the distress conditions found in I-75. In this analysis, the modulus of subgrade reaction determined from the FWD test was used for simplicity by providing compression-only springs at the bottom face of the JPCP section. It is intended that a single slab of I-75 be analyzed to diagnose the causes of pavement distress (or longitudinal cracks) in a three-dimensional FEA model using the best engineering judgment.

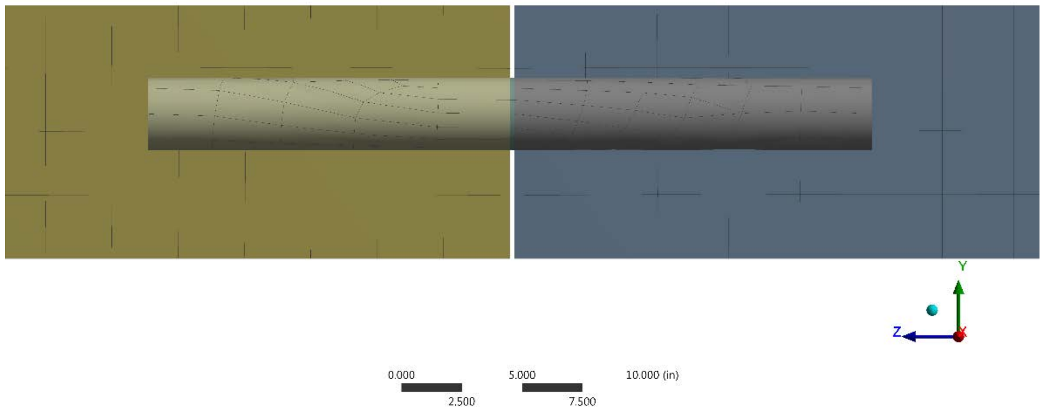
A typical three-lane JPCP slab with a uniform lane width of 12 feet was considered for modeling, and one lane was modelled for analysis as shown in Figure 16 (a). A joint spacing of 15 feet was selected for this study as shown in Table 7. For the purpose of illustrating the cause of distress in a clear manner, the entire concrete panel was analyzed, despite the axis of symmetry in a single panel. Furthermore, the two adjacent panels in the direction of travel are modeled half way between joints using the centerline as an axis of symmetry in order to accurately evaluate the behavior of joints. Joint dowels and concrete panels are modeled with solid elements, and dowels are assumed perfectly bonded to the one side of two adjoining concrete slabs. The joint model is illustrated in Figure 16(a) and includes a small gap between the two concrete slabs. A single wheel load of 9,000 lb. of an equivalent 18,000 lb. single axle load was considered in this study. An uniform contact pressure of 100 psi was applied over a rectangular area, as illustrated in Figure 16(b). The 100 psi pressure was applied to reflect the minimum cold pressure for a 9000 lb single wheel load.

Curling occurs in the form of a three-dimensional deformation which provides a positive curvature in two directions. A positive curvature from a temperature gradient mainly occurs in the direction of travel. Generally, in the stress analysis of JPCP, traffic load is applied at the mid-span to simulate transverse cracks for positive curling in the longitudinal direction. However, in a relatively square concrete slab, a positive curvature could also become noticeable in the transverse direction due to thermal restraints provided by the adjacent lane and shoulder. Therefore, the critical traffic loading was placed close to the joint locations as shown in Figure 16 (b).

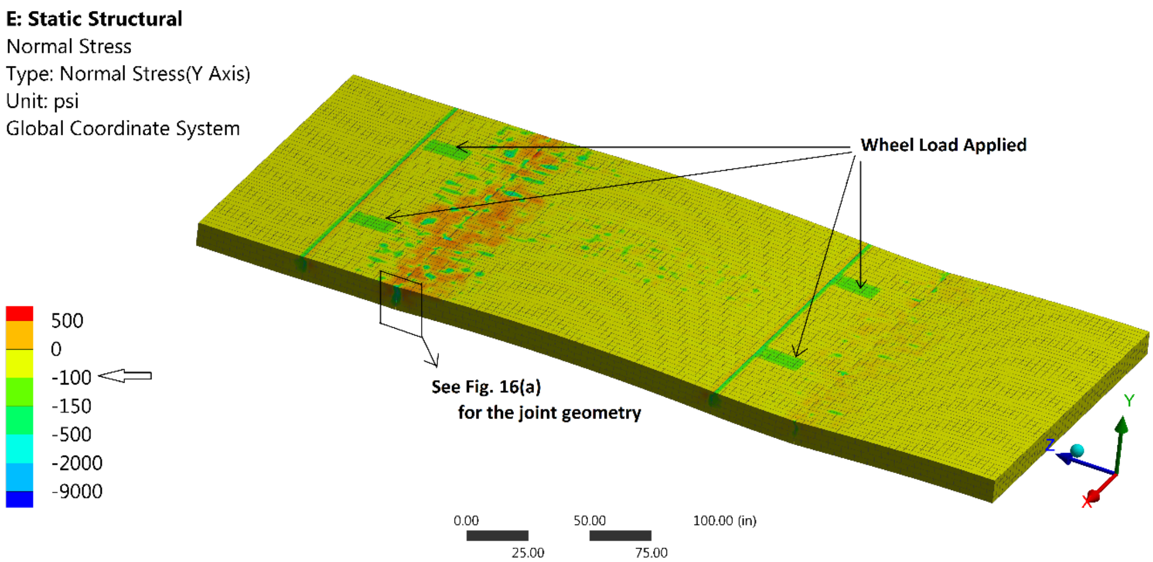
The uniform temperature of 70ºF was applied to the FEA model, with a gradient temperature of 30ºF through the slab thickness, to account for convection and solar radiation representing a summer-weather condition. The top surface of the slab was simulated warmer (100ºF) than the bottom of the slab (70ºF). The structural model reads the temperature profiles determined from the thermal analysis, and a structural analysis is performed to evaluate stress and strain solutions. Figure 16 (c) shows a schematic diagram of principal strains in the concrete slab for combined thermal and wheel loads (18kip single axle load). Thermal stress relieved by concrete cracking was not considered in this study.

In this analysis, it is determined that the concrete slab develops a tensile crack when the principal elastic tensile strain exceeds 0.00012 in/in because concrete generally cracks when the tensile strain exceeds 0.010 to 0.012 percent. Curling of the JPCP slab due to a daytime positive temperature difference combined with a critical traffic loading position results in high tensile strain (greater than 0.00012 in/in) at joints and initiates a crack in the direction of travel parallel to the centerline of the joint (Figure 16(d)).

Based on the FEA analysis, it is ascertained that thermal deformation combined with structural deformation from the wheel loads causes longitudinal cracks at joints parallel to the centerline of the concrete slabs. The analysis did not account for the microcracks developed during the concrete placement, minor ASR damage in the concrete material, or an increased cover depth during construction found in the I-75 site investigations. The extent of cracking may have been exacerbated if these effects were taken into consideration in the material model. Many of these problems are not common in Georgia pavements, therefore these conclusions apply only to I-75. A full report of conclusions and recommendations can be viewed in Section 8 – Conclusions.



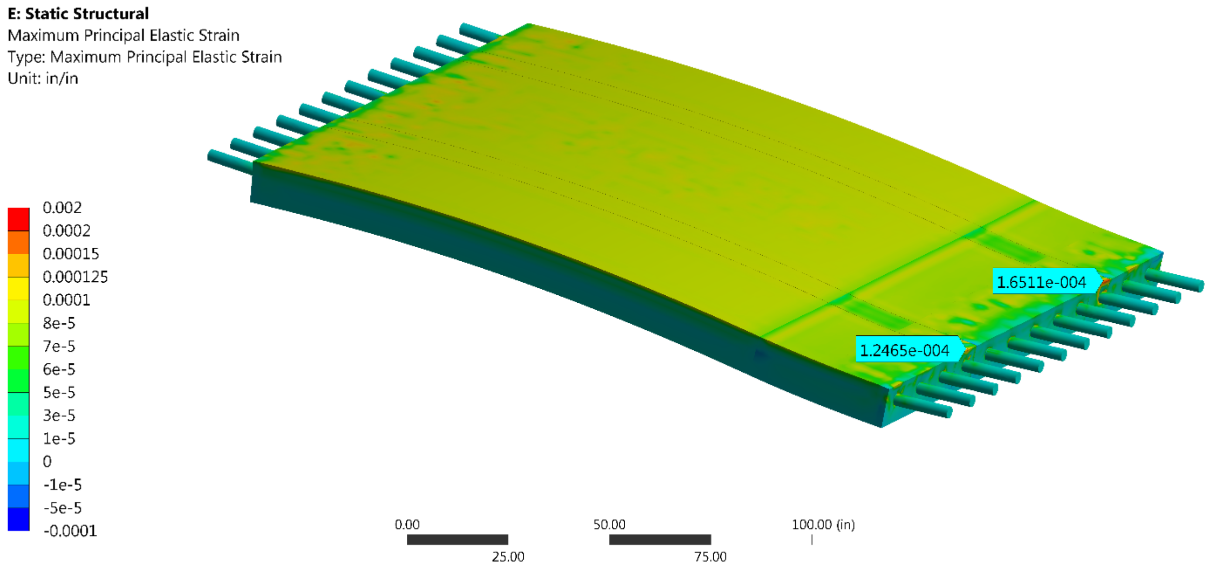
(a) Typical Joint Model



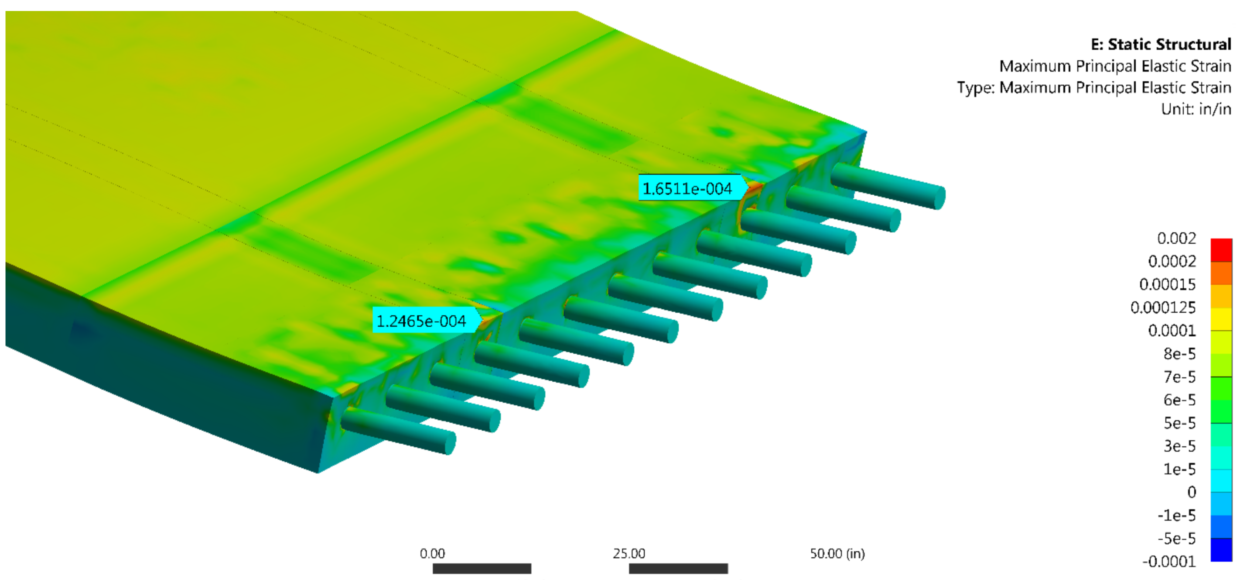
(b) Isometric view of the FEA analysis model under traffic loading

Figure 16 – Applied pressure and FEA strain results.

(note: The deformation is magnified by a factor of 500).



(c) Principal strain plot of the middle slab under combined traffic and thermal loading



(d) Enlarged joint view

Figure 17 Continued – Applied pressure and FEA strain results

(note: The element mesh is removed from the view for clarity and deformation magnified by a factor of 500).

5. CONTINUOUSLY REINFORCED CONCRETE PAVEMENT

5.1 Introduction

Continuously Reinforced Concrete Pavement (CRCP) consists of a concrete slab reinforced throughout its entire length by longitudinal reinforcement. The continuous steel reinforcement eliminates the need for contraction joints, while efficiently distributing load. CRC is susceptible to issues such as longitudinal cracking, which can induce punchouts in the pavement.

Two trends have been observed between multiple CRC pavements. Over time, many CRC roads stay in fair condition, while others deteriorate quickly. To study this phenomenon, a forensic investigation has been conducted to find the underlying causes of fair and poor pavement performance. As a guideline, the National Cooperative Highway Research Program (NCHRP) Report 747- Guide for Conducting Forensic Investigations of Highway Pavements document is utilized to understand the performance of two CRC sites.

Interstate 85 stretches across the southeast region of the United States. Two pavement sections from this interstate have been investigated as shown in Figure 17. At a distance of 10 miles apart, one pavement site exhibits fair pavement performance, while the other pavement shows poor performance. A forensic site investigation has been performed at each site, including nondestructive, destructive, and laboratory testing.

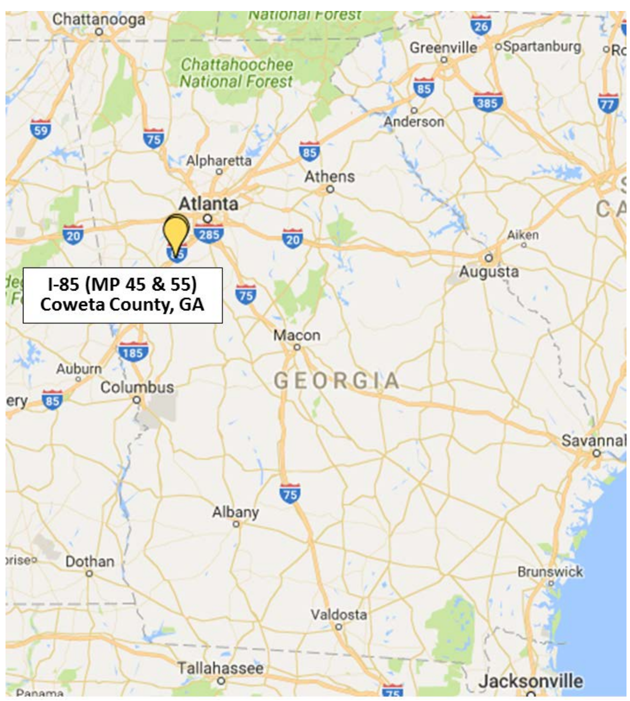


Figure 18- CRCP Site Locations

5.2 Test Site and Field Setup

The two sections used for this study are part of Interstate (I) 85 which runs through Coweta County, Georgia. The site that exhibits fair pavement performance is located between mileposts 45-44. The site showing poor performance is located between mileposts 55-54. A visual comparison of both sites is shown in Figure 18. Table 9 includes a comparison of site conditions and pavement profile/construction parameters in the two CRC sections. Currently, the PACES rating for CRCP is calculated based on the JPCP distress types (i.e., faulting), which is invalid for CRCP condition evaluations. Therefore, the CRCP PACES rating was not taken into the consideration for the site investigations. For the remainder of the CRCP section, I-85 milepost 45-44 will be referred to as MP 45 and I-85 milepost 55-54 will be referred to as MP 55.

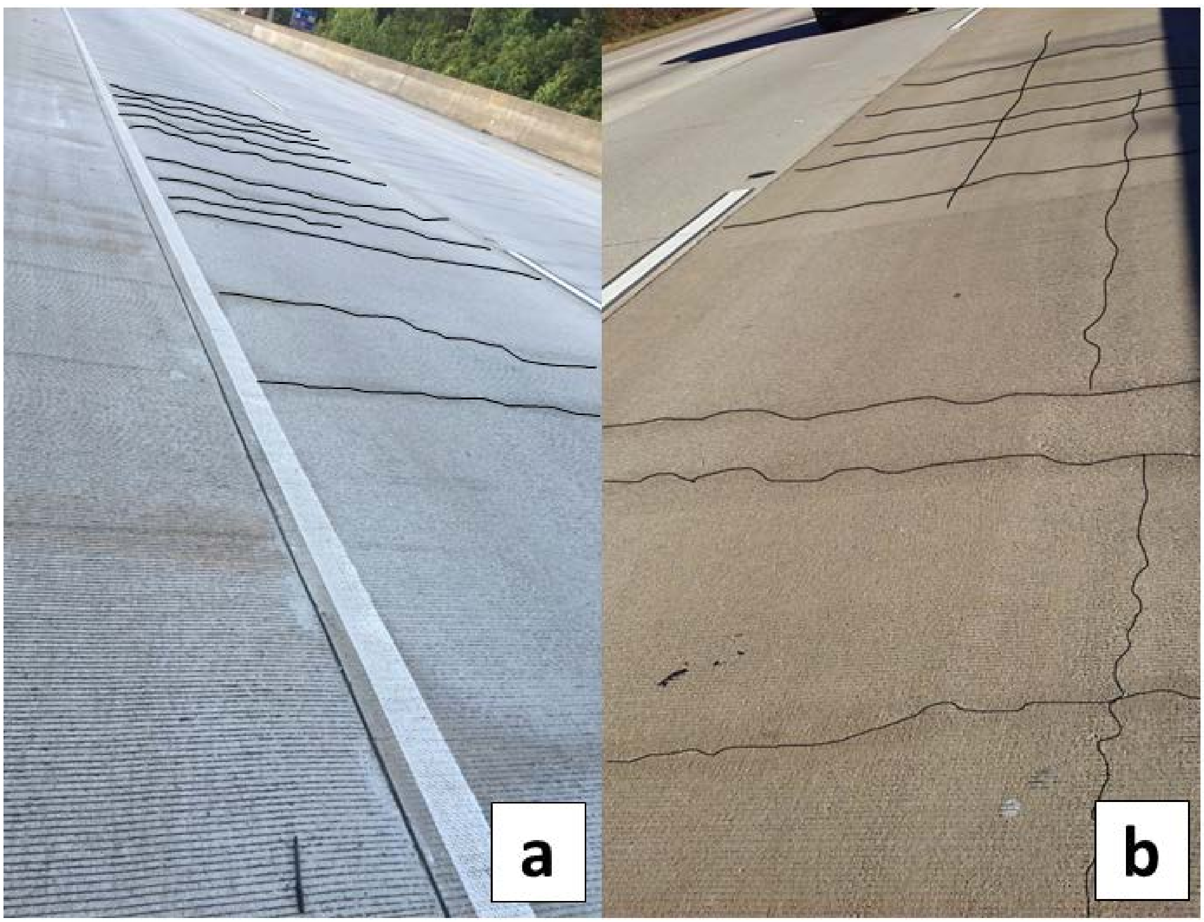


Figure 19 - Site Photos of CRC Pavement

. (a) I-85 MP 45-44 (Fair Condition) (b) I-85 MP 55-54 (Poor Condition).

Note: Cracks enhanced for clarity.

	Parameters	I-85 MP 45-44 (Fair)		I-85 MP 55-54 (Fair/ Poor)	
		Outside	**Inside**	**Outside**	**Inside**
Condition & Profile	Condition	Fair		Fair	Poor
	Visual Distress Observed	None		None	Longitudinal Cracking, Punchouts, Joint Spalling, and Corner Breaks
	Crack Spacing (in.)	3.5 - 13		3.5 - 13	
	Age (years)	10 (2006)		10 (2006)	
	Pavement Structure (in.)	11.5"PCC/ 3.5"AC/GAB	11.5"PCC/ 3.5"AC/GAB	12"PCC/ 3"AC/GAB	12.5"PCC/ 2.5"AC/GAB
	AADT	50,400		71,700	

Table 9 - CRC Conditions

MP 45 is in fair condition and has a profile consisting of approximately 11.5 in. of Portland Cement Concrete (PCC), 3.5 in. of Asphalt-Concrete (AC). MP 55 is composed of 12" CRC, followed by 3"AC, and 12" AGBS underneath. The outside lane of MP 55 is in fair condition while the inside lane is poor condition. I-85 is composed of 3 lanes in one direction. However, the inside lane (lane #1), appears to be composed of a better mixture than the middle and outside lanes (lanes #2 and #3). Closely spaced transverse cracking (cluster cracking) is seen throughout all three lanes in both sections, as shown in Figure 19. These cracks vary in length from 8 to 36 inches. Signs of pavement distress (punchouts, spalling, and delamination) were occasionally seen throughout the inside lane of MP 55.

Several recommendations on crack spacing are available to prevent unnecessary damage. To minimize cluster cracking, the Federal Highway Administration recommends a crack spacing between 2 feet to 8 feet (FHWA, 2012). Caltrans has a similar recommendation for crack spacing, with a distance between 3 feet and 7 feet between cracks (Caltrans, 2007). The Texas DOT warns that a crack spacing less than 2 feet could “be a precursor to punchouts” (TxDOT, 2011). With respect to crack spacing, cases of cluster cracking and Y-cracking are unique aspects of short crack spacing that can be problematic in terms of their contribution to localized failures including punchouts. These types of cracking are generally more associated with certain inadequate construction activities such as localized weak support, variable slab base friction, inadequate concrete consolidation, and/or variation in the quality of concrete curing (FHWA, 2016).

In the MP 45 section, several transverse cracks were observed directly above the transverse reinforcement. The cracks measured approximately 3.5 inches in depth and 0.5 to 1 millimeter in width. In between the 3 ft. rebar spacing, 2 or 3 longitudinal cracks were also observed. In the MP 55 section, crack spacing was observed to be very similar to MP 45 in that most of the cracks ranged from 3.5 inches to 13 inches. Neither pavement sections meet the recommended crack spacing from other DOT's, but they are not seen as a sign of distress in Georgia pavements.

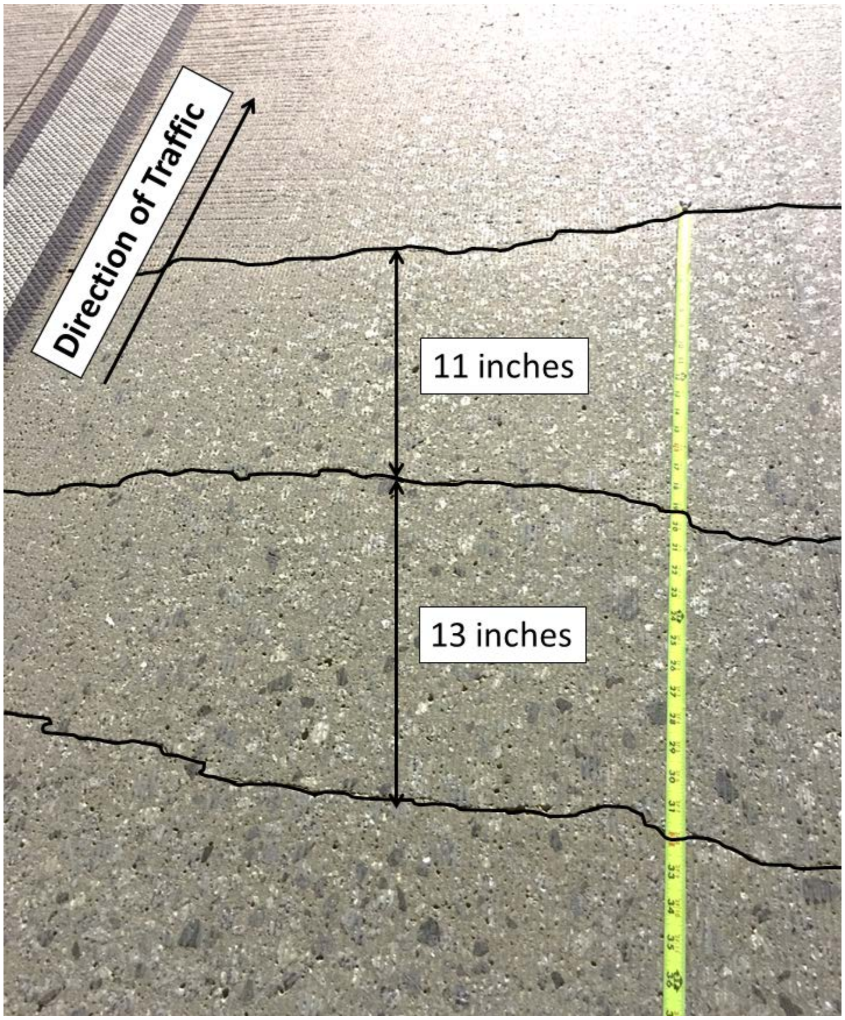


Figure 20 - I-85 Typical Transverse Crack Pattern (Cluster Cracking)

5.3 Non-Destructive Testing

Non-destructive testing was carried out by using a Falling Weight Deflectometer (FWD) and Ground Penetration Radar (GPR). More information on these technologies can be found in Section 3.2 - Review of Pavement Forensic Technologies – Non-destructive. A summary of information acquired from NDT testing is shown in Table 10. The GPR data shows consistently level pavement layers. The transverse rebar spacing is also identified as 3 feet on center, as shown in Figure 20. When the GPR machine scans metal rebar, the resulting image is slightly distorted. This results in arrow-like shapes seen below the rebar in Figure 20.

Table 10 - CRC NDT Results and Design Parameters.

	Parameters	**I-85 MP 45-44 (Fair)**		**I-85 MP 55-54 (Fair/ Poor)**	
		Outside (Lane 3)	**Inside (Lane 1)**	**Outside (Lane 3)**	**Inside (Lane 1)**
	Average ISM (kip/in)	9400	3800	3600	4100
	Back-calculated subgrade reaction (pci)	460	221	--	--
CRC Design Parameters	Surface Texture	Transverse Tining		Transverse Tining	
	Epoxy Coated Rebar	No	No	No	No
	Longitudinal Rebar Depth (Clear Cover) (in.)	3.75	3.75	3.25	4.5
	Longitudinal Rebar Diameter (No.)	0.75" (#6)	0.75" (#6)	0.75" (#6)	0.75" (#6)
	Transverse Rebar Depth (Clear Cover) (in.)	4.25	4.25	4	5.75
	Transverse Rebar Diameter (No.)	0.5" (#4)	0.5" (#4)	0.5" (#4)	0.5" (#4)
	Longitudinal Rebar Spacing (ft.)	0.45 to 0.5	0.45 to 0.5	0.42 to 0.46	0.42 to 0.46
	Transverse Rebar Spacing (ft.)	3	3	3	3

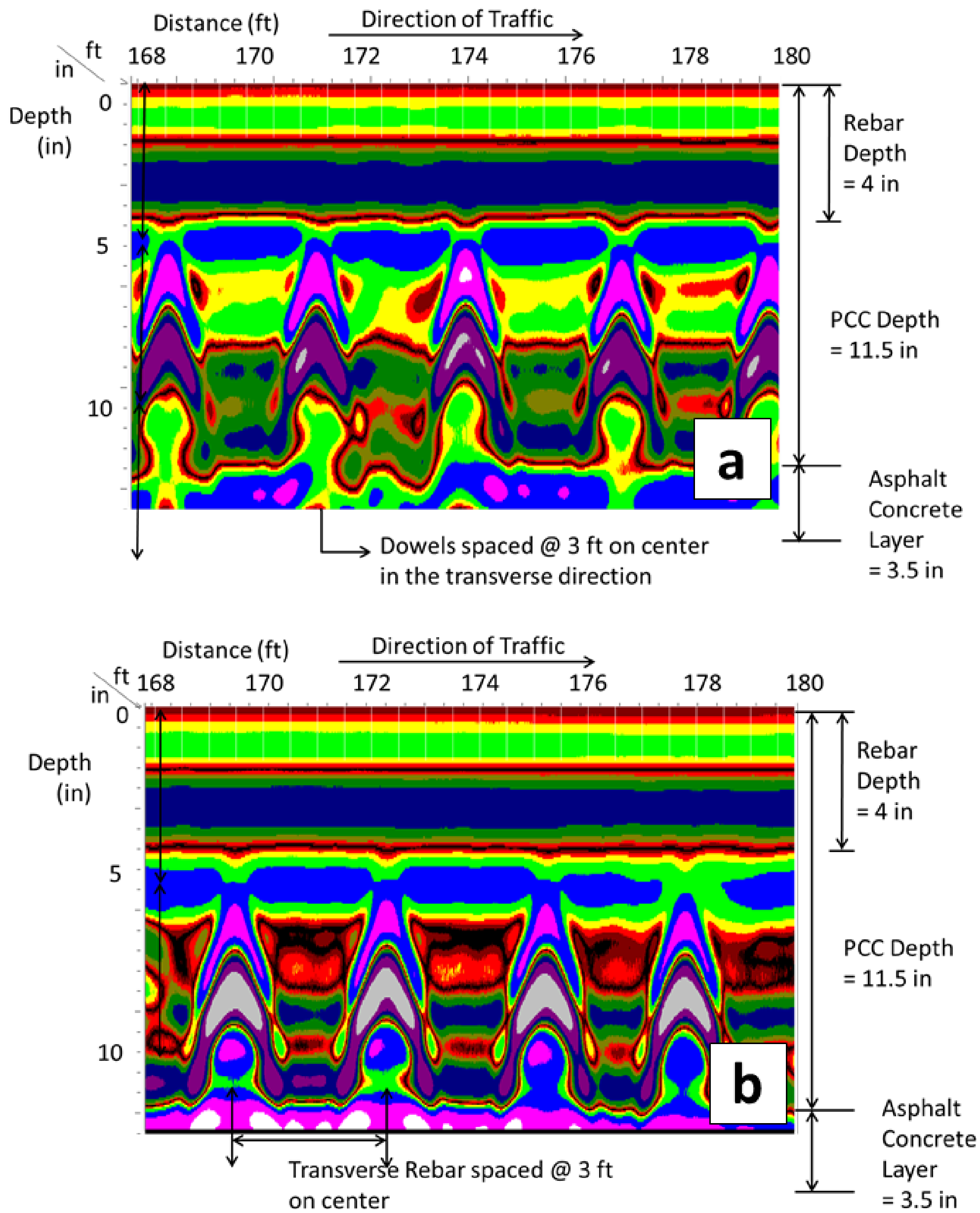


Figure 21 - GPR scans in the direction of traffic:
(a) I-85 MP 55-54 -Fast Lane; (b) I-85 MP 45-44 -Fast Lane;

Using the FWD data, a modulus of subgrade reaction, *k* was back-calculated based on the 1993 AASHTO design guide (AASHTO, 1993). ISM results and the back-calculated subgrade reaction are summarized Table 10. As seen in Figure 21, the outside lane of MP 45 has an unusually high ISM value and irregular variation, which might be interpreted as a possible structural variation. After taking coring samples, a very stiff subgrade was located underneath the slow lane (lane 3). This results in an abnormally high ISM value, as seen in Figure 21.

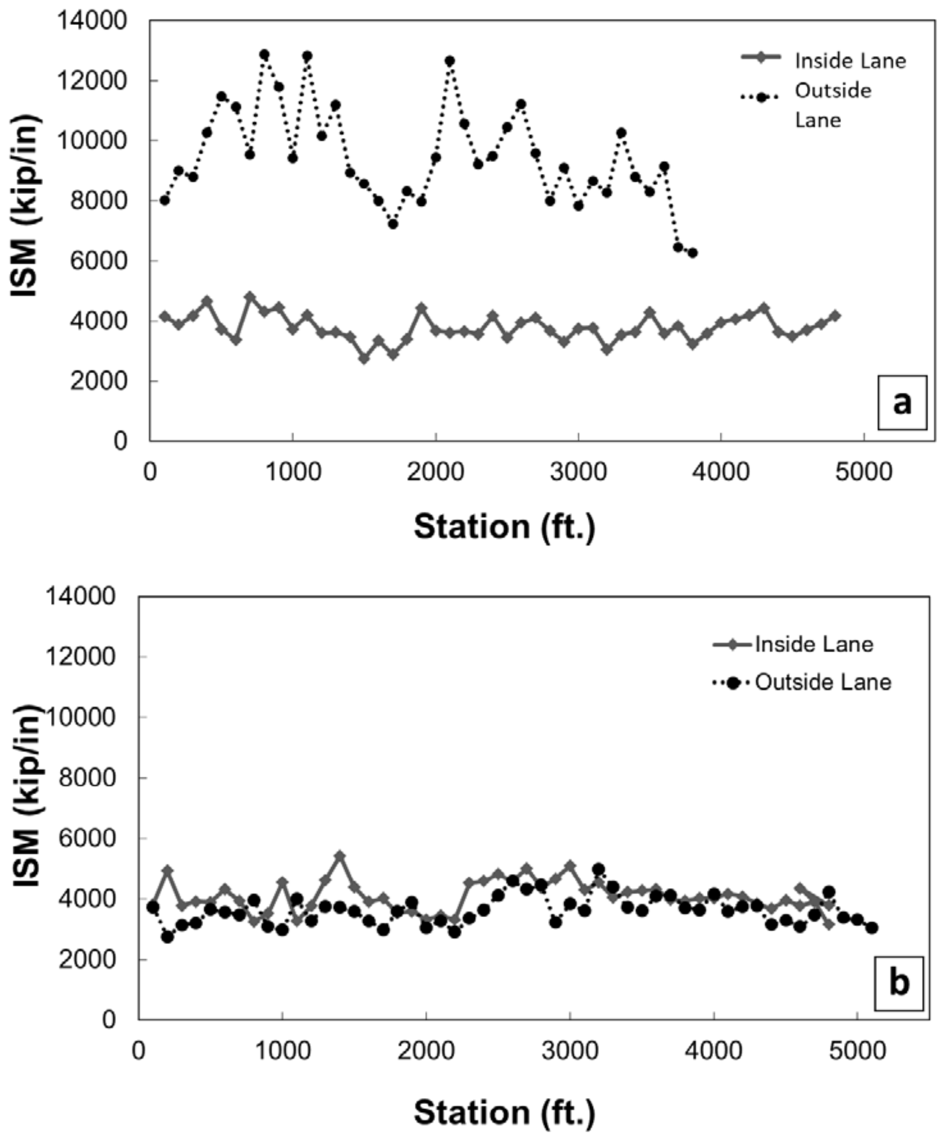


Figure 22 - ISM Plots for CRC Pavements
(a) I-85 MP 45 (b) I-85 MP 55

5.4 Destructive Testing – Coring and Field Testing

The coring process, including the machine used to drill cores, is shown in Figure 22. For both sites on I-85, a 4-inch core drill was used for laboratory tests and a 6-inch core bit was utilized to observe existing pavement thickness and reinforcement size & location. Based on the recommendations in Section 7.3 of the NCHRP 747 report, the cores were taken on the centerline of the slab, wheel paths in slow and fast lanes, and cracks to document the crack depth (Rada, 2013). Additionally, the coring locations for both sites were reviewed from the non-destructive test data and visual inspection information. The locations of cored specimens are shown in Figures 23 and 24 which provide a 3D schematic of the pavement section.

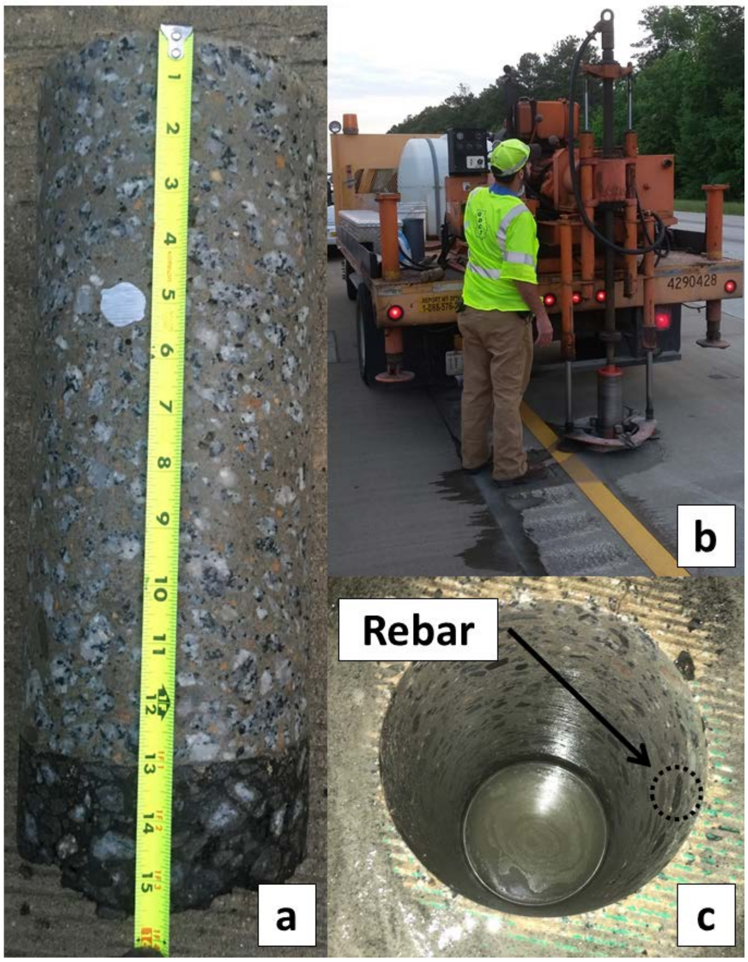


Figure 23 - Typical Cores at Rebar Locations, (a) Core sample; (b) Coring machine; (c) Inside view of a cored pavement.

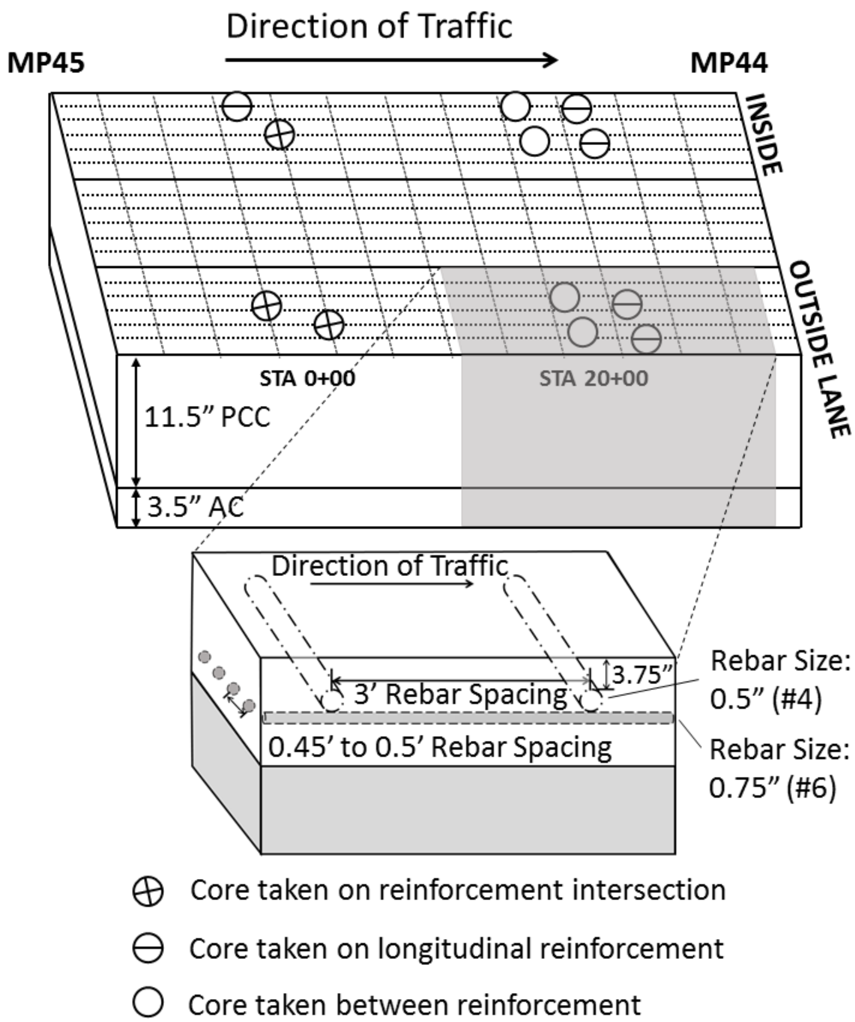


Figure 24 - 3D View of Pavement Design Parameters for Fair CRC (I-85 MP 45-44)

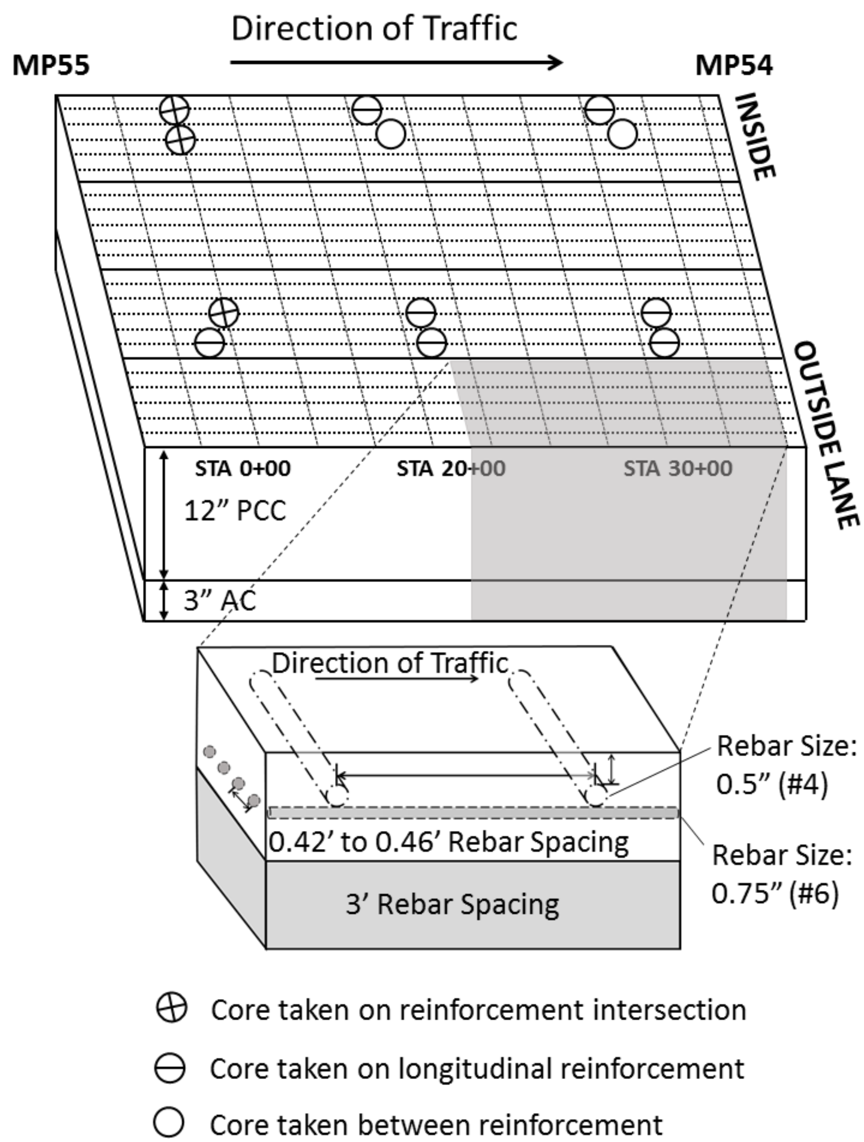


Figure 25 - 3D View of Pavement Design Parameters for Poor CRC (I-85 MP 55-54)

Photos of all cores extracted are shown in Figure 25. As seen in the Figure, the reinforcement depth varies by ± 0.5 inches. MP 45 shows relatively consistent consolidation with few voids. MP 55 has consolidation problems in the slow lane, more specifically, sample C1MTR. A visible difference can be seen when comparing the concrete from the outside and inside lanes from MP 55. More details on these two material differences are explained in section 5.6 -Petrographic Examination.

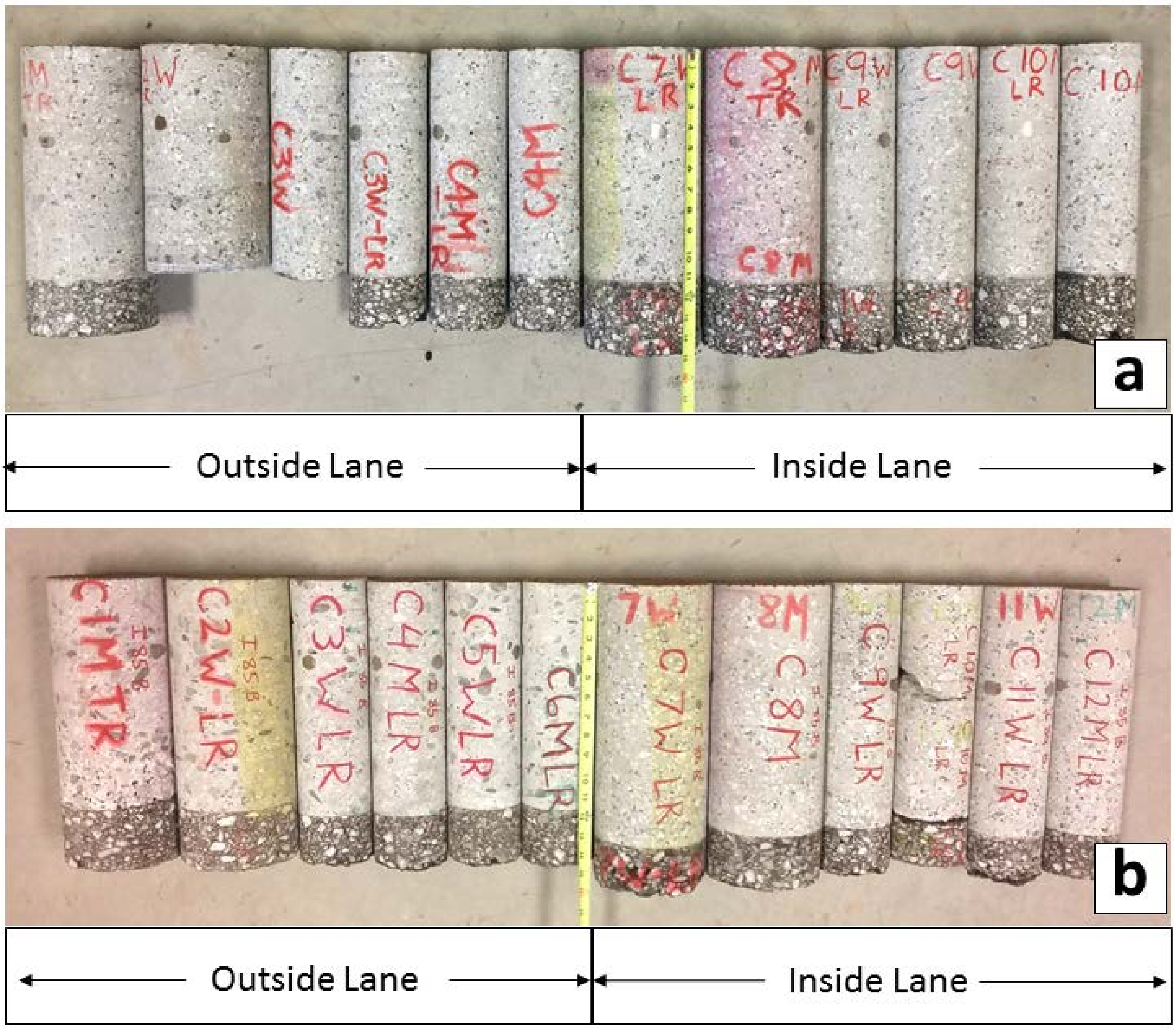


Figure 26 - All cores extracted

(a) Fair CRC (I-85 MP 55-54) (b) Poor CRC (I-85 MP 45-44)

5.4.1 MP 45 Section Coring and On-site Testing

The core (C8M-TR) was extracted from the MP 45 location and tested for carbonation, as well as alkali-silica reaction using a chemical testing kit. Both test results showed a negative reaction, which means no carbonation or alkali-silica reaction was observed. The remaining cores maintained a relatively consistent measurement of 11.5 inches, with an occasional variation no more than ±0.5 inches. The longitudinal reinforcement depth remained consistent in the outside lane. In the inside lane, however the longitudinal depth varied by as much as 0.75 inches. Variations in reinforcement depth can be caused by leveling on pavement surface. It was also observed that neither longitudinal nor transverse reinforcement were epoxy coated.

5.4.2 MP 55 Section Coring and On-site Testing

Visually, many transverse cracks were discovered during coring and on-site testing. Two cores, one from the outside lane and one from the inside lane, were tested for carbonation and alkali-silica reaction using a chemical testing kit. Both test results showed a negative reaction, meaning no carbonation or alkali-silica reaction was observed. A core sample (C-1M-TR) was taken over a transverse crack, to determine the crack depth (5 in.). The crack was observed to propagate through the coarse aggregate, not around it.

Neither longitudinal nor transverse reinforcement were epoxy coated. During the site investigation, it was observed that the seal has worn between the inside lane (lane 1) and the adjacent lane (lane 2). It was also noticed that the seal was missing between the outside (lane 3) and shoulder. Lack of a proper seal could result in water seeping underneath the pavement layer and penetrating the soil below, causing erosion of the soil particles over an extended period of time.

5.5 Destructive Testing – Laboratory

A summary of laboratory test results is described in this section. More information on these specific technologies can be found in Section 3.4 – Laboratory Testing Methods.

The CTE tests were conducted using cored specimens in accordance with AASHTO T 336 (AASHTO T 336, 2011). The CTE of Portland cement concrete (PCC) generally ranges between 4.4 and 5.5 microstrains/°F. The measured CTE values for MP 45 and MP 55 are 4.73 and 4.6, respectively. These values are within their recommendations.

RCP tests were performed according to ASTM C1202-12: Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration (ASTM C1202-12, 2012). As shown in Table 11, the RCP values in sections MP 45 and MP 55 were determined to be low and high, respectively.

The MOE tests for the JPC specimens were conducted in accordance with the ASTM C469: Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression (ASTM C469, 2014). The MOE for MP 45 showed reasonable values. MP 55 showed particularly low stiffness, although the average compressive strength was comparable to values determined from MP 45 (Table 11). To investigate the possible reasons, petrographic analyses were performed. The next section presents a petrographic analysis and results from the cored specimens.

Table 11 - Summary of Core Test Results.

	Parameters	I-85 MP 45-44 (Fair)		I-85 MP 55-54 (Poor)	
		Outside	Inside	Outside	Inside
Condition	Good/Fair/Poor	Fair	Fair	Fair	Poor
On-site Field Testing	ASR	No	No	No	No
	Carbonation	No	No	No	No
Laboratory Testing	MOE (ksi)	3687	3417	3125	2450
	f'c (psi)	7,400	7,300	7,700	7,900
	RCP (Coulomb)	2085	3058	3382	3909
	CTE (in/in/°F)	4.73	4.6	5.25	5.34
Petrographic Analysis	Coarse Aggregate	See I-85 MP 55-54 Fast Lane		Crushed Granite and Amphibolite	Crushed Granite
	Maximum Aggregate Size			3/8"	3/4"; Segregation at the surface
	Fine Aggregate			Natural quarts; The max sand particle size s 3mm	Natural quartzite and gray quartz; The maximum sand particle size is 1/5"
	W/C ratio			0.4-0.45	0.4-0.45
	Fly ash			Class C fly ash and no slag in the cement	Class C fly ash and no slag in the cement
	Paste			The paste is of fair quality	The paste is of fair quality. The paste is somewhat soft as it can be scratched by a Mohs 3 hardness point.
	Air entrained			No	Yes
	Air content			Approximately 3% air consisting of mostly entrapped voids.; The air is not evenly distributed as there is more air in the middle of the core.	Approximately 5-7%. Mostly air entrained air voids. There is frequent ettringite in the voids.
	Cracks			Rare microcracks in the paste.	There are occasionally internal cracks in the aggregate. These cracks could present durability issues but do not appear to be presently detrimental.
	Other distresses to note			No corrosion is present at the periphery of the rebar. It has 3and3/4 inches of top surface concrete cover	

5.6 Petrographic Examination

Two cores from this site were selected for petrographic analysis, one in the outside and inside lane of MP 55. The outside lane of the MP 55 has a concrete mixture that is visually similar to the pavement from MP 45. Therefore, no cores were selected from MP 45. The petrographic analysis of selected cores (C3W-LR and C8M-LR from I-85 MP 55-54) was performed by TEC Services, Inc. located in Lawrenceville, Georgia. This test involves taking an in-depth examination of selected cores to determine multiple construction and material parameters that are not available otherwise. The analysis of the sample from the outside lane of MP 55 indicated that the material makeup of the section consisted of a nominal maximum aggregate size (NMAS) of 3/4 inch crushed granite as coarse aggregate and natural quartzite and gray quartz for fine aggregate. The water-to-cement ratio ranged between 0.4 and 0.45. Class C fly ash was included, but no slag was included in the mixture. The concrete is air-entrained, resulting in a 5%-7% air content. In reference to quality acceptance criteria from GDOT, the water-to-cement ratio is acceptable. The design air content range is between 4.0 to 5.5, so the concrete from MP 45 may meet the design requirements (GDOT 430.3.06, 2013).

The analysis of the CRCP from the inside lane of MP 55 revealed that the material makeup of the section consisted of a NMAS of 3/8 inch crushed granite and amphibolite as coarse aggregate. Natural quartz was used for fine aggregate. The paste was identified as fair quality with a water-to-cement ratio of 0.4 and 0.45. Class C fly ash was included. Low air content was observed (3%). In reference to quality acceptance criteria from GDOT, the water-to-cement ratio is acceptable. The design air content range is between 4.0 to 5.5, so the concrete from MP 55 does not meet the design requirements (GDOT 430.3.06, 2013). The air was entrapped and described as being “not evenly distributed” and having “more air in the middle of the core”. The poor air distribution can be attributed to poor consolidation, as shown in Figure 26(a). Although the paste was reported to be of fair quality, it was much softer than the aggregate, as seen in Figure 26(b). Also, segregation issues can be seen in Figure 26(c), which can be attributed to excessive vibration.

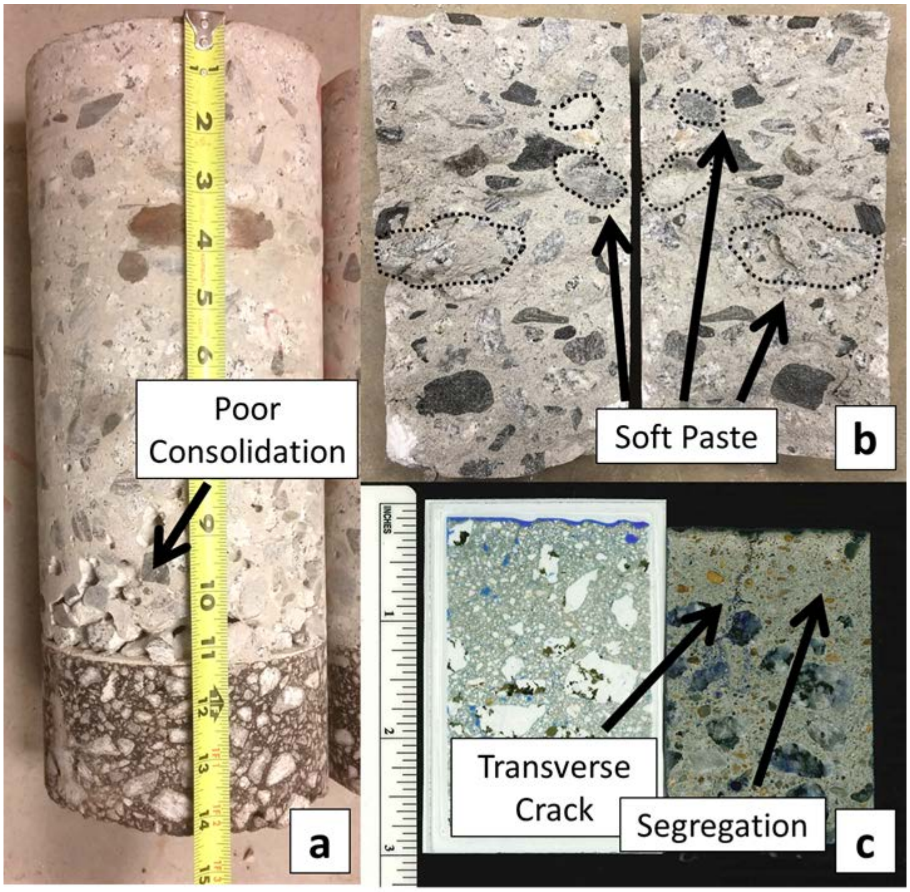


Figure 27 - Construction Signs of Distress* on I-85 MP 55-54 (Poor Condition)

5.7 Analysis of Testing Results

A summary of the test results is shown in Table 11. MP 45 has a CTE value of 4.6-4.73, and MP 55 exhibited a value of 5.25-5.34. Both results are within an acceptable range (Kim, 2012).

RCP tests were run on MP 45 in the inside and outside lanes. Values ranged from 2000 to 3500, with an average value of approximately 2100 for the outside lane and 3100 for the inside lane. This indicates a low-to-moderate chloride ion penetrability. RCP tests run on MP 55 in the inside and outside lanes resulted in values from 2800 to 3900. The average value was approximately 3400 for the slow lane and 3900 for the fast lane, respectively. This indicates a moderate chloride ion penetrability.

The compressive strength for both sections ranged between 7700 and 7900 psi for MP 45 section, whereas it ranged between 7300 and 7400 psi for MP 55. Both sites are well within the acceptable ranges of 3,000 psi for Class 1 and 3,500 for Class 2 mixtures (GDOT 430.3.06, 2013).

Several punchout sections were observed within the inside lane of MP 55. The NCHRP 747 guide prescribes possible causes of punchouts in CRCP can be attributed to “low PCC strength” or “steel reinforcement corrosion”. The same guide states that longitudinal cracks in CRCP may result from “high stabilizer contents in [the] base”. Pavement Interactive reports that punchouts can be caused by “steel corrosion, inadequate amount of steel, excessively wide shrinkage cracks or excessively close shrinkage cracks” (Pavement Interactive, 2012). The causes of punchout failure are a clear result of closely spaced transverse cracks (FHWA, 2012; Caltrans, 2007; TxDOT, 2011). It is also warned that if the cracks widen more than 0.02 inches, moisture can infiltrate the pavement (Pavement Interactive, 2012). A full report of conclusions and recommendations can be viewed in Section 8 – Conclusions.

6. HOT MIX ASPHALT (HMA) PAVEMENT – SUPERPAVE

6.1 Introduction

The NCHRP 747 Guideline reports that AC pavement is often susceptible to distresses such as rutting, roughness, potholes, excessive noise, and skid resistance (Rada, 2013). Additionally, AC pavement often experiences many cracks, such as alligator, transverse, longitudinal, and block cracking. Most deficiencies observed nationwide include: rutting, alligator cracking, transverse cracking, longitudinal cracking, block cracking, roughness, potholes, excessive noise, and frictional characteristics (Rada, 2013). Long-term aging increases the viscosity of asphalt, causing it to become hard and brittle. Combined with vehicle traffic, these effects can lead to various types of distress within asphalt pavements.

To investigate how HMA pavements behave in Georgia, a forensic investigation was conducted using the NCHRP 747 report as a guideline. Two HMA sites have been investigated, SR-38 in ‘fair’ condition and SR-54 in ‘poor’ condition (Figure 27). SR-38 section shows less severe signs of longitudinal cracking and raveling (Figure 28(a)). Similarly, SR-54 shows visual signs of severe distress, mainly longitudinal cracking and raveling(Figure 28(b)). Field investigations were performed in two phases: non-destructive and destructive investigations. The non-destructive site investigation involved a visual inspection, Ground Penetration Radar (GPR) testing and Falling Weight Deflectometer (FWD) testing. Destructive field testing involved collecting pavement cores from the sites and conducting laboratory tests on the cored specimens.

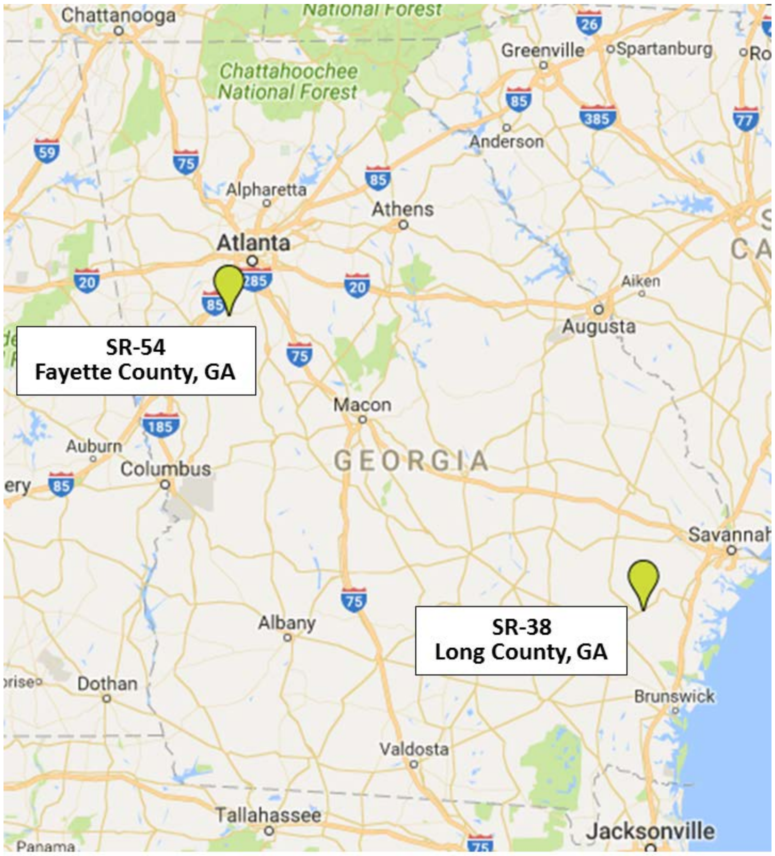


Figure 28 - AC Site Locations

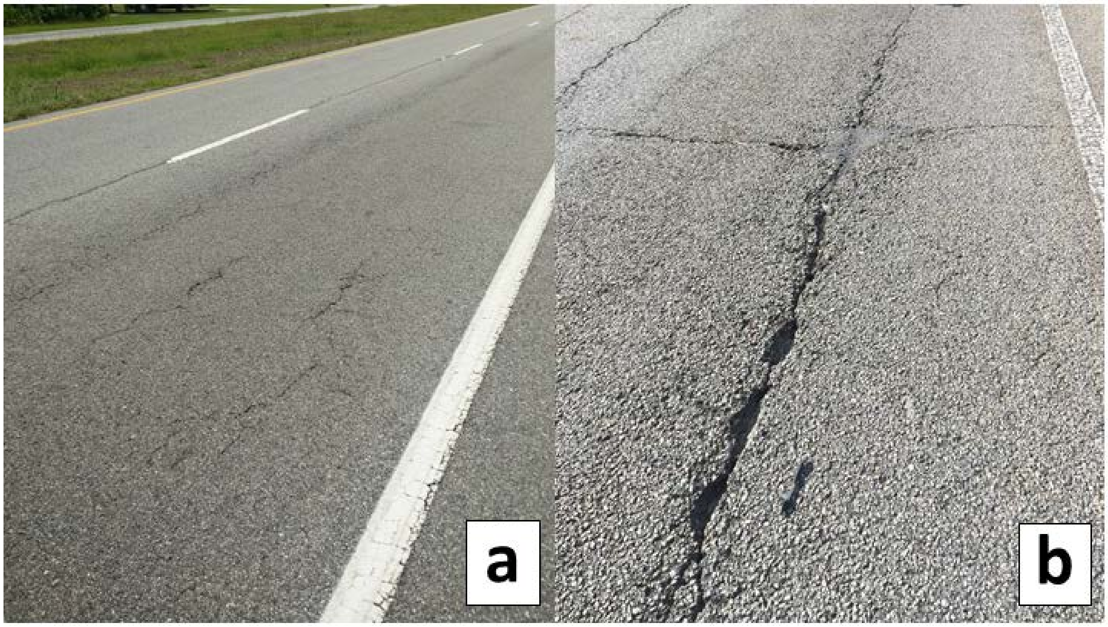


Figure 29 - Site Photos (AC pavements)

(a) SR 38 (fair condition) (b) SR 54 (poor condition)

6.2 Test Site and Field Setup

State Route (SR) 38 in Long County is composed of multiple layers of AC as shown in Table 12. In addition, soil cement is used instead of GAB. The state route consists of four-lane divide highway (two lanes in a single direction). This section was intended to be observed as a section containing SP in "fair" condition, however, upon inspection, the road was observed to have experienced significant deterioration. Numerous surface cracks of a moderately high severity were observed running through most of the pavement. These surface cracks consisted of longitudinal cracks which ranged in moderate to severe conditions over the majority of the section. Intersection cracks occasionally resulted from the amount of transverse and longitudinal cracks. A moderate level of particle loss was also observed in the left wheel path of the outside lane (Figure 29(a)). Most damage was observed in the outside lane, which typically experiences more truck traffic. The PACES data for this road shows a steady decline from a 98 rating on 2008 to 71 in 2013. After 2013, the pavement quality significantly decreased to 58 due to an increase in load cracking and block cracking.

State Route (SR) 54 (Eastbound) in Fayette County is composed of AC with a densely packed Superpave (SP) surface layer. The road is a four-lane divided highway (two lanes in a single direction), with moderate levels of traffic. In addition, SR 54 contains many stoplights and frequently experiences traffic congestion. A section between MP 5 to MP 4 was selected for this evaluation as it showed poor pavement performance in both fast and slow lanes. The test section exhibited severe longitudinal cracking along the wheel-paths of the road that extended throughout a majority of the section with occasional transverse cracking in both fast and slow lanes. Severe longitudinal cracking and raveling were the most commonly observed signs of distress (Figure 29(b)). Potholes, patching, block cracking, reflective cracking, and alligator cracking were observed occasionally during the visual site investigation. The PACES rating for this section sharply decreased from 85 in 2010 to 50 between 2012 to 2014. In 2015, the pavement was seen to increase from 50 to 54. This small increase is considered to be the result of pavement rehabilitation in the form of patches and the filling of potholes. A comparison of site conditions and pavement profiles of the two HMA pavement sections is shown in Table 12.

Table 12 - Site Condition and Pavement Profile

	Parameters	SR 38 (Fair Condition)		SR 54 (Poor Condition)	
		Outside	**Inside**	**Outside**	**Inside**
General	Condition	Poor	Good	Poor	Poor
	Age (years)	N/A		26 (1990)	
	Total Pavement Structure (inches)	1.75 (12.5mm SP)/ 6.5 (19mm SP)/ 2 (25mm SP)/ 0.75 (12.5mm SP)/ 0.75 (19mm SP)/ 6" Soil Cement		1.5 (12.5mm SP)/ 2 (19mm SP)/ 4 (25mm SP)/ 10" GAB	6.5 (12.5mm SP)/ 7.75 (19mm SP)/ 8.75 (37.5mm SP)
	Visual Distress Observed	Longitudinal Cracking and raveling near the left wheel path of the outside lane (see Figure 28a)		Severe Longitudinal and Transverse Cracking, Raveling	
Traffic	PACES Score (2015), %	72		64	
	ADT (% Trucks)	5,860 (9.5% Trucks)		21680 (5% Trucks)	

Figure 30 - Typical Distress

(a) SR-38 (b) SR-54

6.3 Non-Destructive Testing

Non-destructive testing was carried out by using a Falling Weight Deflectometer (FWD) and Ground Penetration Radar (GPR). More information on these technologies can be found in Section 3.2 - Review of Pavement Forensic Technologies – Non-destructive. A representative scan from SR-38 and SR-54 is shown in Figure 30. The scan of SR-38 (Figure 30(a)) shows surface, base, and subgrade profiles with density information. As seen, the GPR scan color code of density is uniform, which is expected for less variability in pavement condition. However, SR-54 shows an irregular GPR scan color code of subgrade (Figure 30(b)). Variability of subgrade density and moisture level is expected, which can result in problems such as longitudinal and fatigue cracking, as well as rutting and potholes (Rada, 2013).

The ISM plots for SR-38 and SR-54 are shown in Figure 31. The ISM plot of SR-38 has an average ISM value of 1100 and 1200 kip/in for the inside and outside lane, respectively. The ISM plot of SR-54 shows an irregular trend in the inside lane (Figure 31(b)). The ISM value is steady around 1100, but rapidly increases to a value close to 3000. Three coring samples were taken within this irregular area. It was discovered that the total length of the extracted cores varied from 12 inches to 23. These irregular cores contained very thick binder and base mixes, with no GAB underneath. The most logical explanation is that the previous roadway underwent a full-depth rehabilitation that left some pieces of the existing roadway in place. It is assumed that the road was most likely widened at that time, which explains why the outside lane has a steady ISM value. The average ISM value for the outside lane is 900 kip/in, which is lower than the value of pavement from SR-38 (around 1100 kip/in).

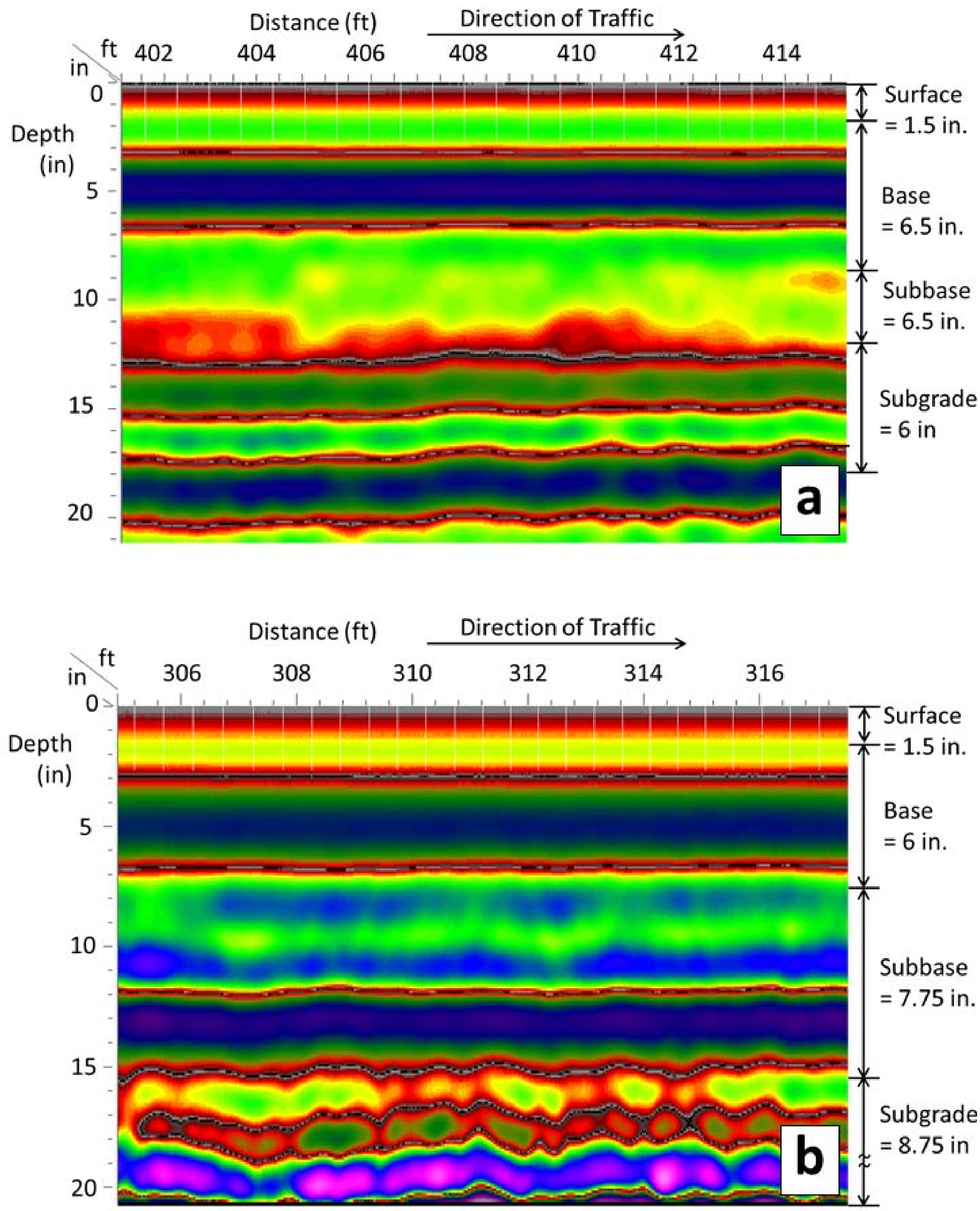
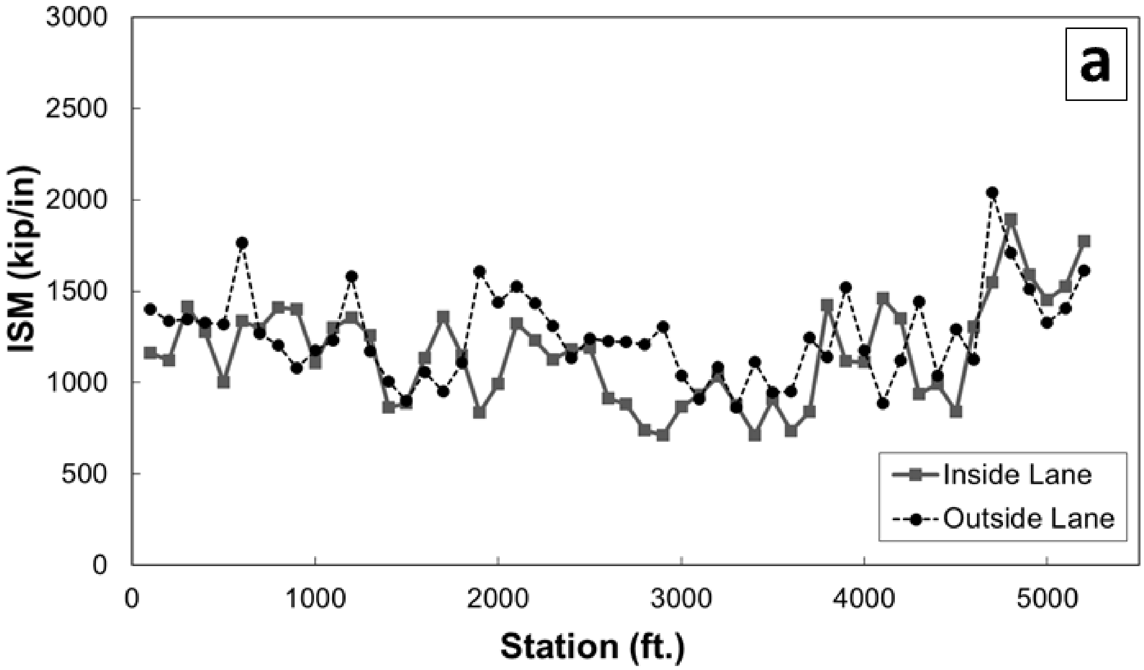


Figure 31 - SuperPave pavement scan of (a) SR-38 and (b) SR-54



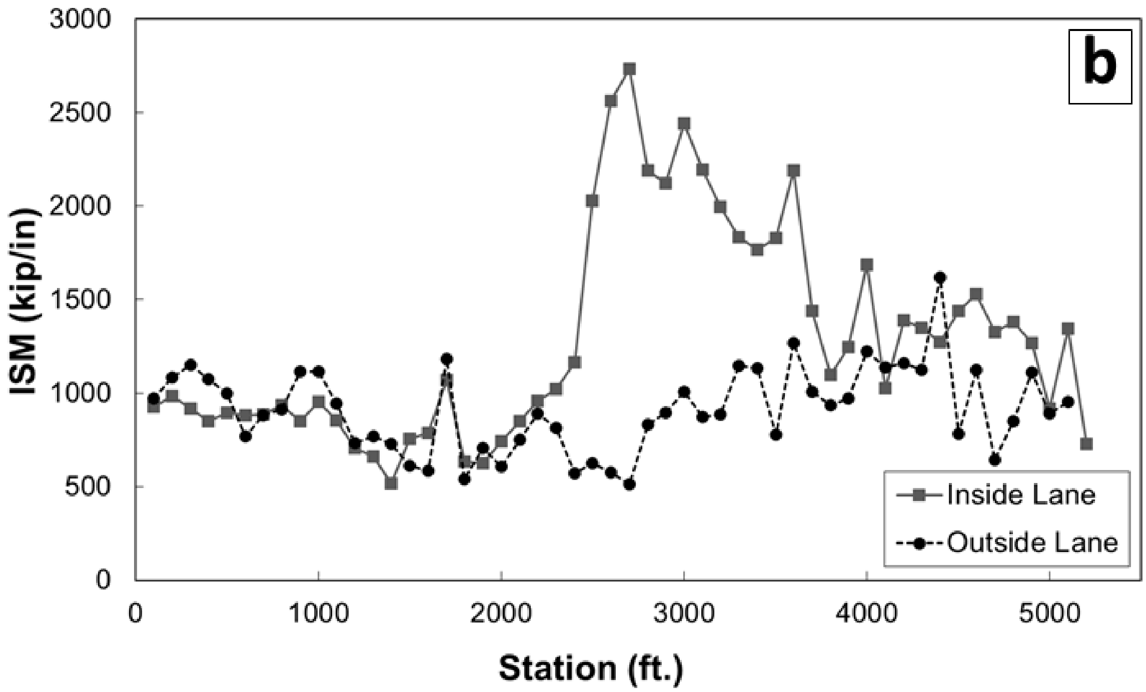


Figure 32 - ISM Plots for AC pavements

(a) SR-38 and (b) SR-54

The subgrade modulus for the subgrade layer of each pavement section was backcalculated using the AREA-based method described in the USDOT-FAA Advisory Circular (USDOT, 2004). This method uses the area of the radial deflection of the pavement found during the FWD procedure to estimate the subgrade modulus ($E_{subgrade}$) of the pavement layers. The value for $E_{subgrade}$ may then be used to calculate the design thickness required for the pavements to carry specific load conditions.

Additionally, the design Structural Number (SN), which represents an index of required pavement depths, was calculated for each pavement section using the AASHTO pavement design guide (1993). For these evaluations, 2 million and 1.3 million Equivalent Single Axle Loads (ESALs) were used to determine the required SN number for the SR-38 and SR-54 AC sections, respectively. ESALs were obtained from the traffic records provided. A summary of the calculated SN and required subgrade base depths are shown in Table 13. SR-38 meets the requirements for SN, meaning that the pavement is structurally sound to hold its loading. However, SR-54 does not meet the SN requirements, meaning that the pavement need a rehabilitation.

Table 13- Subgrade modulus, Effective and Required SN.

Parameter	SR 38	SR 54
Subgrade Modulus, $E_{subgrade}$ (psi)	9,776	4,714
Effective Structural Number, SN_{eff}	5.13	4.30
ESAL	2 Million	1.3 Million
SN, Required	3.85	4.60
$SN_{req'd} > SN_{eff}$	Yes	No

The subgrade soil modulus for SR-54 was determined to be 4,714 psi, which indicates silty-clay type soils (CL, CH, ML, MH) normally found in Fayette county, Georgia. However, the subgrade modulus for SR-38 was 9,776 psi and significantly higher than the modulus determined for SR 54 AC section. The sandy soils (SW, SP, SM, SC) may be present in SR 38 pavement section. This observation matches with soil survey of Georgia, which indicates that sandy soils are generally found in Long county, Georgia (USDOA, 1982).

6.4 Destructive Testing – Coring and Field Testing

6.4.1 SR-38 Coring and On-site Testing

When coring the samples from SR-38, the cores contained compacted soil cement below their respective AC layer. The use of soil cement is typical in South Georgia to save GAB material and haul costs as most of quarries for GAB are located in North Georgia. Figure 32 shows typical cores takes at joints from SR-38. Figures 32(a) and 32(b) show a core sample before and after extraction. As shown in Figure 32(c), there are several AC lifts within the cored specimen. This could be attributed to milling and overlay rehabilitation. All cores extracted from SR-38 and SR-54 are shown in Figure 34. Cores from SR-38 (Figure 34a) appear very similar in length and layer density.

As previously mentioned, SR-38 exhibits moderate raveling and fatigue cracking on the left side of the outside lane (Figure 29(a)). Due to the unique location of longitudinal cracking, it is suspected that the distress may be initiated by reflective cracking and deteriorated further due to traffic. Observation of cored specimens from SR-38 confirms that cracking is reflected from soil cement. Reflective cracking is a common distress among pavements when a soil cement is used as the soil cement typically experiences shrinkage cracks, which is reflected upward to the pavement surface (Adaska, 2004).

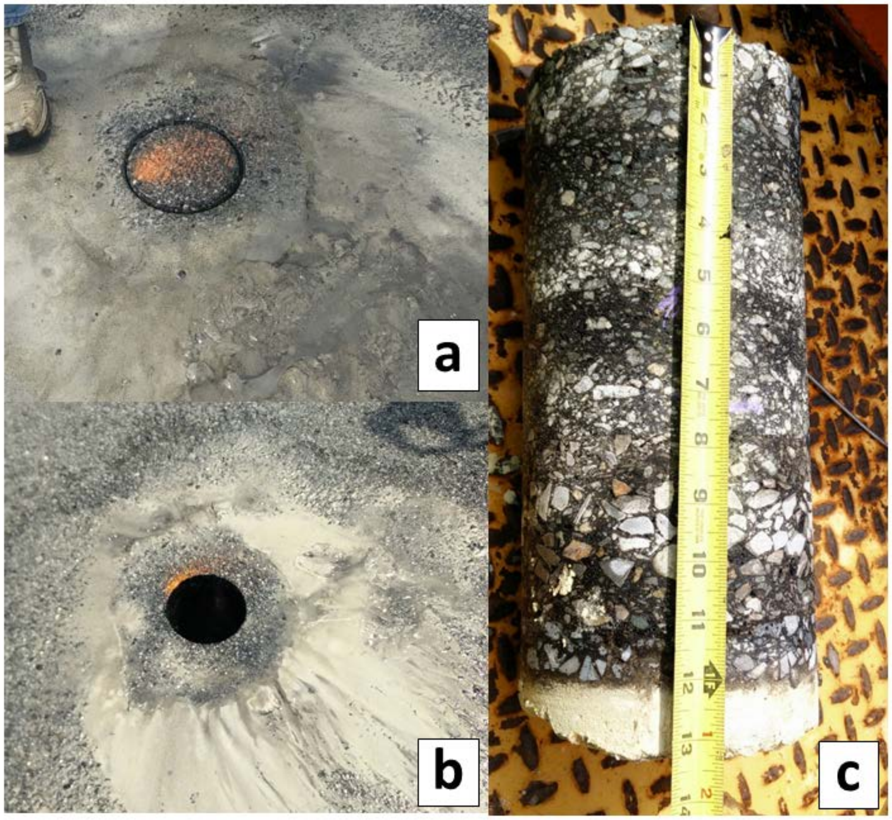


Figure 33 - Typical Cores at Joints (SR-38)

(a) Unremoved core sample (b) Pavement with core sample removed (c) Core sample

6.4.2 SR-54 Coring and On-site Testing SR-22 Section Coring and On-site Testing

All of the SR 54 coring samples taken contained large cracks. Many cracks initiated from either the bottom or top, and several samples were extracted in pieces. Figure 33 shows images from the coring process on SR-54. A large, irregularly shaped void was discovered when coring sample C-7 (Figure 33(b)). The void was also part of the extracted sample, as seen in Figure 33(c). The void is assumed to be the result of an organic material (e.g. wood) displaced during construction. All cores extracted from SR-38 and SR-54 are shown in Figure 34. The core samples from SR-54 are consistent in the outside lane. However, the fast lane shows cores of irregular lengths (Figure 33(c)). As mentioned previously, the 4th, 6th, and 7th samples in Figure 34(b) show full depth asphalt composed entirely of asphalt mixes. No GAB was found underneath these samples.

Design drawings were not available for SR-54. The irregular length of coring samples from the inside lane of SR-54 can lead to the conclusion that, the inside lane is composed of an existing pavement that contained an asphalt base (Figure 33c). The existing road underwent a full-depth rehabilitation on partial sections of the road. The two pavements are joined at the wheel path locations in the pavement. It seems that the large cracks shown in Figure 29 (b) initiated from this joint.

Figure 34 - Typical Cores at Joints (SR-54)

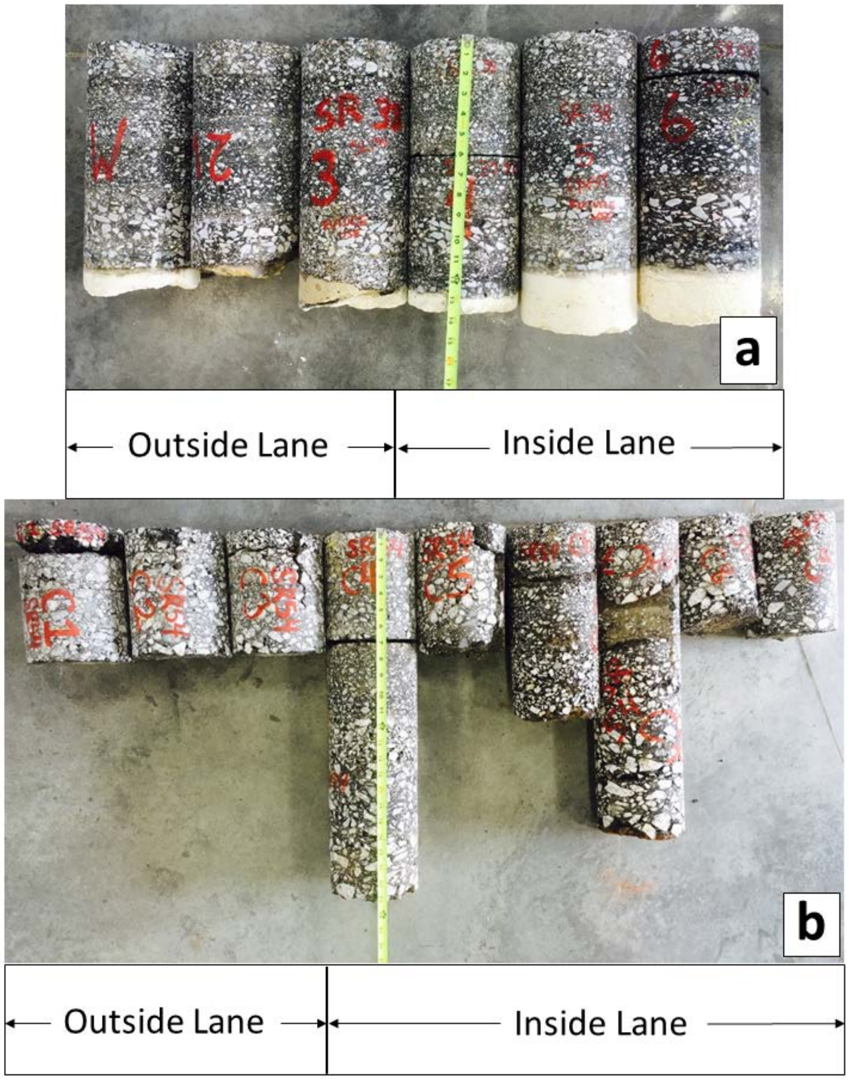


Figure 35 - All Cores – (a) SR 38 and (b) SR 54

6.4.3 Air Content Analysis

The air content observed in sample SR-38 is highly irregular (Table 14). The air void content of the surface lift is 7.3%, which is close to the general guideline of 4 to 7%. The binder lifts each had varying air contents, which are most likely not representative of the sample. SR 54 sample C-2 was taken in the slow lane, and shows a very high air content (11.3%), which is most likely indicative of raveling distress (Table 14). SR 54 sample C-2 taken from the fast lane exhibits a high air content (7.8%), which is also indicative of raveling distress. However, the binder lift below the surface has a low air content (4.0%), which may be the result of compaction from traffic loading.

6.4.4 Binder Content Analysis

The binder content was measured in accordance with AASHTO T 308 “Determining the Asphalt Binder Content of Hot Mix Asphalt (HMA) by the Ignition Method”. Each sample was ignited at a temperature of 538°C until the internal scale reached a constant weight (approximately 60 to 90 minutes). Figure 36 shows how samples look after burning in the ignition furnace. The binder content is calculated by the ignition oven and this information can be viewed in Table 14.

The surface course from SR-38 (sample C-2) contained a binder content of 5.1%, which is 17% higher than the binder content of SR 54 Sample C-2. The binder course has an average binder content of 5.96%, which is 28% higher than the binder content of SR 54 Sample C-2. SR 54 Sample C-2 and SR 38 Sample C-4 also have a differing binder content for their base course, with a 16% relative difference.

Figure 36 - Asphalt Sample after Ignition Burning

Table 14– Summary of Pavement Information for Selected HMA Sites

Crack Type	Site	Condition /Lane	Pvmt Layer	Material Type	Material Sub-Type	Thickness, (in.)	Gmb	Gmm	Air Void (%)	Asphalt Content (%)
LC BC RV RT	**SR 38 C-4**	Fair/ Inside	1	AC	1/2 in NMSA	2	(Not conducted)	2.4928	(Not conducted)	5.12%
			2	AC	3/8 in NMSA	1.5	2.2562	2.5194	10.4499	5.29%
			3	AC	3/8 in NMSA	2	2.3692	2.4358	2.7338	6.51%
			4	AC	1/2 in NMSA	3	2.4316	2.4505	0.7731	6.07%
			5	AC	3/4 in NMSA	2.5	2.2468	2.4906	9.7865	4.78%
			6	Soil Cement	Soil Cement	6	--	--	--	--
LC BC RV RT	**SR 54 C-2**	Poor/ Outside	1	AC	1/2 in NMSA	1.5	(Not conducted)	2.5163	(Not Conducted)	4.58%
			2	AC	3/4 in NMSA	2	2.3781	2.5261	5.86	4.64%
			3	AC	1 in NMSA	4	2.4302	2.5199	3.56	4.13%
			4	GAB	GAB	10	--	--	--	--
LC BC RV RT	**SR 54 C-4**	Poor/ Inside	1	Surface	1/2 in NMSA	1.5	(Not conducted)	2.5020	(Not conducted)	4.38%
			2	Binder	3/4 in NMSA	2.25	2.4071	2.5067	3.97	4.82%
			3	Base	3/4 in NMSA	3	2.3645	2.5272	6.44	4.50%
			4	GAB	GAB	10	--	--	--	--

Note: LC=Load Cracking, BC=Block Cracking, RV=Raveling, RT=Rutting

6.4.5 Sieve Analysis

After burning, each sample was weighed and sieved according to ASTM C136 “Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates”. Samples were sieved to determine the Nominal Maximum Aggregate Size (NMAS). The sieve results from each layer were compared according to NMAS by the GDOT Standard Specifications for the Construction of Transportation Systems (GDOT 828.2.03, 2013). The sieve analysis results can be seen in Tables 15 through 17. In SR-38, all layers meet the GDOT requirements, with the exception of the 4th lift. In SR-54, sample C-2 meets all grading requirements. However, it was observed that gradation of Layer 1 in sample C-4 is out of the GDOT specification range. A full report of conclusions and recommendations can be viewed in Section 8 – Conclusions.

Table 15 - SR 38 C-4 (Outside Lane)

Sieve Size	**Layer 1** (NMAS 1/2 in)	GDOT Spec (1/2 in.)	**Layer 2** (NMAS 3/8 in)	GDOT Spec (3/8 in.)	**Layer 3** (NMAS 3/8 in)	GDOT Spec (3/8 in.)	**Layer 4** (NMAS 1/2 in)	GDOT Spec (1/2 in.)	**Layer 5** (NMAS 3/4 in)	GDOT Spec (3/4 in.)
1	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%
3/4	100%	100%	100%	100%	100%	100%	100%	100%	100%	90-100%
1/2	96%	90-100%	99%	100%	100%	100%	98%	90-100%	76%	60-89%
3/8	82%	70-85%	98%	90-100%	98%	90-100%	88%	**70-85%**	64%	55-75%
4	52%	--	70%	55-75%	68%	55-75%	65%	--	45%	--
8	35%	34-39%	43%	42-47%	46%	42-47%	47%	**34-39%**	35%	29-34%
10	32%	--	39%	--	42%	--	44%	--	32%	--
16	27%	--	29%	--	33%	--	35%	--	26%	--
Pan	0%	--	0%	--	0%	--	0%	--	0%	--

Table 16 - SR 54 C-2 (Outside Lane)

Sieve Size	**Layer 1** (NMAS 1/2 in)	GDOT Spec (1/2 in.)	**Layer 2** (NMAS 3/4 in)	GDOT Spec (3/4 in.)	**Layer 3** (NMAS 1 in)	GDOT Spec (1 in.)
1 1/2	100%	100%	100%	100%	100%	100%
1	100%	100%	100%	100%	99%	90-100%
3/4	100%	100%	98%	90-100%	85%	55-89%
1/2	97%	90-100%	75%	60-89%	66%	50-70%
3/8	81%	70-85%	57%	55-75%	47%	--
4	49%	--	37%	--	33%	--
8	33%	34-39%	30%	29-34%	26%	25-30
10	30%	--	28%	--	25%	--
16	24%	--	24%	--	21%	--
Pan	0%	--	0%	--	0%	--

Table 17 - SR 54 C-4 (Inside Lane)

Sieve Size	**Layer 1** (NMAS 1/2 in)	GDOT Spec (1/2 in.)	**Layer 2** (NMAS 3/4 in)	GDOT Spec (3/4 in.)	**Layer 3** (NMAS 3/4 in)	GDOT Spec (3/4 in.)
1	100%	100%	100%	100%	100%	100%
3/4	100%	100%	99%	90-100%	97%	90-100%
1/2	94%	90-100%	85%	60-89%	75%	60-89%
3/8	70%	70-85%	66%	55-75%	60%	55-75%
4	40%	--	45%	--	39%	--
8	27%	**34-39%**	35%	29-34%	30%	29-34%
10	24%	--	31%	--	28%	--
16	19%	--	26%	--	24%	--
Pan	0%	--	0%	--	0%	--

7. CONCLUSIONS

7.1 Jointed Plain Concrete (JPC) Pavement - Conclusions and Recommendations

A forensic investigation was conducted on two JPCP sections, SR-22 and I-75, in 'good' and 'poor' condition. SR-22 shows no signs of distress. The distress in I-75 is mainly depicted by longitudinal cracks running full-depth along the centerline or wheel paths. Based on the findings of this investigation:

1. The cause of the observed distress in I-75 is the result of a combination of factors including traffic load, poor material composition, and environmental conditions such as thermal (or moisture-related) expansion/contraction and weather cycles.
2. The relatively high RCP results of the core samples and petrographic analysis in I-75 section suggests that the potential for concrete material degradation and punch-out distress is high. The microscopic examination of core samples obtained from I-75 documented the presence of microcracks and ettringite, which is conclusive evidence of ASR damage and temperature-related deformation. Furthermore, these distresses increase the potential for concrete tensile failure, which might have ultimately caused the observed distress, specifically longitudinal cracks.
3. The distresses found on I-75 are unique in that they are not commonly observed on pavements in Georgia although it should be noted that the I-75 JPCP section is an "old" section beyond its design life (>20 years).

7.2 Continuously Reinforced Concrete (CRC) Pavement - Conclusions and Recommendations

The distress in I-85 MP 54-55 in "poor" condition is mainly depicted by transverse cracks spaced at intervals less than 1 foot on center. Based on the findings of this investigation:

1. The cause of the observed distress on I-85 MP 54-55 in "poor" condition is the result of a combination of factors including poor material composition, aggregate segregation, soft paste (of Mohs 3 hardness), and environmental conditions such as thermal (or moisture-related) expansion/contraction and weather cycles.

2. Improper consolidation and irregularity in rebar cover depth may be attributed to roadway profiling, workmanship, and construction processes.
3. Closely spaced crack spacing (cluster cracking) is normal for CRC pavements in Georgia. However, the punchout locations in MP 55 inside lane should be investigated in more detail.
4. It is highly recommended to develop a 5 year monitoring program of CRCP sections in Georgia, in order to systematically identify signs of distress (e.g., crack width, longitudinal cracks, and punchout distress) and recognize the right (most economical) time for providing any rehabilitation, if needed. Although GDOT currently maintains concrete pavements on its interstate highways and state routes using CPACES, assessment of CRCP rating is based on distress types, which are more critical to JPCP rating assessment (I.e., faulting). As critical distress types to assess JPCP and CRCP conditions are quite different, development of a systematic CPACES rating methodology for Georgia CRCP are recommended to optimize the most economical time for maintenance and rehabilitation.

7.3 Hot Mix Asphalt (HMA) Pavement - Conclusions and Recommendations

A forensic investigation of two HMA sections in “fair” and “poor” condition was conducted. The distress observed in both pavement sections is mainly depicted by longitudinal cracking and raveling. Based on the findings of this investigation:

1. SR-38 shows longitudinal cracking and this cracking seems to be reflected from the soil-cement layer. It seems that the reflected longitudinal cracking in the left wheel path of the outside lane has been worsened by traffic.
2. Although SR-38 contains raveling and longitudinal cracking, the pavement is structurally sound based on FWD evaluation. In areas where longitudinal cracks are prevalent, it is recommended to mill and overlay the affected areas.
3. In SR-54, the extreme longitudinal cracking and raveling may have occurred as a result of widening between two existing pavement layers that were constructed at different times. The distresses have been worsened by increased traffic loadings thereafter. Based

on AASHTO 1993 design guide, a major rehabilitation is recommended on this test section.

8. NCHRP RECOMMENDATIONS

Using the National Cooperative Highway Research Program (NCHRP) Report 747 (Guide for Conducting Forensic Investigations of Highway Pavements) was very helpful throughout this investigation. The guide contains a very structured method for carrying out each step of the forensic investigation. Information is clear and easy to follow for pavement engineers who may not have much experience. In regard to recommended testing, each pavement type (JPCP, CRCP, HMA) is covered in meticulous detail. The guide also provides recommendations on how to analyze causes of pavement distress. Based on the experience of conducting a forensic investigation using the NCHRP Report 747, it is highly recommended for GDOT's adoption as the Forensic Pavement Guide for Georgia, with the following additions/recommendations:

- A comprehensive forensic investigation is very extensive, expensive, and time consuming. Precautions should be exercised to determine whether a full investigation is needed. It is recommended to determine the level of forensic analysis based on the "Phased Approach to Forensic Investigations" diagram in the NCHRP 747 Guideline (Appendix A).
- Rather than using NCHRP visual condition survey form, it is recommended to use the GDOT's visual inspection forms that have been used for PACES update (Appendices B and C). However, development of new methodology to assess PACES rating for CRCP is strongly recommended as current methodology doesn't reflect the functional condition evaluation of CRCP properly.
- Based on the GDOT RP 14-13 study, flow charts for pavement forensic investigations were developed (Appendices D, E, and F). The flow charts will provide the GDOT engineers with a systematic procedure when pavement forensic investigations deemed necessary.
- Traffic information along with pavement service life has large impact on pavement design and performance. To accurately investigate the pavement performance, it is

recommended that traffic information is efficiently archived and easily accessible. This includes: traffic volumes, traffic loads/load spectra, traffic growth, seasonal trends, load restrictions, and any related traffic information during entire pavement service life.

- It is recommended that all construction documents be efficiently archived and easily accessible when forensic investigations are started. This includes: all construction drawings, rehabilitation history, mix design, and other construction information.

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APPENDIX A

NCHRP Guide 747 “Phased approach to forensic investigations”

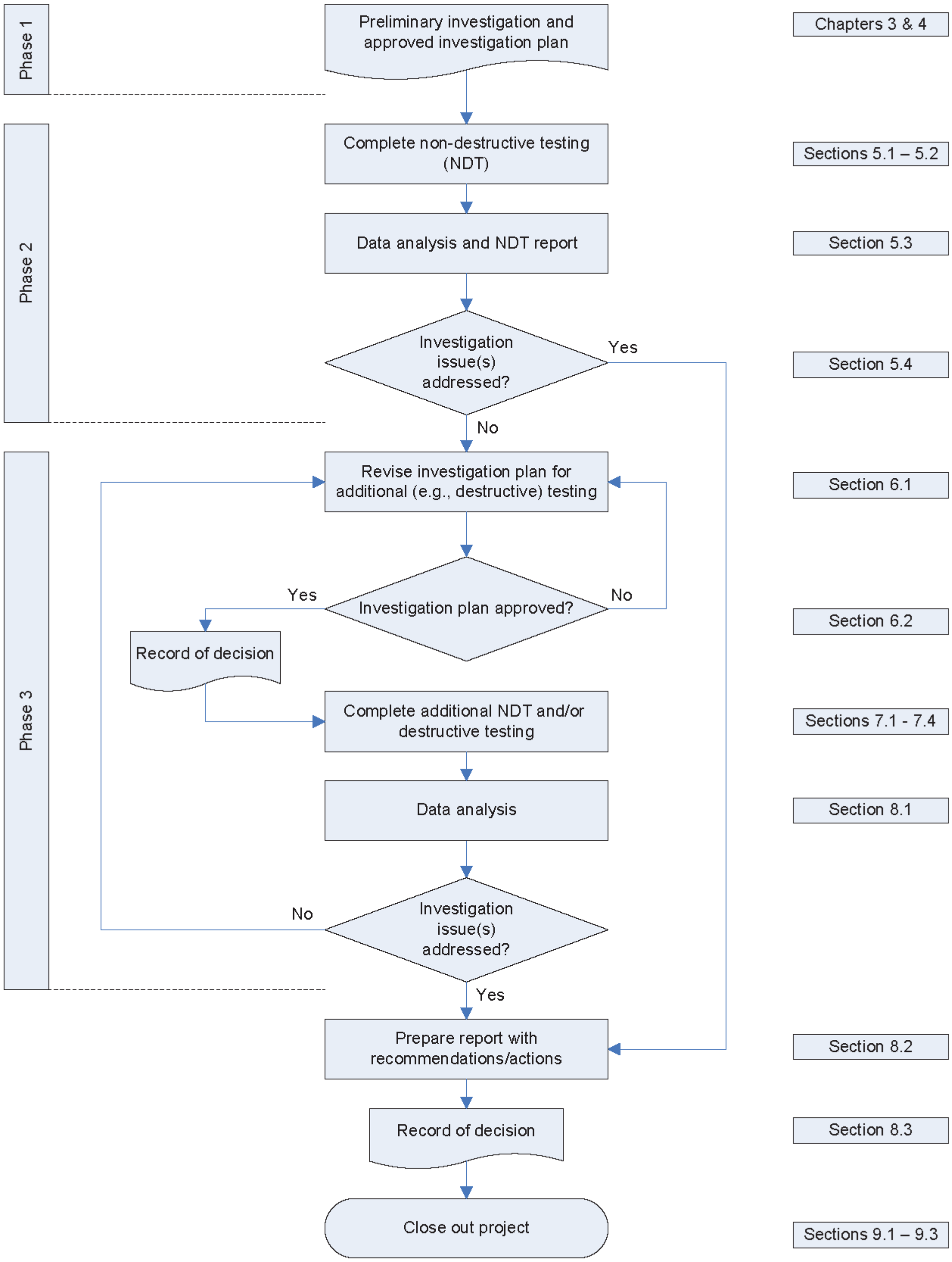


Figure A4.1. Phased Approach to Forensic Investigations.

APPENDIX B

Visual Assessment form for AC pavement

PAVEMENT FORENSIC INVESTIGATION	VISUAL ASSESSMENT - AC	Form #8
Investigation Name		**Investigation #**

Surfacing assessment							Sketch
Surfacing type							
Texture	Varying	Fine	F - M	Medium	M - C	Course	
Voids	Varying	None	N - F	Few	F - M	Many	

	Degree						Extent					Length	Width	Number	Location
	Slight					Severe	<5				>80				
Mechanical distress	0	1	2	3	4	5	1	2	3	4	5				
Other distress	0	1	2	3	4	5	1	2	3	4	5				
Bleeding/flushing	0	1	2	3	4	5	1	2	3	4	5	Narrow	Wide	Position	
Surface cracks	0	1	2	3	4	5	1	2	3	4	5				
Binder condition	0	1	2	3	4	5	1	2	3	4	5	Active	Stable	Position	
Aggregate loss	0	1	2	3	4	5	1	2	3	4	5				

Structural assessment

	Degree						Extent					Narrow (% area)	Wide (% area)	Position	Location
	Slight					Severe	<5				>80				
Cracks - block	0	1	2	3	4	5	1	2	3	4	5				
Cracks - longitudinal	0	1	2	3	4	5	1	2	3	4	5				
Cracks - transverse	0	1	2	3	4	5	1	2	3	4	5				
Cracks - fatigue	0	1	2	3	4	5	1	2	3	4	5				
Pumping	0	1	2	3	4	5	1	2	3	4	5				
Rutting	0	1	2	3	4	5	1	2	3	4	5				
Undulation/settlement	0	1	2	3	4	5	1	2	3	4	5				
Edge cracking/break	0	1	2	3	4	5	1	2	3	4	5	Number	Diameter		
Potholes	0	1	2	3	4	5	1	2	3	4	5				
Delamination	0	1	2	3	4	5	1	2	3	4	5				
												Small	Medium	Large	Location
Patching/digouts	0	1	2	3	4	5	1	2	3	4	5				

Functional assessment

	Degree					Influencing factors									
	Good				Poor										
Riding quality	1	2	3	4	5	Potholes		Patching		Undulation		Corrugation		Fatigue	
Skid resistance	1	2	3	4	5	Bleeding		Polishing							
Surface drainage	1	2	3	4	5										
Side drainage	✓	✗													

Notes	**Photos**	

APPENDIX C

Visual Assessment form for PCC pavement

PAVEMENT FORENSIC INVESTIGATION	VISUAL ASSESSMENT - PCC	Form #9

Investigation Name		Investigation #	

Surfacing assessment

Sketch

Pavement type	JPCP	JPCP + dowel	CRCP	JRCP		
Surface finish						
Texture	Varying	Fine	F - M	Medium	M - C	Course
Voids	Varying	None	N - F	Few	F - M	Many

	Degree: Slight					Severe	Extent: <5				>80	Length	Width	Number	Location
Mechanical distress	0	1	2	3	4	5	1	2	3	4	5				
												Narrow	Wide	Position	
Other distress	0	1	2	3	4	5	1	2	3	4	5				
Surface cracks	0	1	2	3	4	5	1	2	3	4	5				
Aggregate loss	0	1	2	3	4	5	1	2	3	4	5				

Structural assessment

	Degree: Slight					Severe	Extent: <5				>80	Narrow (% area)	Wide (% area)	Position	Location
Cracks - corner	0	1	2	3	4	5	1	2	3	4	5				
Cracks - longitudinal	0	1	2	3	4	5	1	2	3	4	5				
Cracks - transverse	0	1	2	3	4	5	1	2	3	4	5				
Cracks - D	0	1	2	3	4	5	1	2	3	4	5				
Cracks - other	0	1	2	3	4	5	1	2	3	4	5				
Pumping	0	1	2	3	4	5	1	2	3	4	5				
Rutting (chain/snotire)	0	1	2	3	4	5	1	2	3	4	5				
Faulting	0	1	2	3	4	5	1	2	3	4	5				
Spalling	0	1	2	3	4	5	1	2	3	4	5	Number	Diameter		
Punchouts	0	1	2	3	4	5	1	2	3	4	5				
Joint damage	0	1	2	3	4	5	1	2	3	4	5				
												Small	Medium	Large	Location
Patching/digouts	0	1	2	3	4	5	1	2	3	4	5				

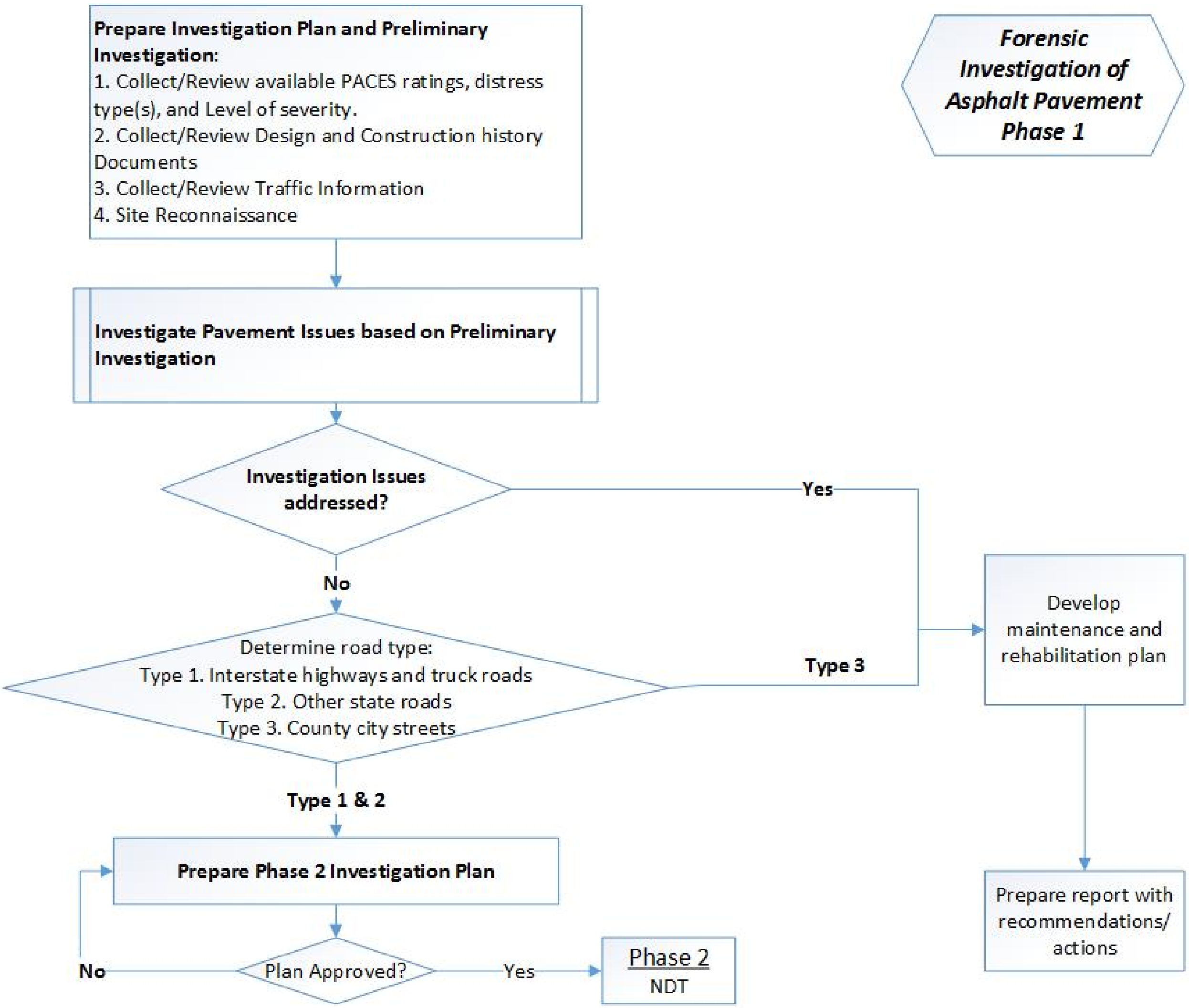
Functional assessment

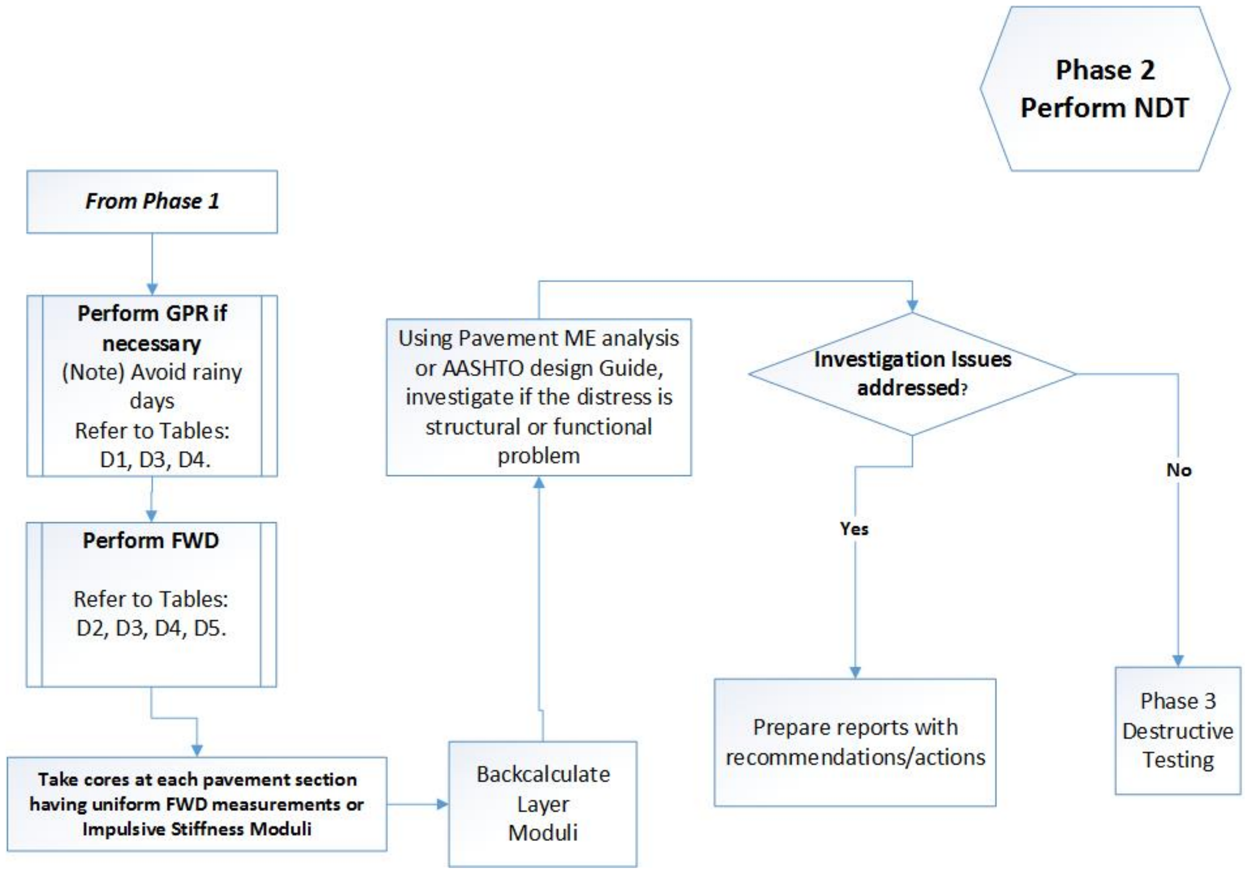
	Degree: Good				Poor	Influencing factors									
Riding quality	1	2	3	4	5	Joints		Cracks		Spalling		Punchouts			
Skid resistance	1	2	3	4	5	Tining		Polishing							
Surface drainage	1	2	3	4	5										
Side drainage	✓	✗													

Notes	Photos	

APPENDIX D

Flow Charts for Forensic Investigation of Asphalt Pavement


Prepare Investigation Plan and Preliminary Investigation:
1. Collect/Review available PACES ratings, distress type(s), and Level of severity.
2. Collect/Review Design and Construction history Documents
3. Collect/Review Traffic Information
4. Site Reconnaissance
Forensic Investigation of Asphalt Pavement Phase 1
Investigate Pavement Issues based on Preliminary Investigation
Investigation Issues addressed?
Yes
No
Determine road type:
Type 1. Interstate highways and truck roads
Type 2. Other state roads
Type 3. County city streets
Type 3
Develop maintenance and rehabilitation plan
Type 1 & 2
Prepare Phase 2 Investigation Plan
Prepare report with recommendations/ actions
No
Plan Approved?
Yes
Phase 2
NDT


Phase 2
Perform NDT
From Phase 1
Perform GPR if necessary
(Note) Avoid rainy days
Refer to Tables: D1, D3, D4.
Perform FWD
Refer to Tables: D2, D3, D4, D5.
Take cores at each pavement section having uniform FWD measurements or Impulsive Stiffness Moduli
Backcalculate Layer Moduli
Using Pavement ME analysis or AASHTO design Guide, investigate if the distress is structural or functional problem
Investigation Issues addressed?
Yes
No
Prepare reports with recommendations/actions
Phase 3 Destructive Testing

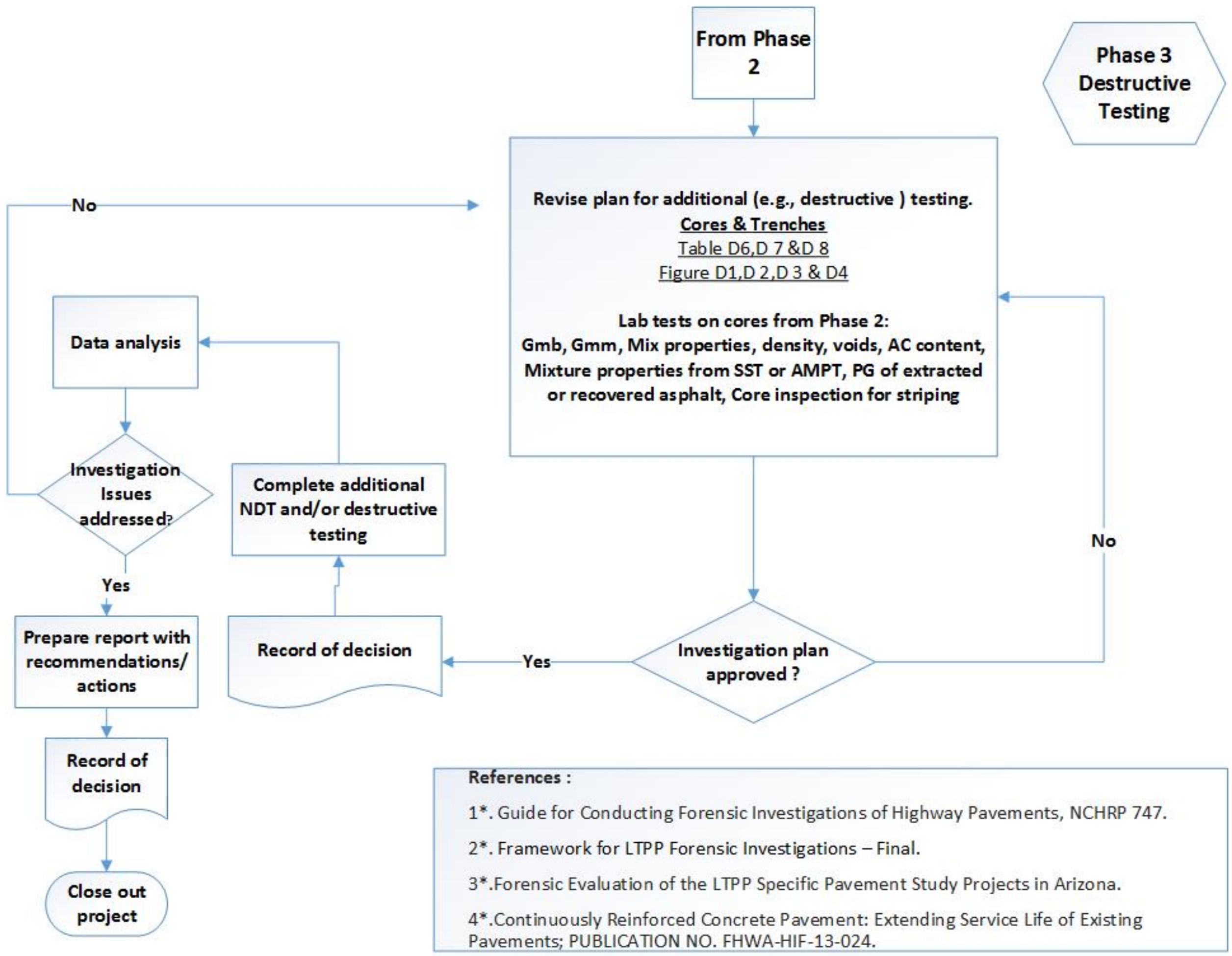

From Phase 2
Phase 3 Destructive Testing
Revise plan for additional (e.g., destructive) testing.
Cores & Trenches
Table D6,D 7 &D 8
Figure D1,D 2,D 3 & D4
Lab tests on cores from Phase 2:
Gmb, Gmm, Mix properties, density, voids, AC content, Mixture properties from SST or AMPT, PG of extracted or recovered asphalt, Core inspection for striping
No
Data analysis
Investigation Issues addressed?
Complete additional NDT and/or destructive testing
No
Yes
Prepare report with recommendations/ actions
Record of decision
Yes
Investigation plan approved ?
Record of decision
Close out project
References :
1*. Guide for Conducting Forensic Investigations of Highway Pavements, NCHRP 747.
2*. Framework for LTPP Forensic Investigations – Final.
3*.Forensic Evaluation of the LTPP Specific Pavement Study Projects in Arizona.
4*.Continuously Reinforced Concrete Pavement: Extending Service Life of Existing Pavements; PUBLICATION NO. FHWA-HIF-13-024.

Table D.1. GPR Performing Details.

GPR	
Location	Identical to FWD location
Frequency	2 Scans/Mile (for thickness)
	20 Scans/mile (for distress and problem identification)
Radar Frequency	> 1 GHz
Scan Depth	< 3ft
Dielectric Value	3 to 5.

Table D.2. FWD Performing Details.

FWD	
Location and interval	50 ft., if Section Length < 2 miles
	100 ft., if : 2 miles <Section length < 4 miles
	250 ft., if :Section Length > 4 miles
	3 to 15 ft. (1 to 5 m) in defined problem areas; offset in adjacent lanes
The surface temp.	> 60F (15C)

Table D.3. NDT on Asphalt Surfaced Pavements.

Issue	Possible Contributing Factors	Type of Non-Destructive Testing	Typical Testing Frequency
Exceptional performance	Design Construction Materials	GPR, FWD GPR, FWD, nuclear gauge GPR, FWD	- GPR: continuous (2 scans/yd [m]) - FWD: 75 ft (25 m) intervals, 3 to 15 ft (1 to 5 m) in defined problem areas, offset in adjacent lanes
Rutting	Asphalt densification Asphalt shearing Base, subbase or subgrade failure Stabilization failure Insufficient layer thickness Moisture damage Poor compaction Incorrect binder Inappropriate or not followed mix design	Nuclear gauge[1] Transverse profilometer/ straightedge FWD, drain inspection FWD GPR, FWD, nuclear gauge[1] GPR, FWD, nuclear gauge[1] GPR, nuclear gauge[1] Not appropriate Not appropriate	- GPR: continuous (2 scans/m) - FWD: 75 ft (25 m) intervals, 3 to 15 ft (1 to 5 m) in defined problem areas - Nuclear gauge: per state test method - Transverse profilometer/straightedge in defined problem areas
Alligator cracking	Base, subbase or subgrade failure Moisture damage Layer debonding Thickness, compaction Incorrect binder Excessive binder aging Inappropriate or not followed mix design	FWD, drain inspection GPR, FWD GPR, FWD GPR, nuclear gauge[1] Not appropriate Not appropriate Not appropriate	- GPR: continuous (20 scans/yd [m]) - FWD: 75 ft (25 m) intervals, 3 to 15 ft (1 to 5 m) in defined problem areas - Nuclear gauge: per state test method
Transverse cracking	Compaction Incorrect binder Reflection cracking Shrinkage in stabilized base Frost/moisture damage in unbound layer	Nuclear gauge[1] Not appropriate Not appropriate (GPR in some situations)[2] Not appropriate (GPR in some situations) Drain inspection	- Nuclear gauge: per state test method
Longitudinal cracking	Base, subbase or subgrade failure Moisture damage Construction joint compaction Shoulder design and construction Excessive stabilizer in recycling overlaps Stabilization failure	GPR, FWD, drain inspection GPR, FWD Nuclear gauge[1] GPR, FWD GPR Not appropriate	- GPR: continuous (2 scans/yd [m]) - FWD: 75 ft (25 m) intervals, 3 to 15 ft (1 to 5 m) in defined problem areas - Nuclear gauge: per state test method
Block cracking	Shrinkage in stabilized base Binder properties (burning or rapid aging)	Not appropriate (GPR in some situations) Not appropriate	
Ride quality/roughness	Constructed ride quality Cracks Potholes Large aggregates Raveling	Profilometer[3] Profilometer[3] Profilometer[3] Profilometer[3] Not appropriate	- Profilometer continuous (use measurement from between wheelpaths to determine initial IRI)
Surface failure/potholes	Moisture damage Delamination Shoulder design and construction Poor cross slope	GPR, FWD GPR, FWD GPR, FWD, drain inspection Survey or measure with level	- GPR: continuous (20 scans/yd [m], multiple scans) - FWD: 75 ft (25 m) intervals, 3 to 15 ft (1 to 5 m) in defined problem areas - 15 ft (5 m) intervals in affected area
Excessive noise	Mix design Raveling Cracking Clogging of porous surface	OBSI OBSI, longitudinal profilometer[3], laser texture meter[4] OBSI Permeameter	- OBSI: continuous - Profilometer: continuous - Laser texture meter: affected area - Permeameter: In and between wheelpaths
Frictional characteristics	Polished aggregate Flushing/bleeding	Friction tester, texture meter Friction tester	- Friction tester: continuous - Texture meter: affected area

[1] Take at least three cores of each material to determine TMD per ASTM D2041 or AASHTO equivalent. Take nuclear gauge measurements between the wheelpaths and in the wheelpath to determine construction compaction and extent of densification, minimum 10 in problem areas.
[2] GPR may be used to identify the source of reflective or shrinkage cracks deep within a pavement structure.
[3] Profilometer can potentially be run between the wheelpaths to estimate as-constructed ride quality.
[4] Mean profile depth can be measured using a high-speed longitudinal profilometer on a test vehicle requiring no closure or by a stationary laser texture meter in a traffic closure

Table D.4. Example NDT Intervals.

Test	Interval	Test Duration/ Lane-mile[1]	Road Closure Required?
GPR – General layer thickness/layer definition	Continuous (2 scans/m)	2 minutes	No[2]
GPR – Asphalt densification	Continuous	2 minutes	No
GPR – Problem identification/delineation on AC	Continuous (20 scans/m)	90 minutes	Yes
GPR – Problem identification on PCC	Joint/joint area/crack	-	Yes
GPR – Void location	Suspected area	-	Yes
FWD – Problem delineation on AC pavement	80 ft (25 m)[3]	90 minutes	Yes
FWD – Specific problem investigation on AC	30 ft (10 m)	225 minutes	Yes
FWD – Problem delineation on PCC pavement	Not appropriate	-	-
FWD – Specific problem investigation on PCC	Joint/crack/slab center	50 drops/hour	Yes
Profilometer – Overall smoothness	Continuous	2 minutes	No
Friction tester – Skid resistance	Continuous	2 minutes	No
OBSI – Noise levels	Continuous	2 minutes	No

1 Test duration does not include closure set up and take down.
2 A limited number of cores are required for calibration. A road closure is required for coring.
3 Longer test intervals can be adopted if there are constraints such as traffic or limited closure schedules; however, this increases the risk of missing weaker sections. A second round of testing with closer intervals (e.g. 30 ft [10 m]) may be required to test specific problem areas.

Table D.5. Example Modulus Ranges for Different Layer Types.

Layer Type	Modulus Range[1]			
	Lower Bound		Upper Bound	
	psi	MPa	psi	MPa
Portland cement concrete	2,200,000	15,000	7,000,000	50,000
Asphalt concrete	100,000	700	1,000,000	7,000
FDR[2] + cement	80,000	550	800,000	5,500
FDR + foamed asphalt	50,000	350	600,000	4,100
FDR + asphalt emulsion	50,000	350	600,000	4,100
FDR/no stabilizer	40,000	275	150,000	1,035
PDR[3] + emulsion	80,000	550	800,000	5,500
Asphalt-treated base	100,000	700	900,000	6,750
Asphalt emulsion base	50,000	350	500,000	3,500
Cement treated base[4]	-	-	-	-
Lean concrete base	1,500,000	10,000	5,500,000	40,000
Aggregate base	15,000	105	50,000	350
Granular subgrade	10,000	70	50,000	350
Fine-grained subgrade	5,000	35	50,000	350

1 Ranges are highly dependent on test temperatures.
2 Full-depth reclaimed.
3 Partial-depth reclaimed/cold in-place.
4 Modulus range depends on the level of cracking.

Table D.7. Field Testing Activities for Collecting Supplemental Data.

Field Activity	Pavement Type	Purpose
Core at crack	All	Visual determination of crack origin
Core at saw and seal locations	AC/JCP	Locate joint relative to saw cut
Core at reflection cracks	AC/JCP, AC/CRCP	Determine PCC joint versus crack
Trench	AC, AC/AC	Detailed layering study; determine location of any permanent deformation
Test pits	JCP, CRCP	Sampling and in situ density and moisture testing
GPR	All	Layer thickness, material condition
SASW	All	Layer thickness, voids under PCC
Pachometer	JCP, CRCP	Location and depth of steel, dowel, and reinforcement
Drainage evaluation	All	Excavate and assess condition of drainage elements (laterals, collectors, filter materials)
Dipstick	All	Cross slope

Table D.8. Tests for End-State Physical Properties of Pavement Materials

Sites	Laboratory Test
All forensic	
From bulk of unbound base	
	Moisture-density relationship
	Resilient modulus
	Engineering properties (Atterberg limits, gradation)
	Specific gravity
From bulk of subgrade	
	Moisture-density relationship
	Resilient modulus
	Engineering properties (Atterberg limits, gradation)
AC and AC/AC	
From cores/bulk samples of AC	
	Mix properties, density, voids, AC content
	Resilient modulus
	SHRP properties from SST
	PG of extracted/recovered AC
	Core inspection for stripping
JCP and CRCP	
From cores	
	Compressive strength
	Splitting tensile strength
	Elastic modulus
	Inspect core for corrosion
	Inspect core for ASR

Note: ASR = alkali-silica reaction, PG = performance grade, SST = Superpave® Shear Tester.

Figure D.1. Core Locations for LTPP-AC Surfaced Test Section

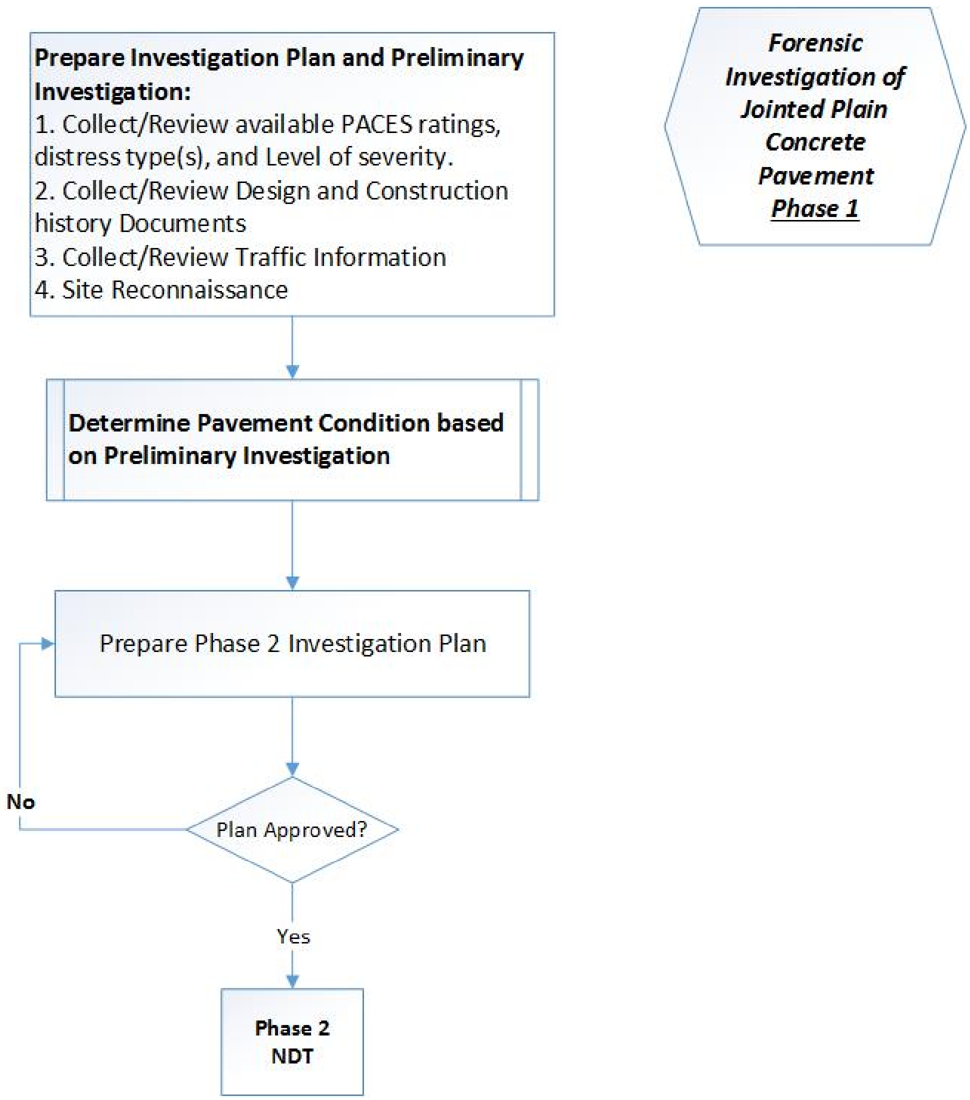
Figure D.2. Crack Core Locations for LTPP AC Test Section

Figure D.3. Trench Locations for LTPP Test Section

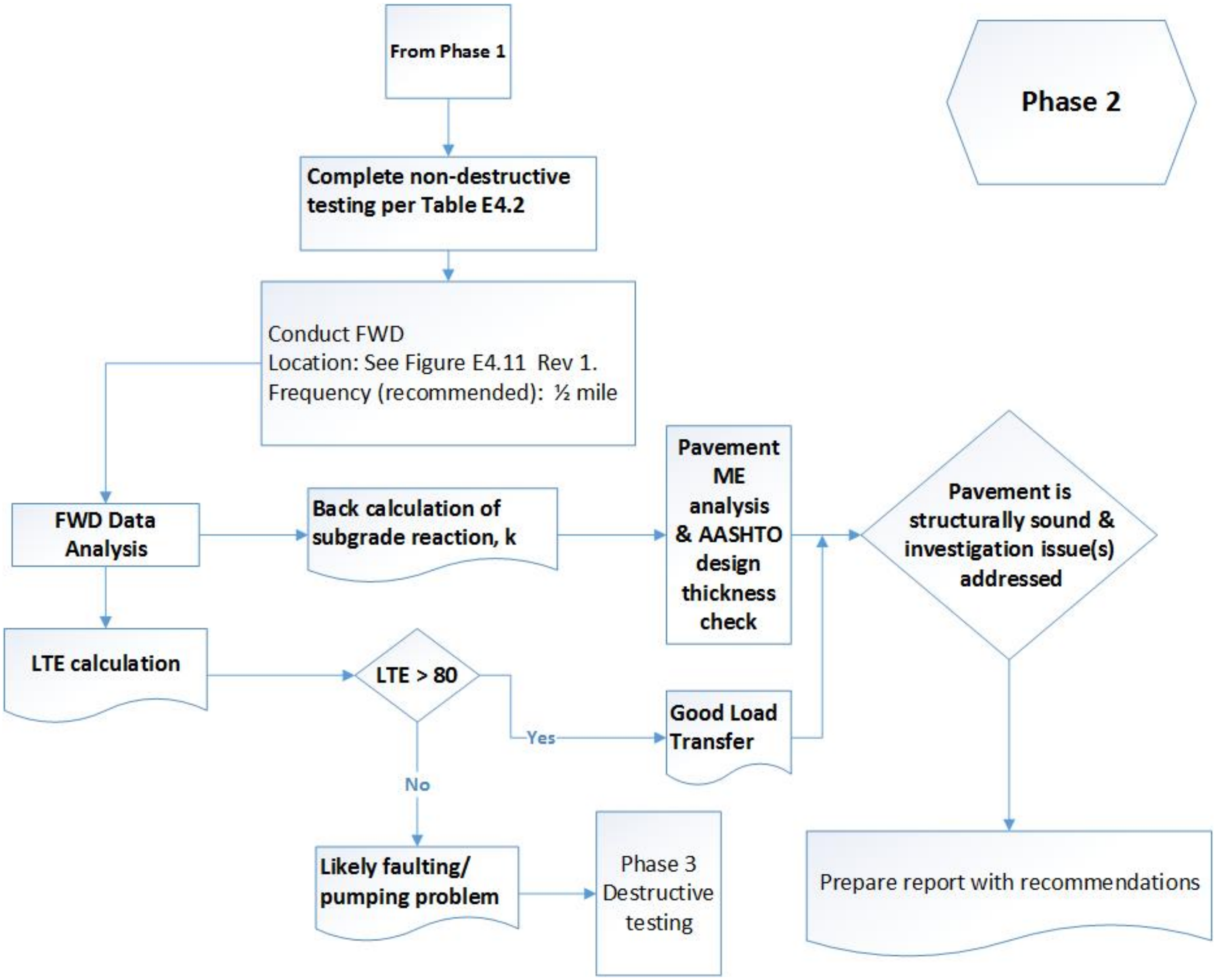
Figure D.4. Dynamic Cone Penetration, Falling Weight Deflectometer, Nuclear Density Gauge, and Saw-Cut Locations

APPENDIX E

Flow Chart for Forensic Investigation of Joint Plain Concrete Pavement



Forensic Investigation of Jointed Plain Concrete Pavement Phase 1
Prepare Investigation Plan and Preliminary Investigation:
1. Collect/Review available PACES ratings, distress type(s), and Level of severity.
2. Collect/Review Design and Construction history Documents
3. Collect/Review Traffic Information
4. Site Reconnaissance
Determine Pavement Condition based on Preliminary Investigation
Prepare Phase 2 Investigation Plan
Plan Approved?
No
Yes
Phase 2 NDT


Phase 2
From Phase 1
Complete non-destructive testing per Table E4.2
Conduct FWD
Location: See Figure E4.11 Rev 1.
Frequency (recommended): ½ mile
FWD Data Analysis
Back calculation of subgrade reaction, k
Pavement ME analysis & AASHTO design thickness check
Pavement is structurally sound & investigation issue(s) addressed
LTE calculation
LTE > 80
Yes
Good Load Transfer
No
Likely faulting/ pumping problem
Phase 3 Destructive testing
Prepare report with recommendations

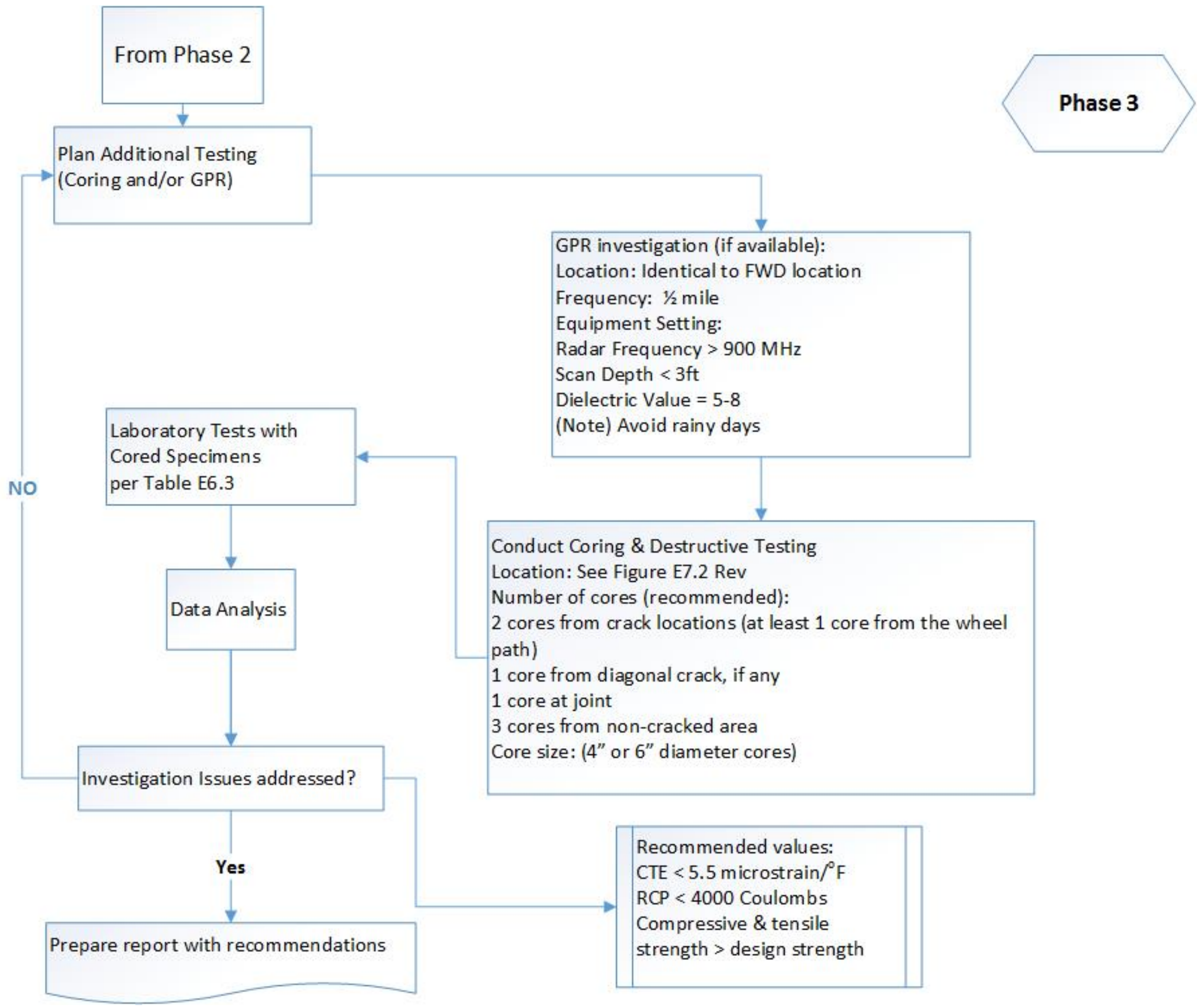

Phase 3
From Phase 2
Plan Additional Testing (Coring and/or GPR)
GPR investigation (if available):
Location: Identical to FWD location
Frequency: ½ mile
Equipment Setting:
Radar Frequency > 900 MHz
Scan Depth < 3ft
Dielectric Value = 5-8
(Note) Avoid rainy days
Laboratory Tests with Cored Specimens per Table E6.3
NO
Conduct Coring & Destructive Testing
Location: See Figure E7.2 Rev
Number of cores (recommended):
2 cores from crack locations (at least 1 core from the wheel path)
1 core from diagonal crack, if any
1 core at joint
3 cores from non-cracked area
Core size: (4" or 6" diameter cores)
Data Analysis
Investigation Issues addressed?
Yes
Prepare report with recommendations
Recommended values:
CTE < 5.5 microstrain/°F
RCP < 4000 Coulombs
Compressive & tensile strength > design strength

Table E4.2. Examples of NDT on Concrete Surfaced Pavements.

Issue	Possible Contributing Factors	Type of Non-Destructive Testing	Typical Testing Frequency
Exceptional performance	Design Construction Materials	GPR[1], FWD[1] Profilometer, OBSI GPR, FWD	- GPR: continuous (2 scans/yd [m]) - FWD: corners, mid-slab, mid-joint - Profilometer: continuous
Corner cracking	Voids Load transfer Temperature and shrinkage curl Concrete stiffness Dowel failure or absence of dowels	GPR[1] FWD[3] Profilometer[2] FWD[4], SPA MTT scan	- GPR: continuous (slow speed, 20 scans/m) - FWD: corners - FWD,SPA: slab center - Profilometer: continuous - MTT scan: affected area
D-cracking	Materials or moisture/frost damage	Not appropriate	
Longitudinal cracking	Base or subgrade failure/stabilization cracks Temperature and shrinkage curl Concrete stiffness	FWD FWD[3,4] FWD[4], SPA	- FWD: either side of crack - FWD: corners - FWD: slab center - SPA: slab center
Transverse cracking	Edge support Load transfer on tied shoulders Swelling soils/frost heave	GPR[1] FWD Not appropriate	- GPR: continuous (20 scans/yd [m]) - FWD: both sides of joint
Early age cracks	Improper curing, late sawing	Not appropriate	
Faulting	Erosion/pumping Load transfer	GPR[1] FWD	- GPR: continuous - FWD: corners and joints
Spalling	Construction/maintenance deficiencies Frost	Not appropriate Not appropriate	
Joint failure/separation	Design/construction/maintenance deficiencies Dowel bar failure/seizure from misalignment	Not appropriate MTT scan	- MTT scan: affected area
Pumping	Load transfer Base erosion	FWD[3] FWD	- FWD: joints and corners - FWD: corners
Punchouts	Base failure and/or subbase erosion	GPR[1]	- GPR: continuous (20 scans/yd [m])
Ride quality/roughness/ Settlement	Construction deficiencies Faulting Moisture/frost Support (voids)	Profilometer[2] Profilometer[2] GPR, FWD GPR	- Profilometer: continuous - GPR: continuous (slow speed, 20 scans/yd [m], multiple profiles) - FWD: joints and corners
Excessive noise	Surface texture from construction, grinding, grooving, fines loss	OBSI Texture meter	- OBSI: continuous - Texture meter: affected area
Poor skid resistance	Polished aggregate Poor surface texture from construction, grinding, grooving	Friction tester Friction tester, texture meter	- Friction tester: continuous - Texture meter: affected area

[1] GPR and FWD may not be appropriate on CRCP as steel reinforcement attenuates the signal.
[2] A wide spot or bar laser is needed for effective road roughness measurements on tined or grooved concrete. Texture measurements with a standard laser profilometer are not effective, and a texture meter should be used if these lasers are not available.
[3] FWD for estimating load transfer.
[4] FWD for backcalculation of stiffness.

Table E4.3. Example NDT Intervals.

Test	Interval	Test Duration/ Lane-mile[1]	Road Closure Required?
GPR – General layer thickness/layer definition	Continuous (2 scans/m)	2 minutes	No[2]
GPR – Asphalt densification	Continuous	2 minutes	No
GPR – Problem identification/delineation on AC	Continuous (20 scans/m)	90 minutes	Yes
GPR – Problem identification on PCC	Joint/joint area/crack	-	Yes
GPR – Void location	Suspected area	-	Yes
FWD – Problem delineation on AC pavement	80 ft (25 m)[3]	90 minutes	Yes
FWD – Specific problem investigation on AC	30 ft (10 m)	225 minutes	Yes
FWD – Problem delineation on PCC pavement	Not appropriate	-	-
FWD – Specific problem investigation on PCC	Joint/crack/slab center	50 drops/hour	Yes
Profilometer – Overall smoothness	Continuous	2 minutes	No
Friction tester – Skid resistance	Continuous	2 minutes	No
OBSI – Noise levels	Continuous	2 minutes	No

[1] Test duration does not include closure set up and take down.
[2] A limited number of cores are required for calibration. A road closure is required for coring.
[3] Longer test intervals can be adopted if there are constraints such as traffic or limited closure schedules; however, this increases the risk of missing weaker sections. A second round of testing with closer intervals (e.g. 30 ft [10 m]) may be required to test specific problem areas.

Table E6.3. Examples of Laboratory Testing Requirements for Concrete Pavement Investigations

Issue	Possible Contributing Factors	Example Types of Laboratory Testing
Exceptional performance[1]	Design, construction, and/or materials	- Any combination of tests below depending on specific issue being investigated
Corner cracking (JPCP, JRCP)	Low PCC strength Load transfer (joint or edge)	- PCC: Compressive strength, splitting tensile strength, chloride content of concrete near dowel/tie bar - Dowel: Dowel/tie bar coating type
D-cracking[1]	Susceptible aggregate Poor drainage	- PCC: Aggregate analysis (sedimentary with high fine pore content), freeze-thaw
Longitudinal cracking[1]	Low PCC strength High coefficient of thermal expansion (CTE) Warping or curling stresses (high CTE) Poor load transfer to tied shoulder	- PCC: Compressive strength, splitting tensile strength, CTE, chloride content of concrete near dowel/tie bar
Transverse cracking (JPCP)	Low PCC strength High CTE Tied shoulder load transfer	- PCC: Compressive strength, splitting tensile strength, CTE, chloride content of concrete near dowel/tie bar - Dowel: Dowel/tie bar coating type
Map cracking[1]	Alkali-silica reaction (ASR)	- ASR tests
Faulting (JPCP, JRCP)	Load transfer (dowel corrosion, looseness, misplacement, incorrect size) Erosion/pumping	- Dowel bar coating type
Dowel bar retrofit failure (JPCP)	Low grout strength Poor bonding of grout to slab Dowel bar corrosion	- PCC: Chloride content of concrete near dowel/tie bar - Grout: Grout strength, compressive strength, splitting tensile strength, Grout/PCC bond strength - Dowel: Dowel/tie bar coating type
Spalling[1]	Poor finishing Weak aggregate Frost	- PCC: Aggregate petrography
Joint failure/separation (JPCP, JRCP)	Dowel bar failure/seizure	- PCC: Chloride content of concrete near dowel
Punchouts (CRCP) (See longitudinal cracking for preceding mechanism)	Low PCC strength Steel reinforcement corrosion	- PCC: Compressive strength, splitting tensile strength, CTE, chloride content of concrete near rebar
Excessive noise[1]	Poor texture from construction or grinding/grooving. Faulting, wide joint openings, spalled joints Chain or studded tire damage	- PCC: Laboratory texture tests
Skid resistance[1]	Poor surface texture from construction, grinding, grooving Polished aggregate, loss of texture	- PCC: Laboratory texture tests, aggregate classification, polishing tests

[1] All types of PCC pavement.

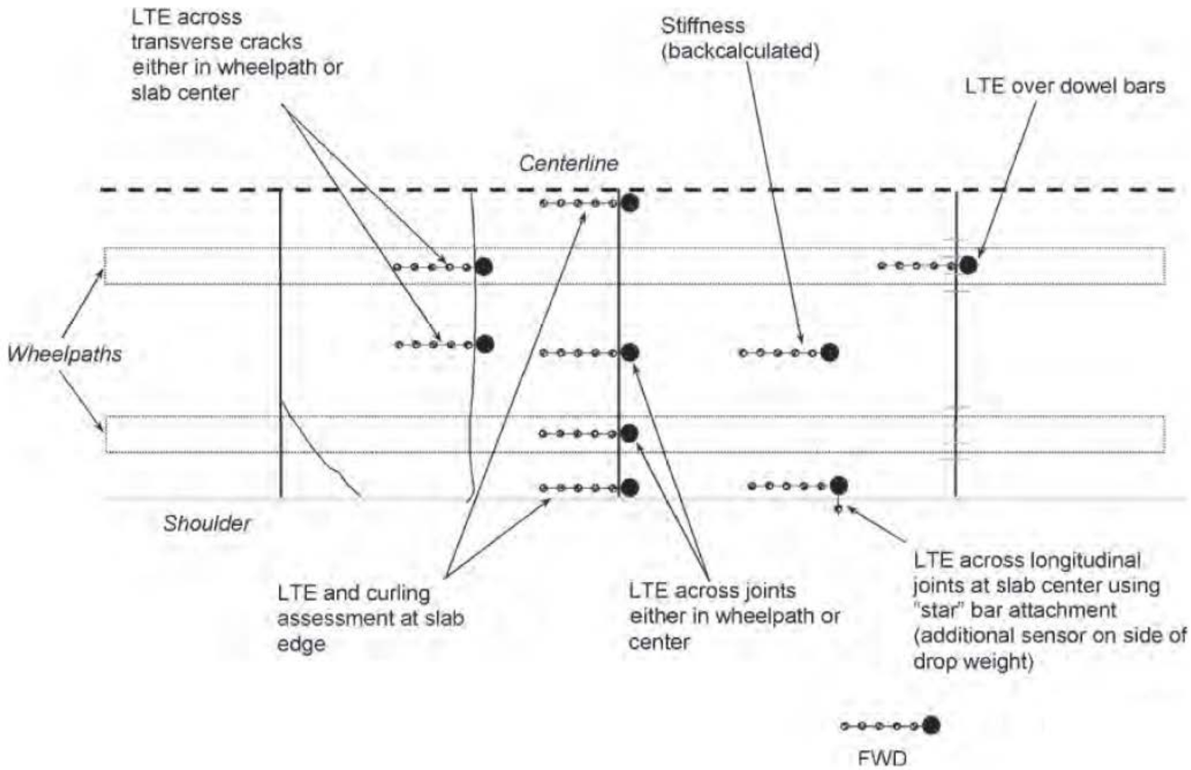


Figure E4.11(a). Example FWD Test Locations on Jointed Plain Concrete Pavements.

Notes: On Concrete pavements, test location on the slab will depend on the issues being investigated. Load transfer efficiency is measured across the joints in the wheel paths, stiffness is measured in the center of the slab, and curling is measured across the joint at the slab corners. Example test locations for jointed concrete pavements are shown in Figure 4.11

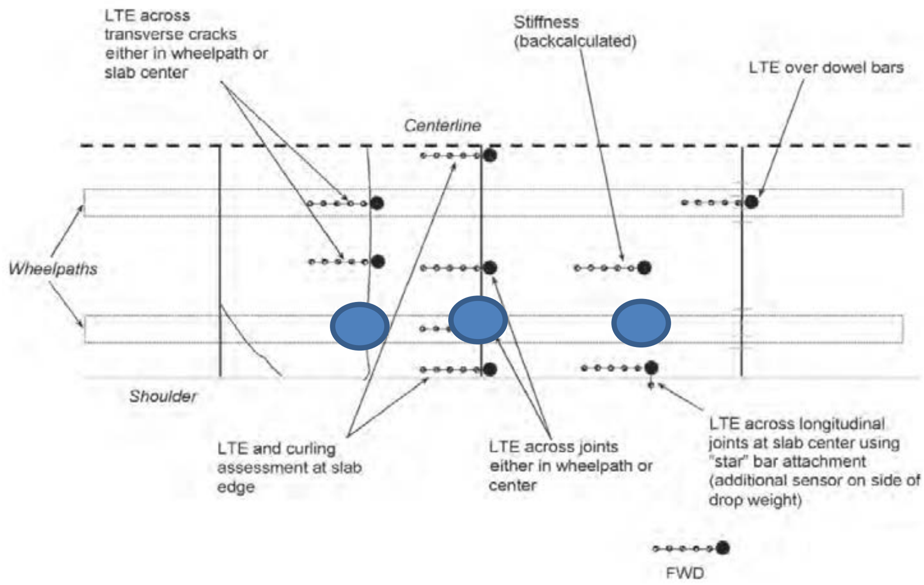


Figure E4.11(b). Example FWD Test Locations on Jointed Plain Concrete Pavements.

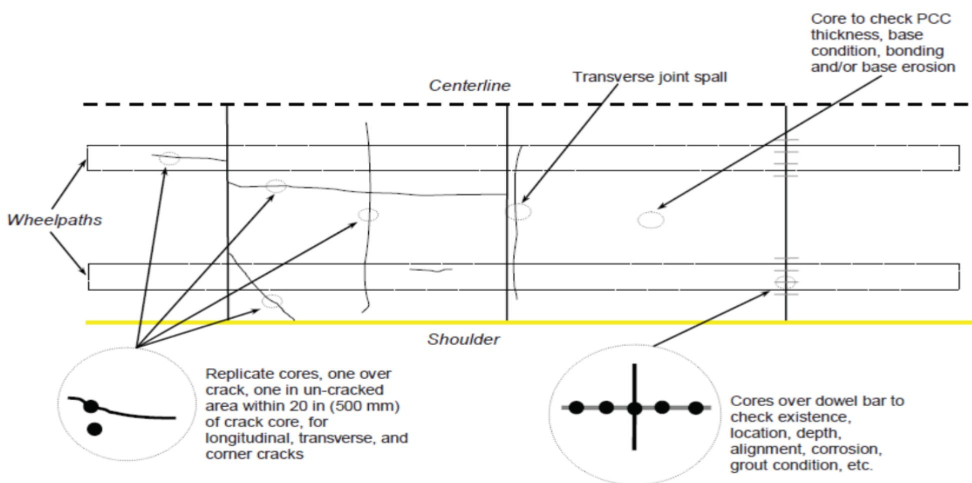
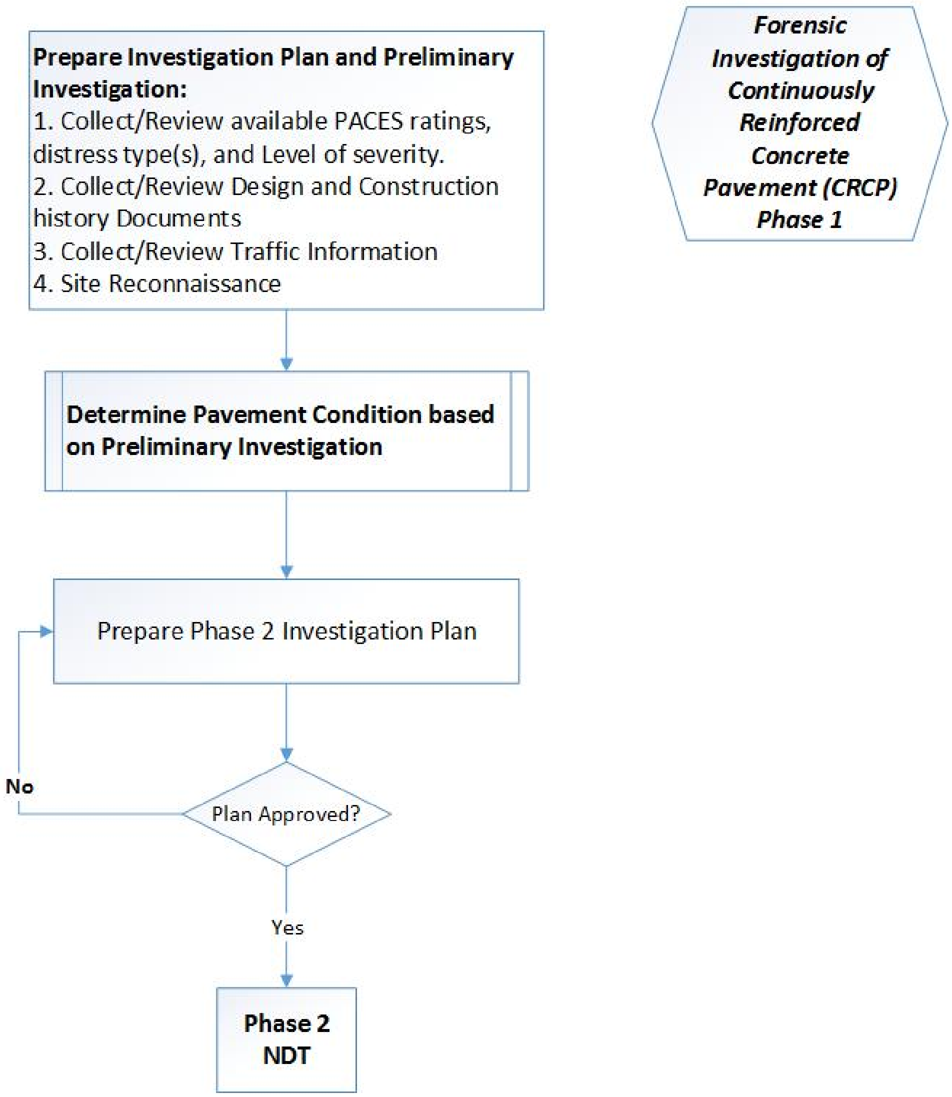
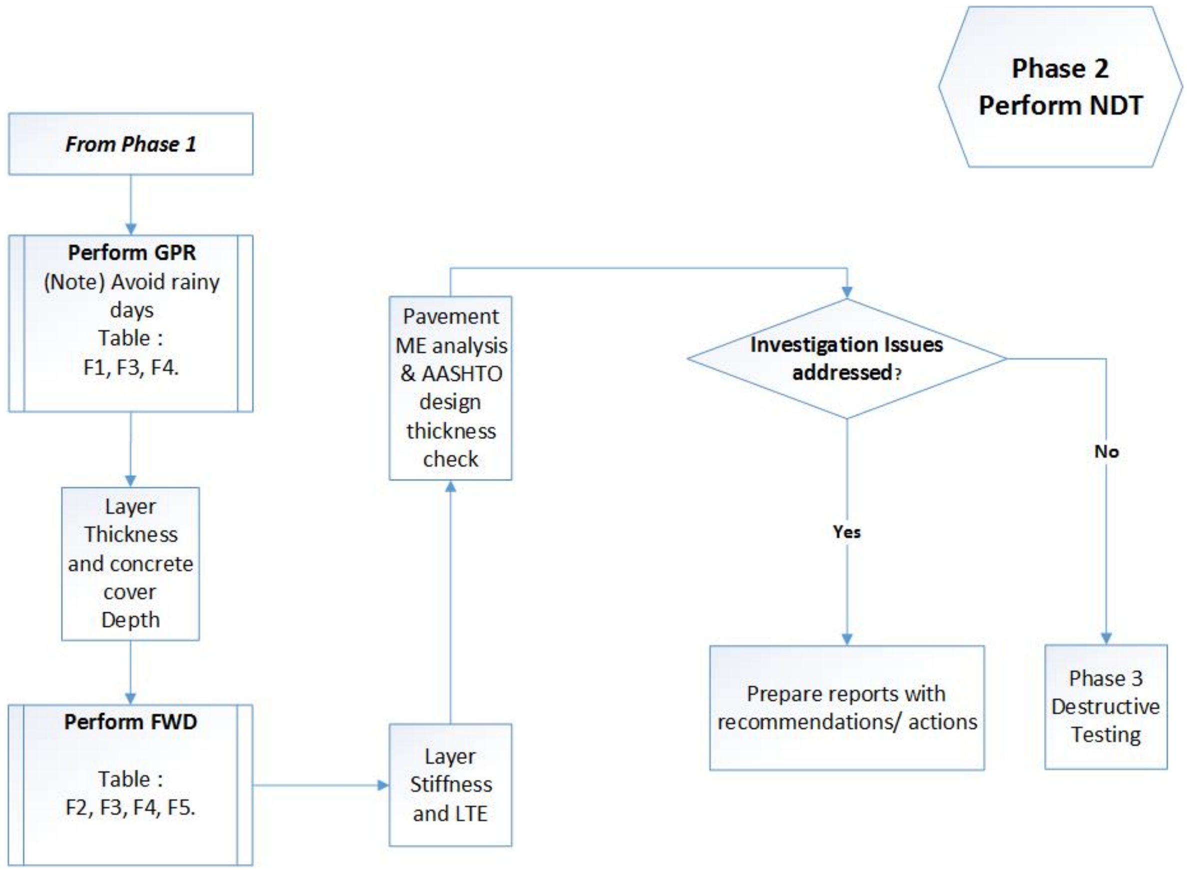


Figure E7.2. Examples of Core Locations for Jointed Plain Concrete Sections.

APPENDIX F

Flow Chart for Forensic Investigation of Continuous Reinforced Concrete Pavement


Forensic Investigation of Continuously Reinforced Concrete Pavement (CRCP) Phase 1
Prepare Investigation Plan and Preliminary Investigation:
1. Collect/Review available PACES ratings, distress type(s), and Level of severity.
2. Collect/Review Design and Construction history Documents
3. Collect/Review Traffic Information
4. Site Reconnaissance
Determine Pavement Condition based on Preliminary Investigation
Prepare Phase 2 Investigation Plan
Plan Approved?
No
Yes
Phase 2 NDT


Phase 2
Perform NDT
From Phase 1
Perform GPR
(Note) Avoid rainy days
Table :
F1, F3, F4.
Layer Thickness and concrete cover Depth
Perform FWD
Table :
F2, F3, F4, F5.
Layer Stiffness and LTE
Pavement ME analysis & AASHTO design thickness check
Investigation Issues addressed?
Yes
No
Prepare reports with recommendations/ actions
Phase 3
Destructive Testing

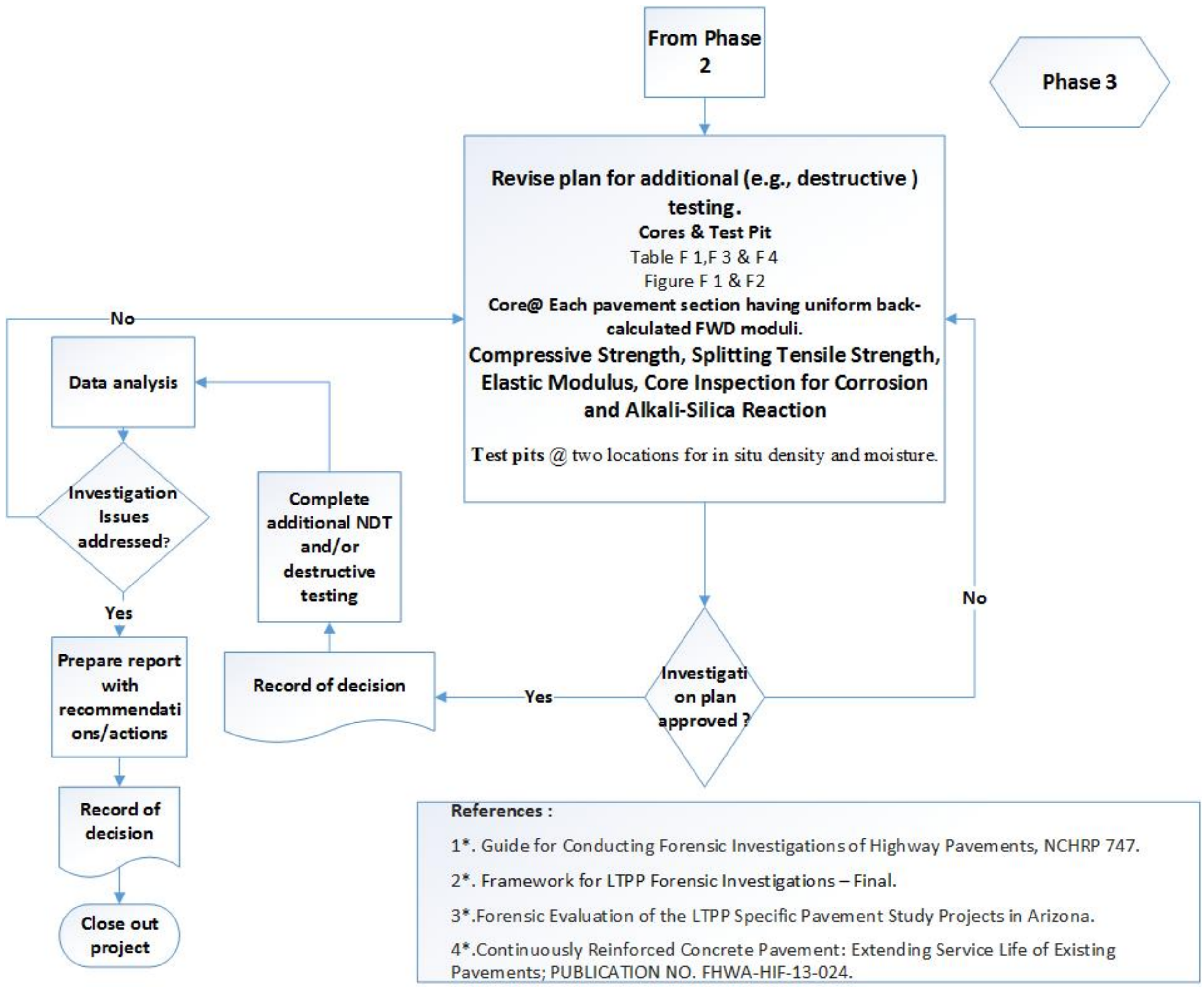

From Phase 2
Phase 3
Revise plan for additional (e.g., destructive) testing.
Cores & Test Pit
Table F 1,F 3 & F 4
Figure F 1 & F2
Core@ Each pavement section having uniform back-calculated FWD moduli.
Compressive Strength, Splitting Tensile Strength, Elastic Modulus, Core Inspection for Corrosion and Alkali-Silica Reaction
Test pits @ two locations for in situ density and moisture.
No
Data analysis
Investigation Issues addressed?
Yes
Prepare report with recommendations/actions
Record of decision
Close out project
Complete additional NDT and/or destructive testing
Record of decision
Yes
Investigation plan approved ?
No
References :
1*. Guide for Conducting Forensic Investigations of Highway Pavements, NCHRP 747.
2*. Framework for LTPP Forensic Investigations – Final.
3*.Forensic Evaluation of the LTPP Specific Pavement Study Projects in Arizona.
4*.Continuously Reinforced Concrete Pavement: Extending Service Life of Existing Pavements; PUBLICATION NO. FHWA-HIF-13-024.

Table F.1. GPR Performing Details.

GPR	
Location	Identical to FWD location
Frequency	2 Scans/Mile (for thickness and concrete cover depth)
	20 Scans/mile (for punchout area)
Radar Frequency	> 900 MHz
Scan Depth	< 3ft
Dielectric Value	3 to8.

Table F.2. FWD Performing Details.

FWD	
Location and interval	50 ft., if Section Length < 2 miles
	100 ft., if : 2 miles <Section length < 4 miles
	250 ft., if :Section Length > 4 miles
	3 to 15 ft. (1 to 5 m) in defined problem areas; offset in adjacent lanes
The surface temp.	< 77F (23C)
LTE of CRCP cracks	LTE must be greater than 75 percent.

Table F.3. NDT on Concrete Surfaced Pavements

Issue	Possible Contributing Factors	Type of Non-Destructive Testing	Typical Testing Frequency
Exceptional performance	Design Construction Materials	GPR[1], FWD[1] Profilometer, OBSI GPR, FWD	- GPR: continuous (2 scans/yd [m]) - FWD: corners, mid-slab, mid-joint - Profilometer: continuous
Corner cracking	Voids Load transfer Temperature and shrinkage curl Concrete stiffness Dowel failure or absence of dowels	GPR[1] FWD[3] Profilometer[2] FWD[4], SPA MTT scan	- GPR: continuous (slow speed, 20 scans/m) - FWD: corners - FWD,SPA: slab center - Profilometer: continuous - MTT scan: affected area
D-cracking	Materials or moisture/frost damage	Not appropriate	
Longitudinal cracking	Base or subgrade failure/stabilization cracks Temperature and shrinkage curl Concrete stiffness	FWD FWD[3,4] FWD[4], SPA	- FWD: either side of crack - FWD: corners - FWD: slab center - SPA: slab center
Transverse cracking	Edge support Load transfer on tied shoulders Swelling soils/frost heave	GPR[1] FWD Not appropriate	- GPR: continuous (20 scans/yd [m]) - FWD: both sides of joint
Early age cracks	Improper curing, late sawing	Not appropriate	
Faulting	Erosion/pumping Load transfer	GPR[1] FWD	- GPR: continuous - FWD: corners and joints
Spalling	Construction/maintenance deficiencies Frost	Not appropriate Not appropriate	
Joint failure/separation	Design/construction/maintenance deficiencies Dowel bar failure/seizure from misalignment	Not appropriate MTT scan	- MTT scan: affected area
Pumping	Load transfer Base erosion	FWD[3] FWD	- FWD: joints and corners - FWD: corners
Punchouts	Base failure and/or subbase erosion	GPR[1]	- GPR: continuous (20 scans/yd [m])
Ride quality/roughness/ Settlement	Construction deficiencies Faulting Moisture/frost Support (voids)	Profilometer[2] Profilometer[2] GPR, FWD GPR	- Profilometer: continuous - GPR: continuous (slow speed, 20 scans/yd [m], multiple profiles) - FWD: joints and corners
Excessive noise	Surface texture from construction, grinding, grooving, fines loss	OBSI Texture meter	- OBSI: continuous - Texture meter: affected area
Poor skid resistance	Polished aggregate Poor surface texture from construction, grinding, grooving	Friction tester Friction tester, texture meter	- Friction tester: continuous - Texture meter: affected area

[1] GPR and FWD may not be appropriate on CRCP as steel reinforcement attenuates the signal.
[2] A wide spot or bar laser is needed for effective road roughness measurements on tined or grooved concrete. Texture measurements with a standard laser profilometer are not effective, and a texture meter should be used if these lasers are not available.
[3] FWD for estimating load transfer.
[4] FWD for backcalculation of stiffness.

Table F.4. Example NDT Intervals.

Test	Interval	Test Duration/ Lane-mile[1]	Road Closure Required?
GPR – General layer thickness/layer definition	Continuous (2 scans/m)	2 minutes	No[2]
GPR – Asphalt densification	Continuous	2 minutes	No
GPR – Problem identification/delineation on AC	Continuous (20 scans/m)	90 minutes	Yes
GPR – Problem identification on PCC	Joint/joint area/crack	-	Yes
GPR – Void location	Suspected area	-	Yes
FWD – Problem delineation on AC pavement	80 ft (25 m)[3]	90 minutes	Yes
FWD – Specific problem investigation on AC	30 ft (10 m)	225 minutes	Yes
FWD – Problem delineation on PCC pavement	Not appropriate	-	-
FWD – Specific problem investigation on PCC	Joint/crack/slab center	50 drops/hour	Yes
Profilometer – Overall smoothness	Continuous	2 minutes	No
Friction tester – Skid resistance	Continuous	2 minutes	No
OBSI – Noise levels	Continuous	2 minutes	No

[1] Test duration does not include closure set up and take down.
[2] A limited number of cores are required for calibration. A road closure is required for coring.
[3] Longer test intervals can be adopted if there are constraints such as traffic or limited closure schedules; however, this increases the risk of missing weaker sections. A second round of testing with closer intervals (e.g. 30 ft [10 m]) may be required to test specific problem areas.

Table F.5. Example Modulus Ranges for Different Layer Types

Layer Type	Modulus Range[1]			
	Lower Bound		Upper Bound	
	psi	MPa	psi	MPa
Portland cement concrete	2,200,000	15,000	7,000,000	50,000
Asphalt concrete	100,000	700	1,000,000	7,000
FDR[2] + cement	80,000	550	800,000	5,500
FDR + foamed asphalt	50,000	350	600,000	4,100
FDR + asphalt emulsion	50,000	350	600,000	4,100
FDR/no stabilizer	40,000	275	150,000	1,035
PDR[3] + emulsion	80,000	550	800,000	5,500
Asphalt-treated base	100,000	700	900,000	6,750
Asphalt emulsion base	50,000	350	500,000	3,500
Cement treated base[4]	-	-	-	-
Lean concrete base	1,500,000	10,000	5,500,000	40,000
Aggregate base	15,000	105	50,000	350
Granular subgrade	10,000	70	50,000	350
Fine-grained subgrade	5,000	35	50,000	350

[1] Ranges are highly dependent on test temperatures.
[2] Full-depth reclaimed.
[3] Partial-depth reclaimed/cold in-place.
[4] Modulus range depends on the level of cracking.

Table F.7. Field Testing Activities for Collecting Supplemental Data

Field Activity	Pavement Type	Purpose
Core at crack	All	Visual determination of crack origin
Core at saw and seal locations	AC/JCP	Locate joint relative to saw cut
Core at reflection cracks	AC/JCP, AC/CRCP	Determine PCC joint versus crack
Trench	AC, AC/AC	Detailed layering study; determine location of any permanent deformation
Test pits	JCP, CRCP	Sampling and in situ density and moisture testing
GPR	All	Layer thickness, material condition
SASW	All	Layer thickness, voids under PCC
Pachometer	JCP, CRCP	Location and depth of steel, dowel, and reinforcement
Drainage evaluation	All	Excavate and assess condition of drainage elements (laterals, collectors, filter materials)
Dipstick	All	Cross slope

Table F.7. Field Testing Activities for Collecting Supplemental Data

Sites	Laboratory Test
All forensic	
From bulk of unbound base	
	Moisture-density relationship
	Resilient modulus
	Engineering properties (Atterberg limits, gradation)
	Specific gravity
From bulk of subgrade	
	Moisture-density relationship
	Resilient modulus
	Engineering properties (Atterberg limits, gradation)
AC and AC/AC	
From cores/bulk samples of AC	
	Mix properties, density, voids, AC content
	Resilient modulus
	SHRP properties from SST
	PG of extracted/recovered AC
	Core inspection for stripping
JCP and CRCP	
From cores	
	Compressive strength
	Splitting tensile strength
	Elastic modulus
	Inspect core for corrosion
	Inspect core for ASR

Note: ASR = alkali-silica reaction, PG = performance grade, SST = Superpave® Shear Tester.

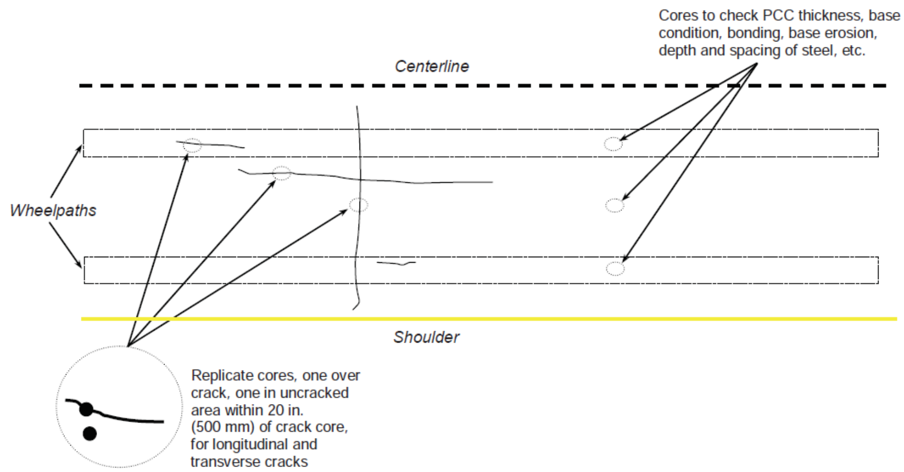


Figure F.1. Core Locations for Continuously Reinforced Concrete Sections.

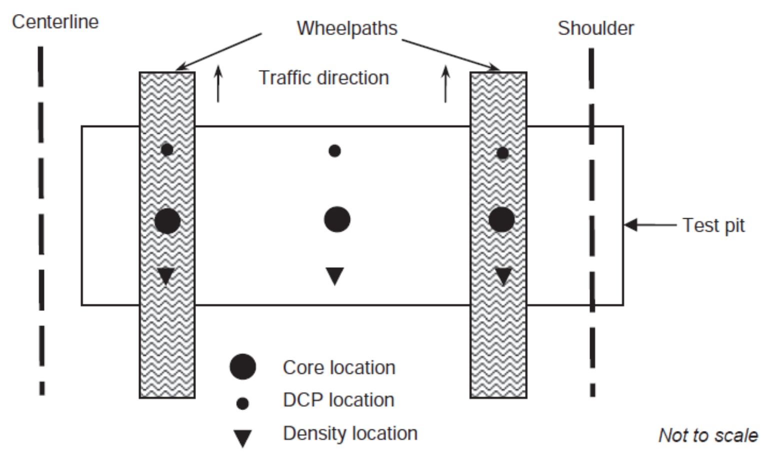


Figure F.2. Test Pit Layout.