AN INVESTIGATION OF DISTRESSES FOUND IN CONCRETE AND ASPHALT PAVEMENTS FOR GEORGIA FORENSIC GUIDE RECOMMENDATION

by

CATHERINE ELIZABETH JOHNSON

(Under the Direction of Mi Geum Chorzepa)

ABSTRACT

This thesis 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. The evaluation of three pavement types using the NCHRP 747 guide is completed: Jointed Plain Concrete (JPC), Continuously Reinforced Concrete (CRC), and Hot Mix Asphalt (HMA). Each type consisted of an evaluation of two sites in "good/fair" and "poor" conditions. Non-destructive testing was performed using a Ground Penetration Radar (GPR) and Falling Weight Deflectometer (FWD). Destructive and on-site field testing were performed consistently with the recommendations of the guide. Laboratory tests were conducted to determine material properties and combined with traffic data to form conclusions about the causes of pavement distress. It is recommended from this study that the GDOT adopts the NCHRP Report 747 for use in Georgia.

INDEX WORDS: Forensic Investigation, Pavement, Ground Penetration Radar, Falling Weight Deflectometer, Jointed Plain Concrete, Continuously Reinforced

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CHAPTER 1

INTRODUCTION

State agencies and civil engineers must remain current in forensic pavement investigation procedures, inspection technologies, and rehabilitation methods to efficiently maintain public roads. Preliminary pavement investigation procedures typically consist of visual condition surveys and subjective observations. When visual inspections indicate a functional or structural problem may exist or is developing in a pavement system, a more refined investigation is necessary. A forensic pavement investigation consists of using non-destructive and destructive testing methods, as well as locally documented pavement performance rating data, to provide valuable information and further determine the extent of a functional or structural problem.

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. However, without an on-going and well-structured forensic pavement investigation program, unwanted downtime and loss of money will be inevitable to mitigate neglected problems (Rens et al. 1997). Despite numerous research projects in this field, state transportation agencies seldom provide a formally written forensic pavement investigation guide. To resolve this issue, in 2013, the National Cooperative Highway Research Program (NCHRP) published the Guide for Conducting Forensic Investigations of Highway Pavements, also known as Report 747.

The NCHRP Report 747 explores the process for conducting forensic investigations of pavements to help understand the reasons behind premature failures or exceptionally good performance. The report recommends performing both functional and structural evaluations of pavements for forensic studies. It provides a general guidance on the organization and planning of the forensic investigation, sampling and testing requirements, analysis of results, and decision making process.

The Georgia Department of Transportation (GDOT) does not currently have an official guide for conducting forensic pavement evaluations. Before adopting the Report 747 as an official guide, a research team was formed to investigate what forensic evaluation methods were used by other states. A survey was developed and distributed to DOT's throughout the nation, as shown in Chapter 2 – Background. The survey was essential to determine the most commonly used forensic investigation methods that other state DOTs use. The survey also provided insight towards the causes of pavement distresses contributing to differences in performance (average, below average, above average) among rigid and flexible pavement systems in other states.

The primary goal of this research study is to evaluate the NCHRP Report 747 for GDOT's adoption by performing functional and structural evaluations of existing concrete and asphalt pavements. Specific objectives for this study include:

(1) Conduct a functional and structural evaluation to identify causes of distress on concrete and asphalt pavements based on the NCHRP Report 747;

(2) Provide a recommendation as to whether a forensic investigation guide for Georgia pavements is warranted based on the functional and structural evaluation in accordance with the Forensic guide;

(3) If warranted, either accept Report 747 as a forensic investigation guide for Georgia pavements or develop a GDOT version of the Pavement Forensic Guide by considering GDOT practices, the unique characteristics of pavements, materials, and weather conditions in Georgia.

This thesis consists of several chapters. Chapter 2 of this thesis contains information about the nationwide survey conducted to understand current forensic investigation methods. An overview of the pavement sections investigated and a literature review of a pavement types and testing methods is located in Chapter 3 – Literature Review. Chapters 4 presents an overview of Report 747 and potential problems. Chapter 5 states the objectives and tasks needed to complete this research. Chapters 6 through 9 present a forensic investigation on each pavement type and results from the investigation. Chapter 10 provides the conclusions of the research and Chapter 11 gives recommendations for the NCHRP Guide. A list of common abbreviations is presented in Appendix 1. A conversion chart for Standard International units is located in Appendix 2.

CHAPTER 2

BACKGROUND

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 2.1. Four responses were received from provinces in Canada.

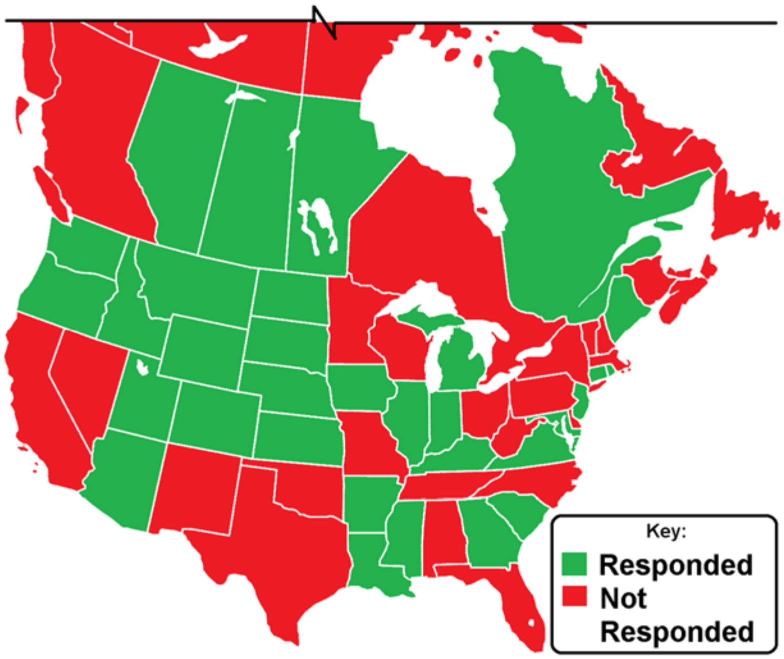


Figure 2.1 - Survey Responses in North America

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 focused on structural distress types and possible causes in pavement systems. Functional distresses such as riding quality and skid resistance vary widely from state-to-state due to climate and pavement types, and 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 used 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 2.1. 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 guides, respectively. 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 to be added to their procedures.

Table 2.1 - Survey Results: States and Canadian Providences with a Forensic Pavement Investigation Guide
Note: * includes 28 US states and 4 Canadian provinces.
** States/Provinces with a guide - Colorado, Maryland, Montana, and Saskatchewan.

	Response	Percentage
Yes	4**	12.5 %
No	28	87.5 %
Total*	32	100.0 %

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 2.2. 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 (Table 2.3). 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 methods that were 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. The methods are summarized in Table 2.4.

Table 2.2 - Survey Results: Non-destructive and Destructive Testing Methods Used
Note: ✓ Yes; — No (or No Response)

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%**

Table 2.3 - Survey Results: Rehabilitation Methods Used.
Note: ✓ Yes; ─ No (or No Response)

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%**

Table 2.4 - Additional Information Provided during the Survey.
Note: ─ No Response

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, Concrete hardened air, 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	─

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 2.5 contains the resources and guides provided during the survey. Nine DOTs responded and attached a guide or web links: Alberta (Canada), Colorado, Illinois, Louisianan, 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 2.5 - 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 2.3. It conducts “localized punch-out repairs, which are full-depth”. Near the end of the CRCP service life, Oregon DOT traditionally “overlay[s] 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 was 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, the use of JOR is common, suggesting that improved maintenance and design process are necessary.

It was discovered that states including Texas, New Mexico, and California had a pavement investigation guide or maintenance program listed on their website although they did not participate in the survey. Texas appeared to have a well-organized forensic pavement assessment process as well as rigid and flexible pavement rehabilitation methods available on its website (TxDOT 2015). California provides a well-organized Pavement Management System or PaveM (Caltrans 2016) which includes an Automated Pavement Condition Survey (APCS). The survey consisted 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 was used as the tool for data collection of continuous layer thicknesses.

New Mexico DOT illustrated the benefits to pavement design, maintenance, and management from 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. The report developed correlations between FHWA's Highly Polymer Modified (HPM) pavement distress data and Pavement Management System (PMS) data and pavement assessment projects (NMDOT 2016).

CHAPTER 3

LITERATURE REVIEW

GDOT selected four pavement types for this forensic investigation: Jointed Plain Concrete (JPC), Continuously Reinforced Concrete (CRC), SuperPave (SP) Asphalt, and Porous European Mix (PEM) Asphalt. Each pavement type contained 2 sections in “good” and “poor” condition, for a total of 8 sections. A forensic investigation was performed on each individual pavement section. A map showing the locations of all 8 pavement sections is shown in Figure 3.1. Please note that the CRC pavement sections are located 10 miles apart and appear as one map marker in the figure.

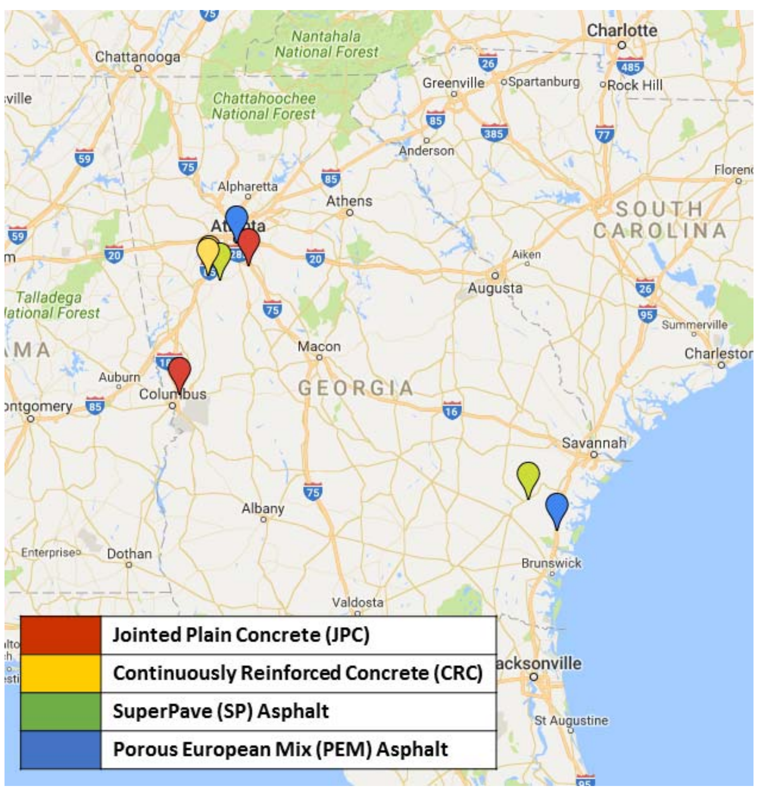


Figure 3.1 - Pavement Site Locations

3.1 Pavement Types Review

This section provides a brief explanation of each pavement type.

3.1.1 Jointed Plain Concrete Pavement

Jointed Plain Concrete Pavement (JPCP) is characterized by its concrete slabs that contain steel dowels to efficiently transfer load from traffic. These dowels are located at joints transverse to the direction of travel. The longitudinal joints connecting the slabs are held together with tie bars. According to the NCHRP Report 747, JPCP is known to be susceptible to distresses such as transverse cracking, joint faulting, and spalling (Rada, 2013).

3.1.2 Continuously Reinforced Concrete Pavement

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. Report 747 notes that CRC is susceptible to issues such as longitudinal cracking, which induces punchouts in the pavement (Rada, 2013).

3.1.3 SuperPave Asphalt Pavement

Superior Performing Asphalt Pavement System (SuperPave (SP)) was created in 1993 by the Strategic Highway Research Program (SHRP) (FHWA, 2010). An SP asphalt pavement is a Hot-Mix Asphalt (HMA) 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". The Report 747 suggests that Asphalt-Concrete AC pavement is often susceptible to distresses such as rutting, roughness, potholes, excessive noise, and skid resistance (Rada, 2013).

3.1.4 Porous European Mix (PEM) Asphalt Pavement

Porous European Mix (PEM) asphalt pavement uses an open-graded HMA mixture as a surface course. The surface course consists of a high level of coarse aggregate with a small amount of fines. In wet weather conditions, the water permeates the surface course and drains at the edge of the pavement layer, preventing tire spray (Barrett, 2006). The base and subbase layers are composed of dense-graded asphalt or a stone-matrix asphalt mixture. Common problems for PEM include raveling and excessive noise due to the clogging of the porous surface (Rada, 2013).

3.2 Visual Inspection/Observation

Visual observation identifies patterns that reveal pavement deficiencies. The pavement distress is 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 AC and Portland Cement Concrete (PCC) pavement is provided as a guideline. A copy of these visual inspection guidelines is shown in Appendix 5 and 6.

GDOT has developed their own visual inspection method, Pavement Condition Evaluation System (PACES). Depending on the type of distress, the cause is linked 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 (GOOD, no distress) to 0 (FAILED). Although the evaluation process may vary widely by jurisdiction, PACES provides a measure of pavement conditions based on the distress observed on pavement surfaces, as well as a practical indication of functional pavement condition as well as structural integrity.

3.3 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 without causing distress to the pavement layers. Further, NDT is responsible for identifying problems that have not appeared on the pavement surface.

3.3.1 Falling Weight Deflectometer

A Falling Weight Deflectometer (FWD) is a device that applies a load to a pavement section and measures the resulting deflections (FHWA 2006). Figure 3.2 shows an image of the FWD harnessed to the back of a van. FWD equipment quantifies 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 influences 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 divided by the maximum deflection of the loading plate (USDOT FAA

2011). Impulse Stiffness Modulus (ISM) plots display the stiffness over the length of pavement, providing a simplified way to check for the overall strength of the pavement section.

Figure 3.2 - Falling Weight Deflectometer

3.3.2 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 unit is shown in Figure 3.3. The GPR unit 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 GPR is effective in detecting moderate to severe stripping in hot-mix asphalt (HMA) pavements (Chen 2003). The GPR energy waves penetrate approximately three feet and operates at highway speeds, when attached to a vehicle, making it a useful addition to non-destructive testing technologies

(Chen 2008). The data results decrease in quality as the highway speeds increase, which may require road closures to receive more 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 3.3 - Ground Penetration Radar

3.3.3 Rolling Dynamic Deflectometer

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.

3.3.4 Other Non-destructive Testing Techniques

The Report 747 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).

3.4 Review of Pavement Forensic Technologies – Destructive

When NDT results indicate a moderate to severe distress, it is typically advised to perform destructive testing (DT). The use of DT involves entering or extracting from the pavement layer. The samples and/or information obtained from DT allow the analysis of pavement to have more accuracy.

3.4.1 Coring

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 3.4. A core sample shows a wide range of pavement layers (e.g., Subbase, Base, Subgrade, Concrete, and Asphalt mixture) available for analysis. When the core is taken, a borescope photographs problem areas, such as voids. Layer thickness and/or cause of distress is measured from taking core samples. To address specific problems associated with pavement, state DOTs conduct many laboratory tests on freshly cored pavement 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 tests performed on flexible pavement (Rada,2013).

Figure 3.4 - Coring machine

3.4.2 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). The amount of penetration is related to the relative strengths of stabilized and unstabilized aggregate base layers (ASTM D6951,2015). DCP testing is useful to evaluate existing pavement base and subgrade layer strength during pavement rehabilitation evaluations (MnDOT 2016).

3.5 Laboratory Testing Methods for Concrete Pavements

While many NDT techniques are effective on all types of pavement, PCC and AC pavements require different testing methods. This subchapter presents common laboratory testing methods for concrete pavements.

3.5.1 Alkali Silica Reaction and Coefficient of Thermal Expansion

It is known that nationwide, a high percentage of concrete pavement slab cracks may be attributed to high coefficient of thermal expansion (CTE) while the contributing factor for mapping cracks is generally Alkali Silica Reaction (ASR) (Kim 2012). ASR occurs when certain aggregates with a high silica content react with the alkali in cement. This reaction causes the formation of a swelling gel of calcium-silicate-hydrate (CSH) and ultimately causes 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 and resulting in its structural failure.

3.5.2 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).

3.5.3 Rapid Chloride Permeability

The Rapid Chloride Permeability (RCP) test is performed by monitoring the amount of electrical current that passes through thr sample, a slice of a pavement core, measuring 2 inches thick and 4 inches 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.

3.6 Laboratory Testing Methods for Hot Mix Asphalt Pavements

Asphalt is a flexible pavement, which requires different tests for effective analysis. This subchapter presents common laboratory testing methods for HMA pavements.

3.6.1 Bulk Specific Gravity and Theoretical Maximum Specific Gravity

The Bulk Specific Gravity (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:

Bulk Specific Gravity (Gmb)= $\frac{A}{B - C}$ (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 pycnometer and filled with water to a point of approximately 1 inch about the sample. Next, the container is sealed and a vacuum pressure of 3330 to 4000 mm Pascals iss applied for 15 minutes. Every 2 minutes, the container is agitated with a hammer to release any air bubbles. After the 15 minutes have elapsed, 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 equation 2 and the air content of as sample is calculated from by the Gmb and Gmm values using the equation 3.

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

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

3.6.2 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 122°F 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). An image of a Hamburg Wheel Tracking Test machine is shown in Figure 3.5.

Figure 3.5 - 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 (1000ºF) 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 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.

CHAPTER 4

PROBLEM STATEMENT

In the absence of a guide for conducting forensic investigations in Georgia, GDOT desires to evaluate and review this latest document for compatibility with current GDOT practices. If discrepancies exist, modifications will be developed and presented to GDOT for acceptance. The Report 747 recommends performing both functional and structural evaluations of pavements for forensic study. It provides a general guidance on the organization and planning of the forensic investigation, sampling and testing requirements, analysis of results, and decision making process. An overview of Report 747 is provided in this chapter.

The guide provides a recommended general approach to forensic investigations via flowchart (Appendix 3). The recommended approach begins with a request for a forensic investigation. The respective authorities evaluate, and support or deny this request. If supported, a preliminary investigation is conducted and a report is formed (Appendix 4). If a detailed investigation is justified, an investigation plan is prepared and executed in a three-phase forensic investigation (Appendix 4). Afterwards, the data is organized, an investigation report is made, and the project is closed out. The Report 747 requires each decision in the forensic investigation to be thoroughly documented. At all points throughout the investigation process, if a request is not supported, approved, or justified, a record of decision is made and the project is closed out.

The Report 747 organizes the forensic investigation into three phases, which is included in Appendix 4. The first phase involves writing the preliminary investigation plan and having it approved. In phase 2, non-destructive testing (NDT) is performed and the results are analyzed. If the investigation issue(s) are addressed, recommendations are made and the project is closed. If

NDT does not address the issues(s), the guide moves into phase 3. However, if the issue(s) are addressed, recommendations are made and the project is closed.

Phase 3 involves revising the investigation plan to include additional (typically destructive) testing. This plan is revised until approved. The additional destructive testing (or NDT) is conducted, followed by a data analysis. If the investigation issue(s) are not addressed, the previous step will be repeated. If all needs are addressed, a report containing recommendations and/or actions is written. Lastly, a record of decision is made and the project is closed out.

There are several concerns with the NCHRP Report 747. The report recommends an extensive list of non-destructive and destructive testing methods for a forensic investigation. A possible shortcoming of this process is not knowing which NDT and destructive testing methods are necessary to achieve an accurate conclusion. This is a concern because some forensic technologies may not be effective for Georgia pavements.

Also, Report 747 provides a visual assessment form for evaluating each type of pavement. However, Georgia uses a different method of visual inspection, if available. For example, there is a visual assessment form for JPC pavement types, but not CRC pavement types. Therefore, by investigating the four pavement types, a well-informed recommendation will be made as to whether accept or reject the testing methods and visual procedures for Georgia pavements.

Preventing premature conclusions is another concern of Report 747. Although comparing design conditions may help isolate one factor, it may not be the underlying cause of the pavement performance. The NCHRP Report 747 recommends an extensive list of possible contributing factors for pavement distress of each pavement type. However, all of these contributing factors may not apply to Georgia pavements. Additionally, Report 747 recommends obtaining information, such as as-designed data and as-constructed data, which in reality may be challenging to obtain.

CHAPTER 5

METHODOLOGY

To examine the functional and structural condition of existing asphalt/concrete pavements in Georgia, the methodology of the proposed study was divided into several tasks as discussed in the following steps:

<u>Step 1: Conduct a literature review that includes a nationwide survey of forensic investigation methods used by other state DOT's.</u>

The purpose of this task was to become familiar with the NCHRP Report 747 and their investigation approach, forensic investigation plan, non-destructive and destructive test methods, laboratory testing, analysis of all testing results, and development of actionable corrective solutions. As discussed in Chapter 2 – Background, a nationwide survey was conducted amongst state DOT's to determine what methods are used for forensic investigations and whether they accepted the Report 747 as their forensic pavement guide.

<u>Step 2: Perform a field investigation to identify current pavement functional and structural conditions.</u>

For tasks 2 and 3, two pavement sections (one with a high level of distress and one with good pavement performance) of the four pavement types (JPCP, CRCP, HMA (SP), and HMA (PEM)) were evaluated according to the Report 747.

The field investigation for the pavement sites consisted of reviewing PACES data to identify the current and past pavement functional conditions.

Next, GPR and FWD testing occured on the pavement sections previously mentioned. Using a GPR determined pavement layer thickness, detected voids underneath pavement, and determined other issues that are normally discovered through coring. The information obtained from FWD testing determined the impulse stiffness modulus (ISM), a parameter that reports how many kips of force are required to cause a deflection of 1 inch in the pavement. FWD testing results also determined the subgrade reaction (k), which is critical to asphalt pavement performance.

<u>Step 3: Conduct asphalt/concrete coring, traffic analysis, material testing, and specification requirement review</u>

In this task, any traffic reports and related information were reviewed as part of the technical approach to the pavement analysis of the test sections. Next, concrete and asphalt coring was conducted in collaboration with GDOT and underwent material testing. Concrete core samples were tested for material properties such as Coefficient of Thermal Expansion (CTE), Alkali-Silica Reaction (ASR), Carbonation, compressive strength, Modulus of Elasticity (MOE), and Rapid Chloride Permeability (RCP). Asphalt samples were tested for material properties such as subgrade reaction (k), Bulk Specific Gravity (Gmb), Theoretical Maximum Specific Gravity (Gmm), Hamburg Wheel Tracking Test (HWTT), and Binder Content. A review of these technologies is located in Chapter 3. Additionally, the specification requirements for Georgia pavements were reviewed with the data to confirm whether the materials are compliant.

<u>Step 4: Develop conclusions and recommendations in reference to testing and analyses</u>

Task 4 dealt with discussing the results and conclusions of the forensic investigation. Outside experts with experience in GDOT pavement practices were consulted to review the decisions developed from the study and identify possible changes if needed. Based on the conclusions of the study, recommendations were made for rehabilitation or other corrective solutions.

Step 5: Decide to recommend NCHRP Report 747 or develop a Georgia forensic guideline

For this task, Report 747 and the forensic study conducted were reviewed to make a recommendation for using the report as the pavement forensic guide in Georgia. If there were any additional amendments or recommendations, they were included as well. If Report 747 was not recommended, forensic design guidelines were created for use on GDOT pavements. These guidelines were based on GDOT specifications and address specific problems that are common with Georgia pavements.

CHAPTER 6

JOINTED PLAIN CONCRETE PAVEMENT

6.1 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.

In this study, two JPCP sites have been selected in consultation with GDOT as shown in Figure 6.1: 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.

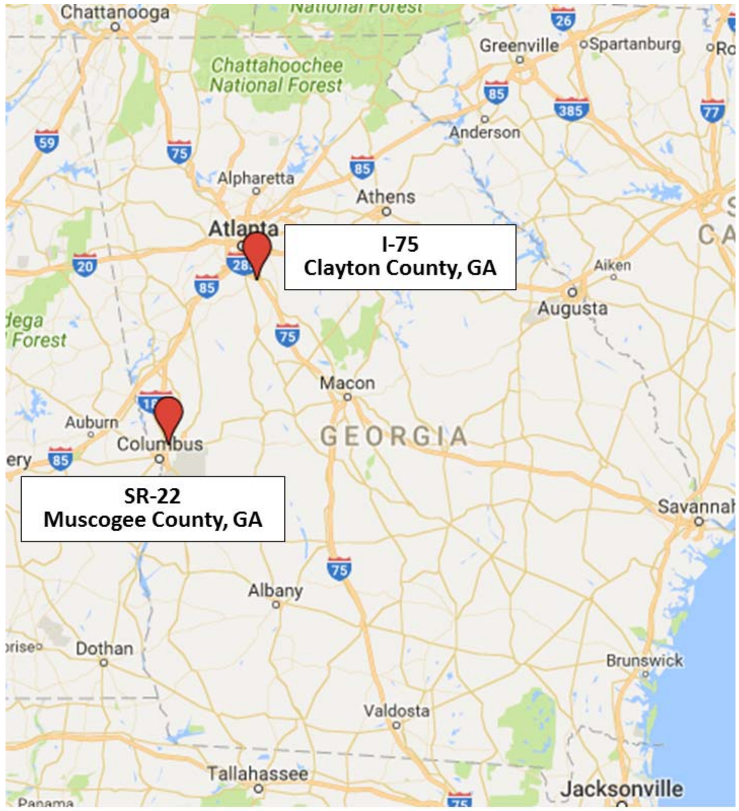


Figure 6.1 - JPCP Site Locations

6.2 Visual Inspection and Non-Destructive Testing

A mile-long section from each site was investigated. The first test site was located on State Route (SR) 22 (Westbound) in Muscogee County, Georgia. The road from milepost 8 to 7 was reported to be in ‘good’ condition, which showed no visible deficiencies. The second site was located on Interstate (I) 75 (Southbound) in Clayton County, Georgia. The road from milepost 226 to 228 was in ‘poor’ condition, with multiple deficiencies observed on its surface. A visual comparison of both sites is shown in Figure 6.2. Table 6.1 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, had a PACES rating of 100 and 40, respectively, as summarized in Table 6.1.

Deductions from I-75 are given for 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 (Figure 6.3). In the outside lane, longitudinal cracks running parallel to the centerline or wheel paths are the most frequently observed distress type. Small aggregate delamination was occasionally observed, as well as spalling.

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

I-75 was believed to have been constructed in 1968 with the earliest documented rehabilitation occurring in 1989. This section was 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) was 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) was composed of 10 inches of PCC over an Asphalt Concrete (AC) layer of 4 inches with GAB underneath. The inside lane consisted 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. In addition, inside lane experienced a lower volume of trucks.

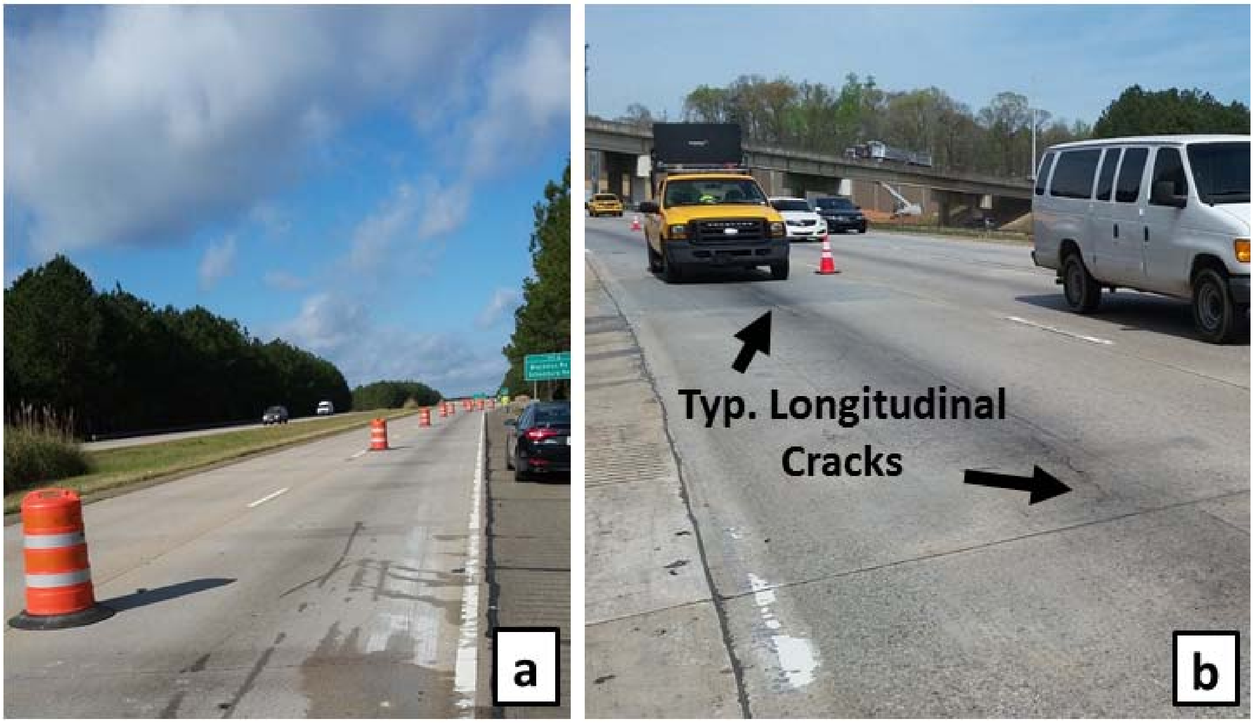


Figure 6.2 - JPCP General Site Conditions
(a) SR 22 (good condition); (b) I-75 (poor condition)

Table 6.1 - JPCP Site Conditions and Pavement Profile

	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)	

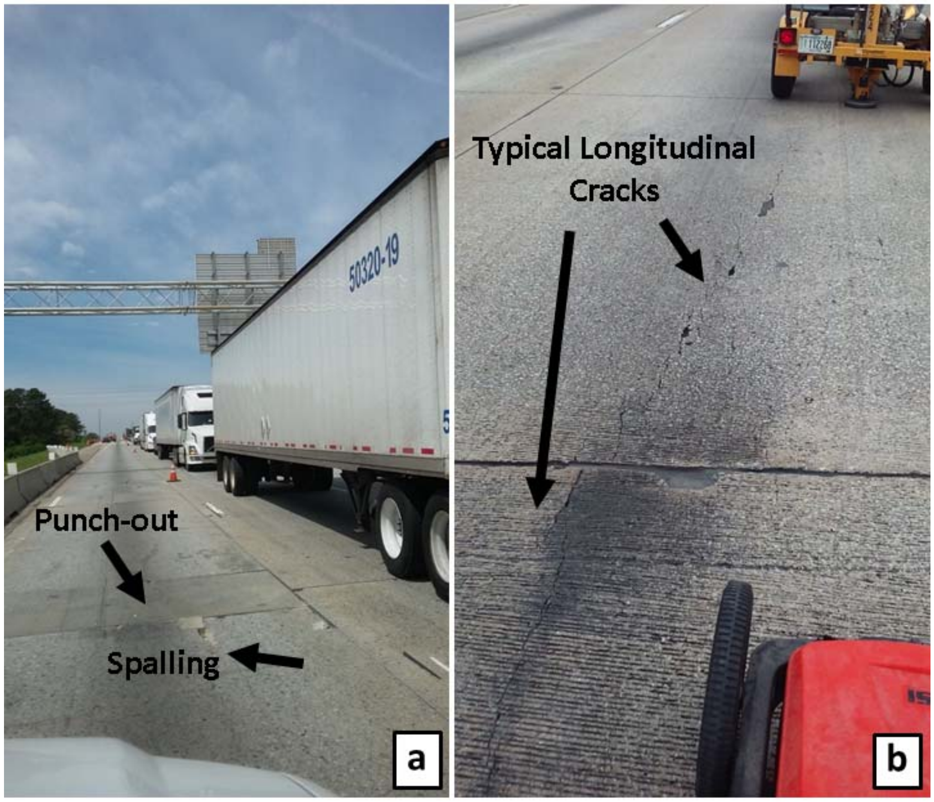


Figure 6.3 - I-75 Typical Distress (Poor Condition)
(a) Spalling Repairs and Longitudinal Cracking; (b) Longitudinal Cracking through joints.

6.3 Non-Destructive Testing

To evaluate in-situ elastic characteristics of in-service pavements, nondestructive testing (NDT) methods, such as FWD and GPR, are widely used. More information on these technologies are presented in Section 3.3 - Review of Pavement Forensic Technologies – Non-destructive.

Table 6.2 shows a summary of pavement structure determined from the GPR scans. This table 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 was representative of their design data (Figure 6.4). Both sites had 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 mid-depth of the pavement section.

Table 6.2 - JPCP NDT Results and Design Parameters.

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

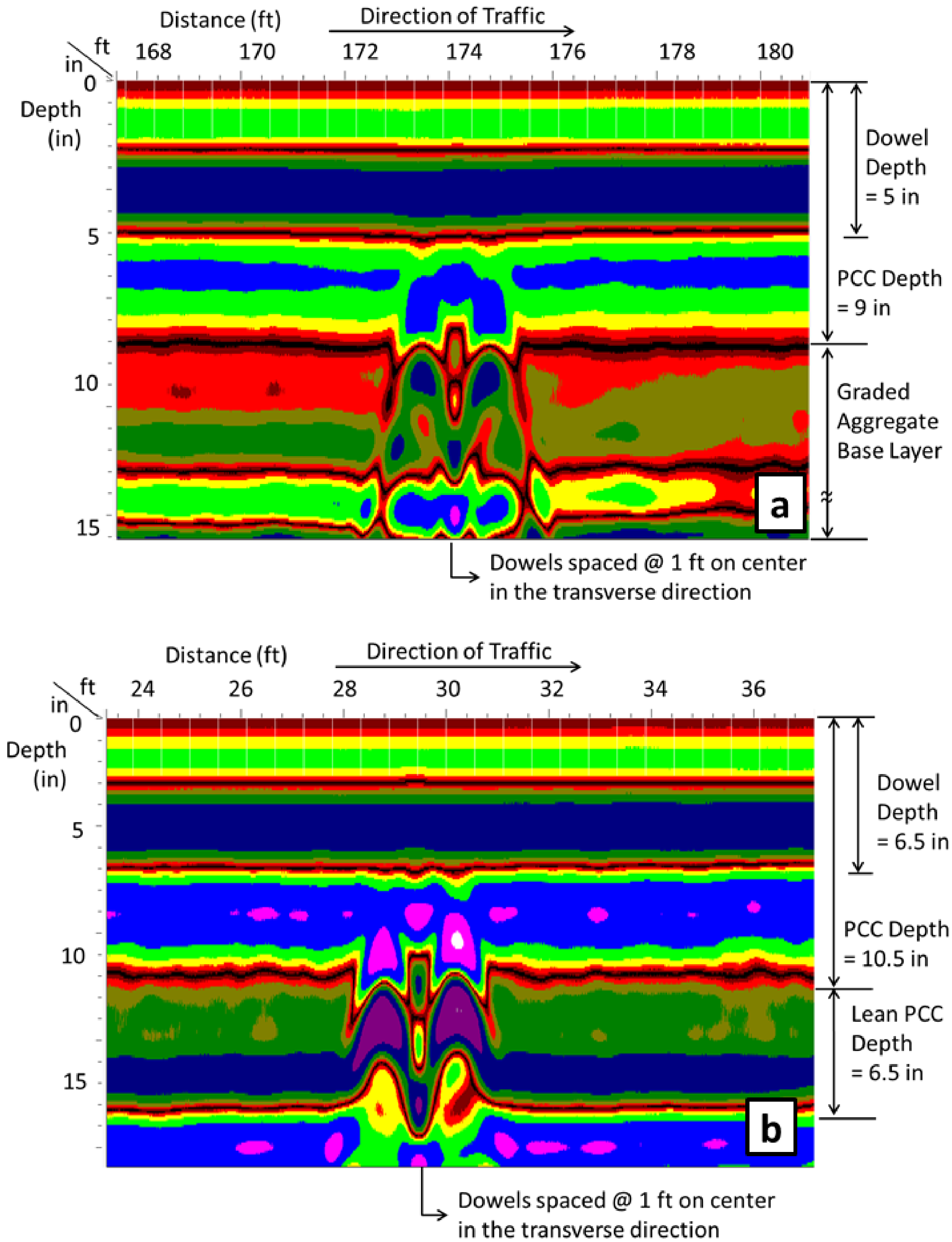


Figure 6.4 - Typical GPR scans showing single joint
(a) JPCP scan of SR 22 (good condition); (b) JPCP scan of I-75 (poor condition) in the direction of traffic.

The ISM plots created from the FWD testing are shown in Figure 6.5. Compared to the ISM plot SR-22, 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 the1993 AASHTO design guide (AASHTO, 1993) and is summarized in Table 6.2. These results show that I-75 had a stronger ISM value than compared to SR-22. However, concrete pavements are not dependent on a strong subgrade soil.

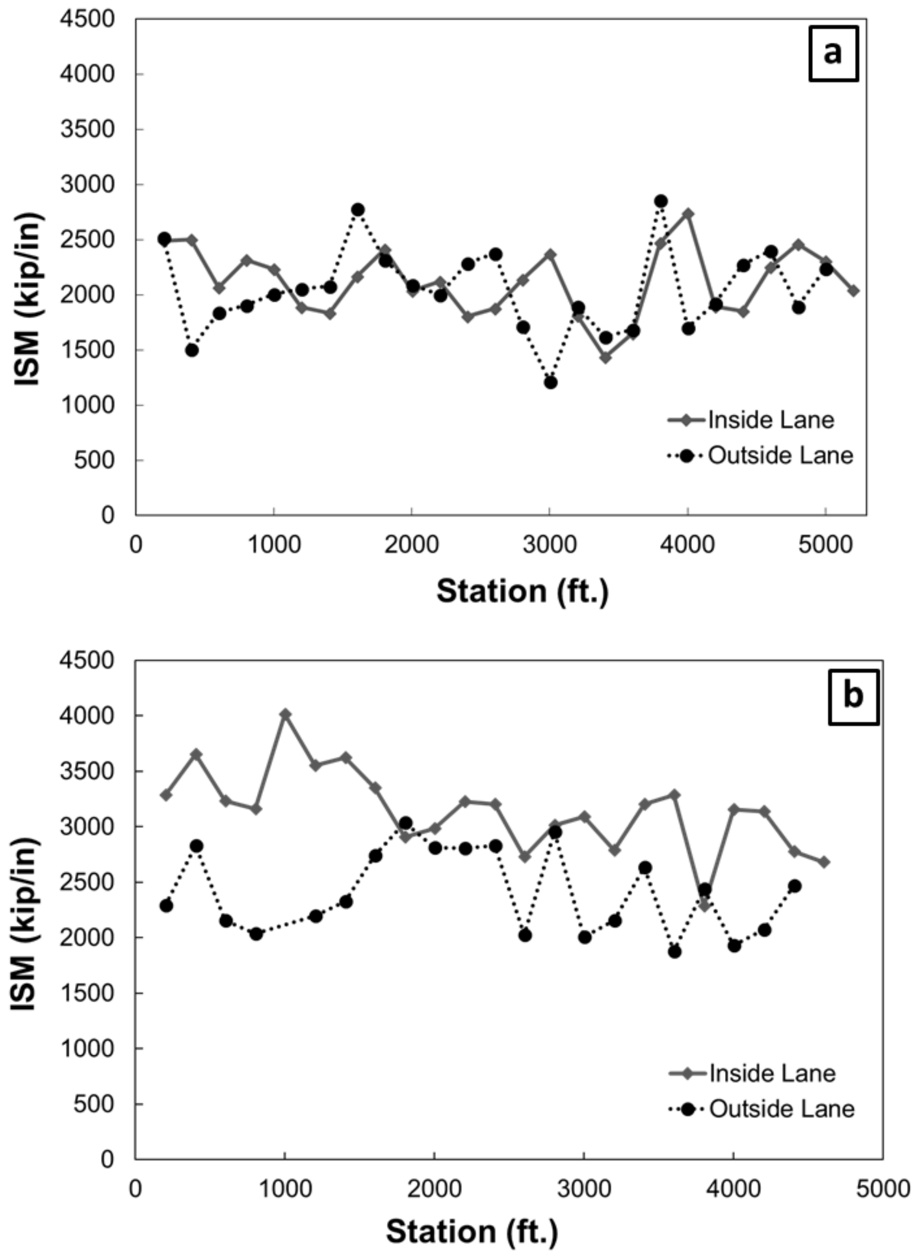


Figure 6.5 - Selected ISM Plots determined from FWD tests
(a) SR-22 (good condition); (b) I-75 (poor condition)

6.4 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 6.6. Figures 6.7 and 6.8 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 on outside and inside 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 6.7 and 6.8. 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 6.9. As shown in Figure 6.9, cores show consistent compaction and few voids.

Figure 6.6 - 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.

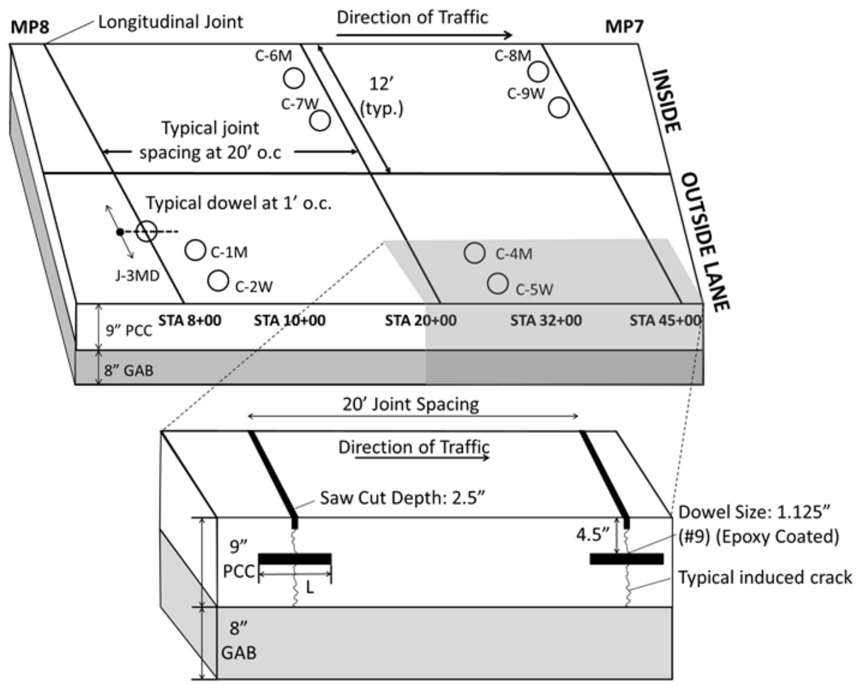


Figure 6.7 - 3D View of Coring Locations and JPCP details for SR 22 (good condition).

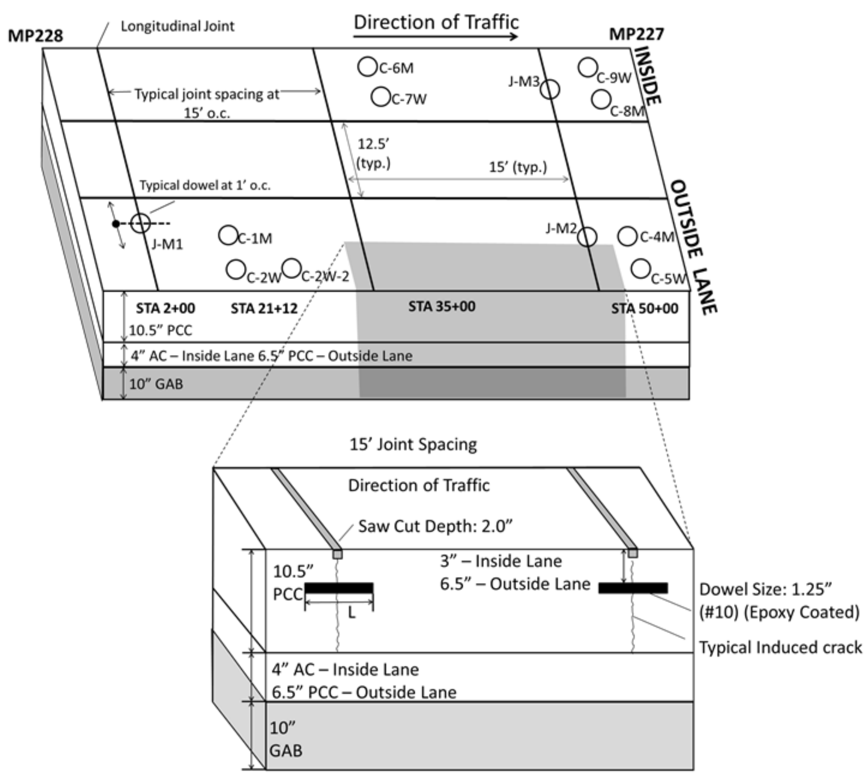


Figure 6.8 - 3D View of Coring Locations and JPCP details for I-75 (poor condition).

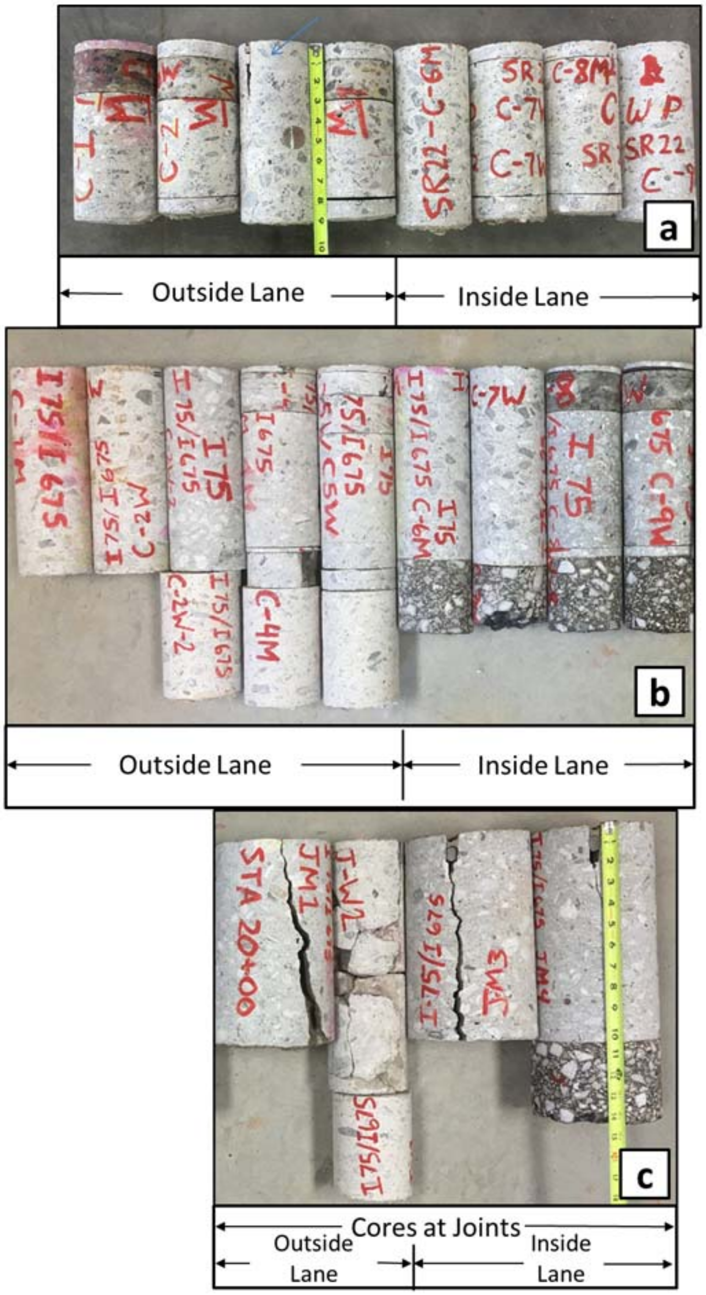


Figure 6.9 - All cores retrieved from JPCP sections
(a) SR 22 cores (good condition); (b) I-75 cores (poor condition);
(b) (c) I-75 cores at joints (poor condition).
Note: Core samples shown in (a) and (b) are re-assembled after conducting the laboratory tests for this Figure.

6.4.1 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, with no carbonation. The saw cut on the sample measured 2.5 inches (See Figure 6.9 (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.

6.4.2 I-75 Section Coring and On-site Testing

At the I-75 site, the inside lane showed less signs of joint failure, as opposed to outside lane, which showed many deficiencies. In addition, the two lanes show a visible difference in mixture design, which strongly indicates that they were constructed at different times. As indicated in Figure 6.9 (c), the saw cut on the sample measured 2 inches. The second core sample taken in the middle of the outside lane (C-1M) was tested for an Alkali-Silica Reaction (ASR). There was a bright yellow reaction around aggregates as shown in Figure 6.10 (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 6.9 (b), cores taken from the inside 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. The dowel (sample J-M4) was epoxy coated and did not show carbonation.

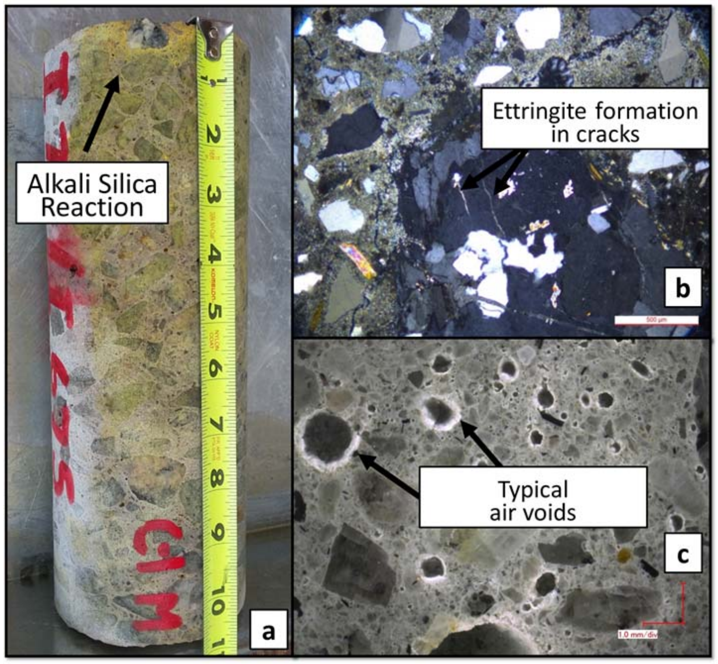


Figure 6.10 - ASR damage found in I-75 section (poor JPCP)
(a) On-site field test; (b) Photomicrograph of ASR gel and a crack filled with ettringite in thin section; (c) Photomicrograph of the typical air void structure from a core.

6.5 Destructive Testing – Laboratory Testing

A summary of laboratory test results is described in this section. More information on these specific technologies are presented in Section 3.2 – Laboratory Testing Methods for Concrete Pavements. The CTE tests were conducted using cored specimens in accordance with AASHTO T 336. The CTE of 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 6.3.

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 6.3, the RCP values in the SR-22 and I-75 were determined to be moderate and high, respectively.

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 outside lane, was relatively low, although the average compressive strength was comparable to values determined from SR-22, as shown in Table 6.3. Both I-75 and SR-22 meet the requirements for minimum compressive strength, which ranges between 3,000 and 3,500 psi (GDOT 430.3.06, 2013).

6.6 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. An entrained air content of 3%-6% was determined from the sample. In reference to quality acceptance criteria from GDOT, the water-to-cement ratio was 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 no slag or fly ash was located. The paste was described as well-hydrated, but mottled and unevenly distributed, which is associated with poor construction practices. Entrained air was not found, and a low air content (2%) was determined for the specimen. The air was entrapped, as shown in Figure 6.10(c), which is indicative of concrete susceptible to damage. 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 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 few 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 6.3 - Summary of CRC Core Test Results.
Note: * Carbonation was discovered within the crack surfaces of the concrete sample.

	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.	

6.7 Analysis of Testing Results

A low/medium level of ASR, as noted in Table 6.3, was observed at the surface of I-75. Both sites tested negative for carbonation. However, carbonation was found within the cracks which were exposed to air. The MOE results were unexpectedly lower in I-75 compared to SR-22. The RCP test results showed that values for I-75 ranged from 4100 to 7600, with an average of 6000. This indicated 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 indicated 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 had 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 prescribed possible causes of longitudinal and corner cracking, similar to the distress observed in I-75. Longitudinal cracking is related to 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) 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 experienced 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 having reasonable compressive strength. Therefore, in addition to the distresses found in the concrete materials, another distress factor was suspected when non-destructive and destructive test results indicated no major deficiency in the concrete performance (strength and stiffness) of the sections. It is 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.

6.8 Finite Element Analysis of the Distress

A finite element analysis (FEA) model was constructed using the ANSYS 16 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 was intended that a single slab of I-75 was 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 6.11(a). A joint spacing of 15 feet was selected for this study as shown in Table 6.2. 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 modelled 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 were modeled with solid elements, and dowels were assumed perfectly bonded to the concrete slabs. A single wheel load of 9,000 lb. of an equivalent 18,000 lb. single axle load was considered in this study. The contact pressure distribution was assumed uniform and the wheel loads were applied over a rectangular area of 10 in. by 20 in. to represent loads from two wheels, as illustrated in Figure 6.11(a).

Curling occurred in the form of a three-dimensional deformation which provided a positive curvature in two directions. A positive curvature from a temperature gradient mainly occurred in the direction of travel. Generally, in the stress analysis of JPCP, traffic load was 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 6.11 (a).

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 6.11 (b) shows a schematic diagram of principal strains in the concrete slab for combined thermal and wheel loads (18 kip 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 the value between 0.00010 and 0.00012 in/in . Curling of the JPCP slab due to a daytime positive temperature difference combined with a critical traffic loading position resulted in high tensile strain (greater than 0.00012 in/in) at joints and initiated a crack in the direction of travel parallel to the centerline of the joint (Figure 6.11(c)).

Based on the FEA analysis, it was ascertained that thermal deformation combined with structural deformation from the wheel loads caused 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 are presented in Section 10 – Conclusions.

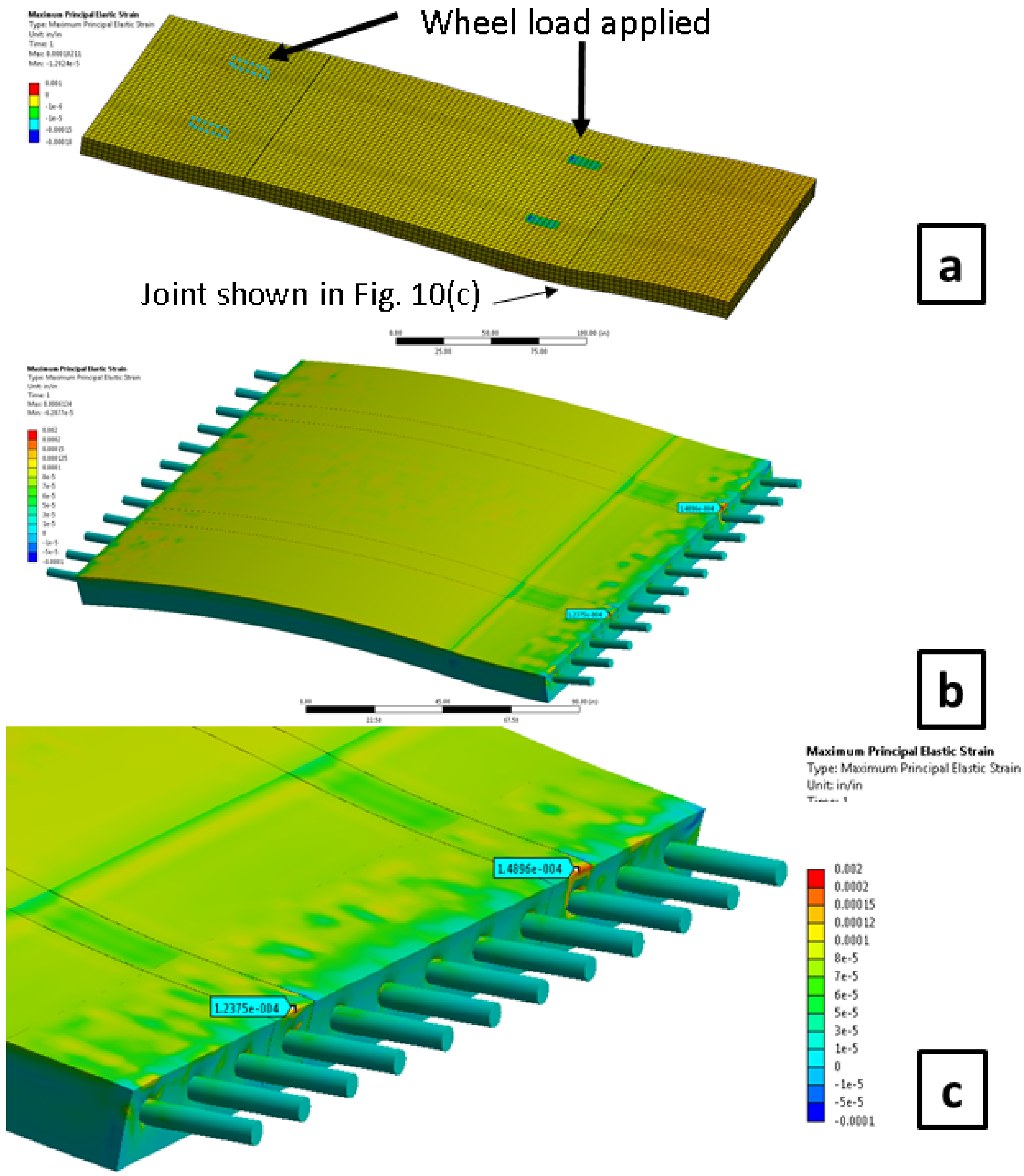


Figure 6.11 - FEA strain results

(a) Isometric view of the FEA analysis model under traffic loading; (b) Principal strain plot of the middle slab under combined traffic and thermal loading; (c) Enlarged joint view. Note: The element mesh is removed from the view for clarity and deformation magnified by a factor of 500.

CHAPTER 7

CONTINUOUSLY REINFORCED CONCRETE PAVEMENT

7.1 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. CRCP is susceptible to issues such as longitudinal cracking, which is known to induce punchouts in the pavement. Two trends have been observed between multiple CRC pavements. Over time, many CRC roads stayed in fair condition, while others deteriorated quickly. To study this phenomenon, a forensic investigation has been conducted to find the underlying causes of fair and poor pavement performance.

Interstate 85 stretches across the southeast region of the United States. Two pavement sections from this interstate have been investigated as shown in Figure 7.1. 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.

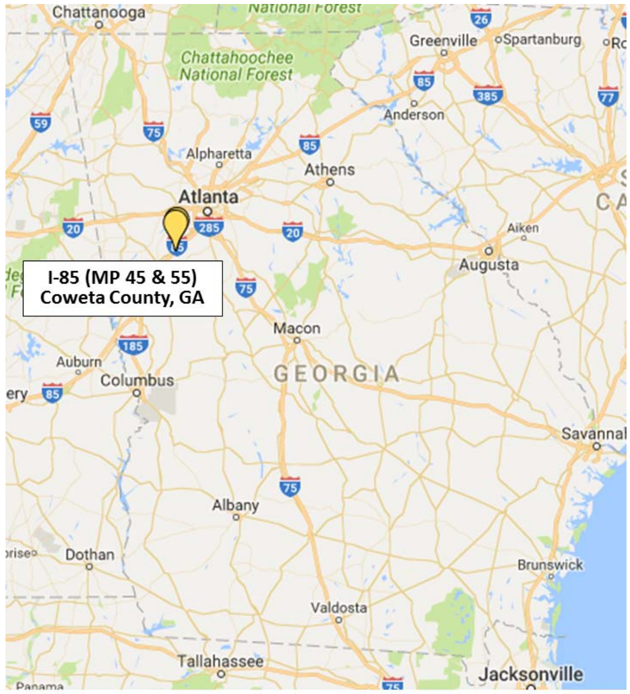


Figure 7.1- CRCP Site Locations

7.2 Visual Inspection and Non-Destructive Testing

The two sections used for this study are part of Interstate (I) 85 which runs through Coweta County, Georgia. The site that exhibited fair pavement performance is located between mileposts 45-44. The site that showed poor performance is located between mileposts 55-54. A visual comparison of both sites is shown in Figure 7.2. Table 7.1 includes a comparison of site conditions and pavement profile/construction parameters in the two CRCP sections. Currently, the PACES rating for CRCP was calculated based on the JPCP distress types (i.e., faulting), which was 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.

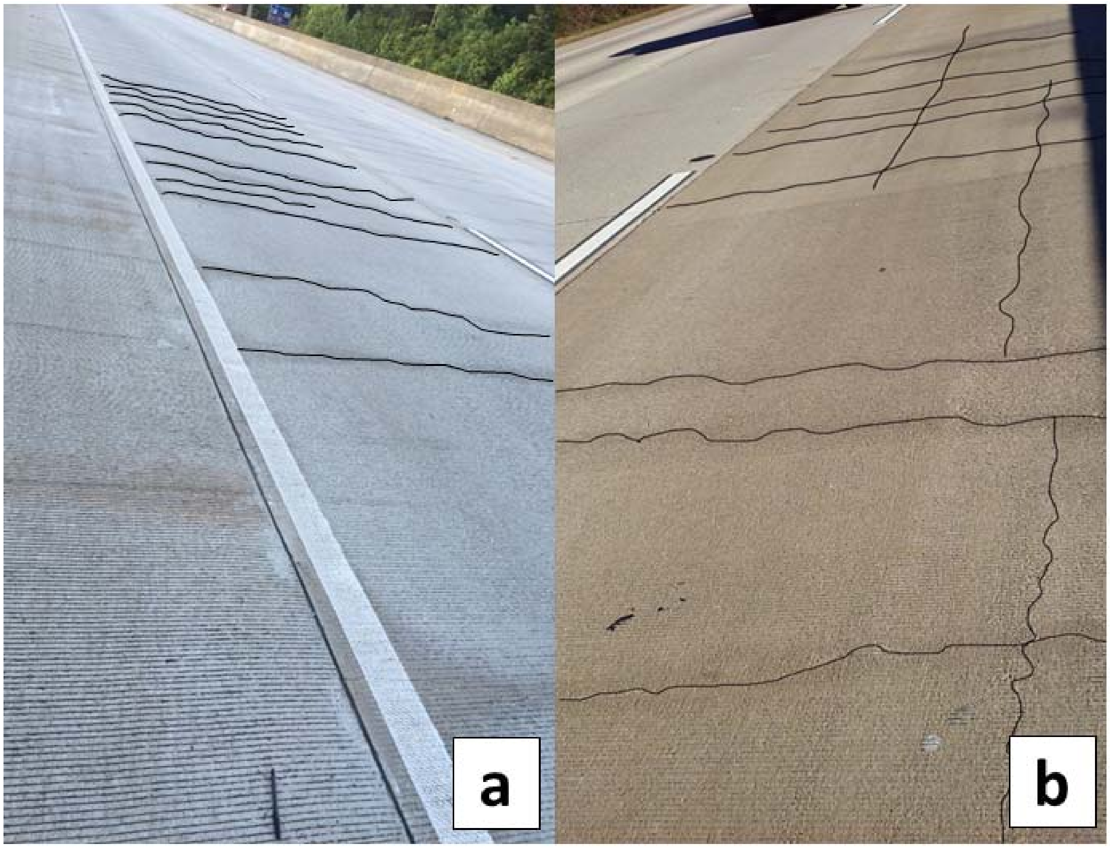


Figure 7.2 - Site Photos of CRC Pavement
. (a) I-85 MP 45-44 (Fair Condition) (b) I-85 MP 55-54 (Poor Condition).
Note: Cracks enhanced for clarity.

Table 7.1 - CRC Site Conditions and Pavement Profile

	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	

MP 45 was in fair condition and had a profile consisting of approximately 11.5 in. of PCC, 3.5 in. of Asphalt-Concrete (AC), and 12 in. of Graded Aggregate Base (GAB). MP 55 was composed of 12" CRC, followed by 3"AC, and 12"GAB underneath. The outside lane of MP 55 was 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) was seen throughout all three lanes in both sections, as shown in Figure 7.3. 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 were available to prevent unnecessary damage. To minimize cluster cracking, the Federal Highway Administration recommended a crack spacing between 24 inches to 96 inches (FHWA, 2012). Caltrans had a similar recommendation for crack spacing, with a distance between 21 inches and 84 inches between cracks (Caltrans, 2007). The Texas DOT warned that a crack spacing less than 24 inches could “be a precursor to punchouts” (TxDOT, 2011). With respect to crack spacing, cases of cluster cracking and Y-cracking were unique aspects of closely-spaced crack spacing and were problematic in terms of their contribution to localized failures (including punchouts). These types of cracking were 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. It was observed that in between the 3 ft. rebar spacing, 2 to 3 longitudinal cracks were observed. In the MP 55 section, crack spacing was observed to be very similar to MP 45, in that most of the cracks were located 3.5 inches to 13 inches apart. 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.

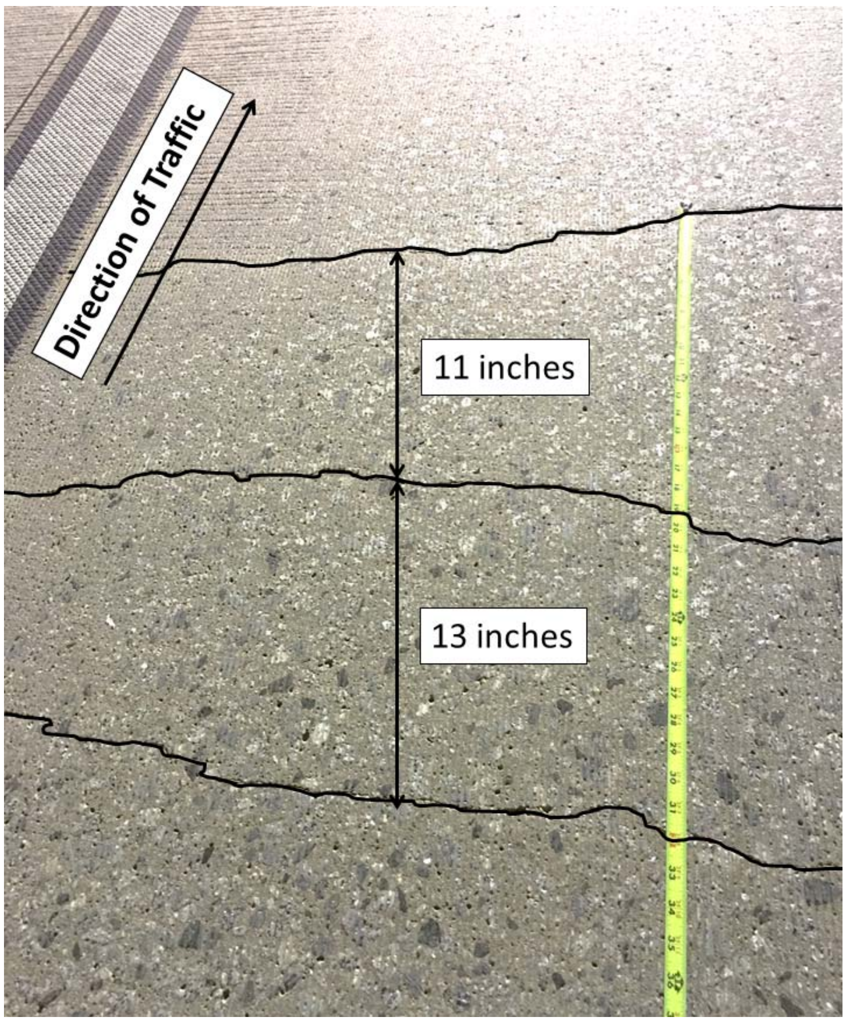


Figure 7.3 - I-85 Typical Transverse Crack Pattern (Cluster Cracking)

7.3 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 is included in Section 3.3 - Review of Pavement Forensic Technologies – Non-destructive. A summary of information acquired from NDT testing is shown in Table 7.2. The GPR data showed consistently level pavement layers. In addition, transverse rebar spacing was identified as 3 feet on center, as shown in Figure 7.4. 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 7.4.

Table 7.2 - CRC NDT Results and Design Parameters.

	Parameters	**I-85 MP 45-44 (Fair)**		**I-85 MP 55-54 (Fair/ Poor)**	
		Outside	**Inside**	**Outside**	**Inside**
	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

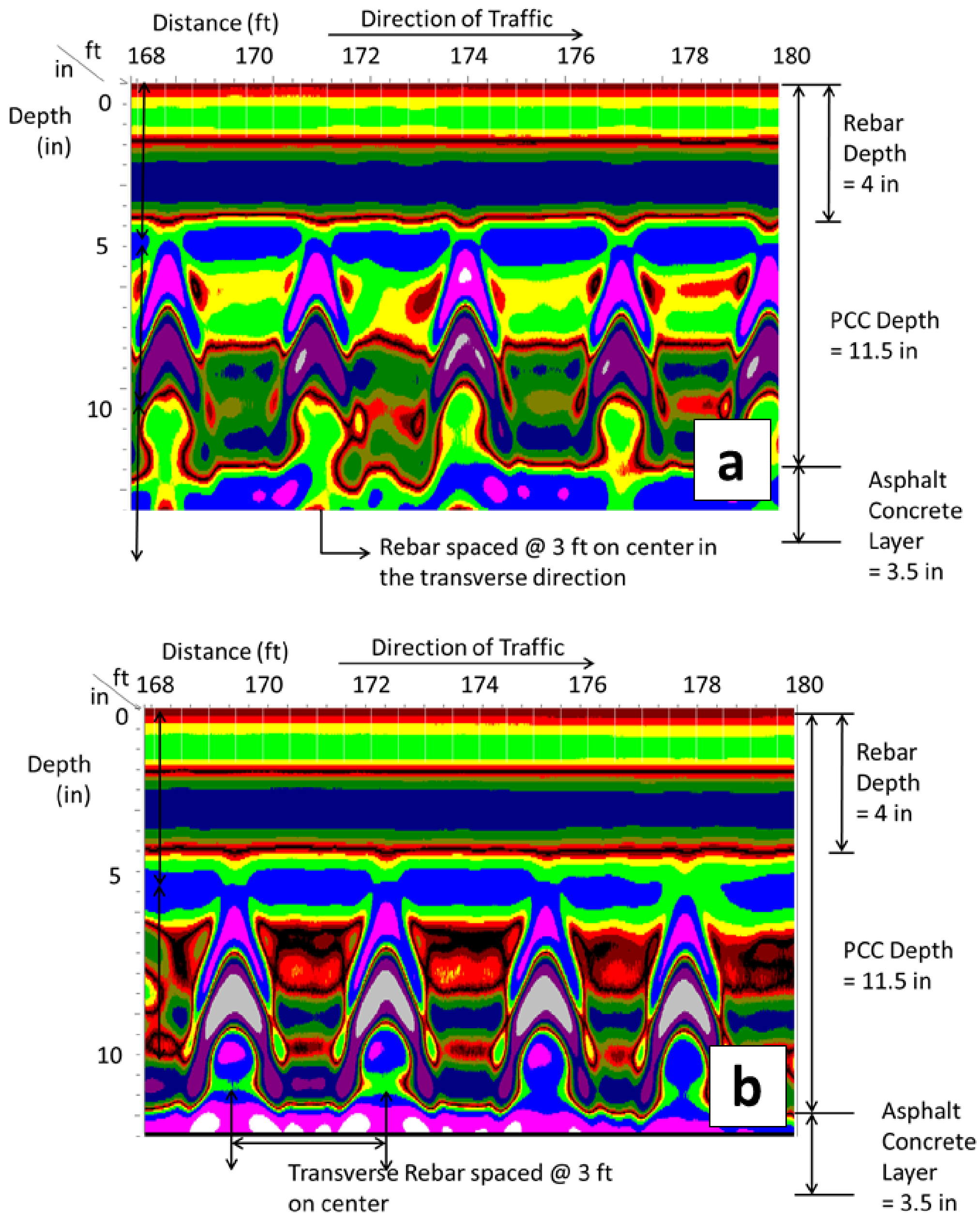


Figure 7.4 - GPR scans in the direction of traffic:
(a) I-85 MP 55-54 - Inside Lane; (b) I-85 MP 45-44 - Inside Lane;

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 7.2. As seen in Figure 7.5, the outside lane of MP 45 had 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 outside lane. This results in an abnormally high ISM value, as seen in Figure 7.5. The ISM test is dependent on the soil strength, although soil strength does not affect pavement performance.

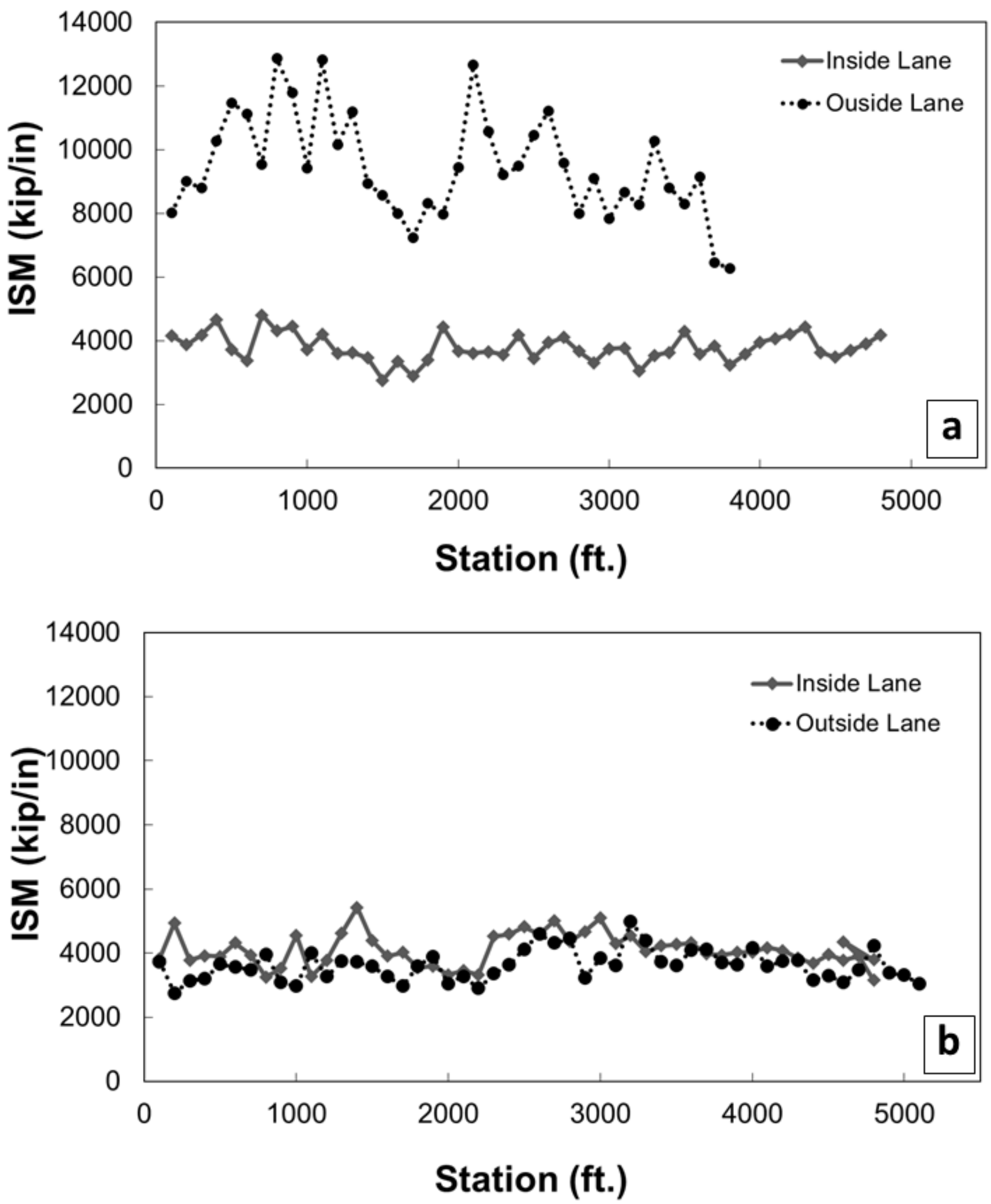


Figure 7.5 - ISM Plots for CRC Pavements
(a) I-85 MP 45 (b) I-85 MP 55

7.4 Destructive Testing – Coring and Field Testing

The coring process, including the machine used to drill cores, is shown in Figure 7.6. 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 the NCHRP 747 report, the cores were taken at the centerline of the slab, wheel paths on the inside and outside 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 7.7 and 7.8 which provide a 3D schematic of the pavement section.

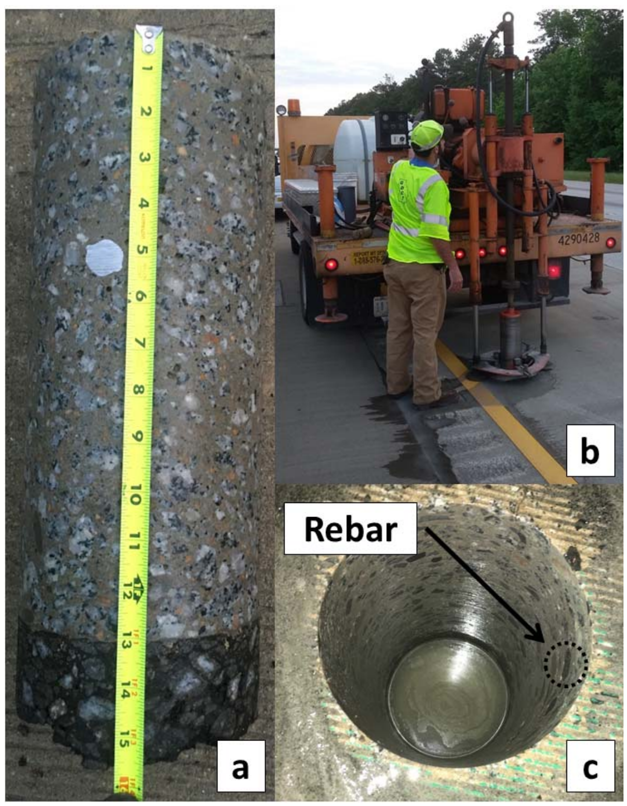


Figure 7.6 - Typical Cores at Rebar Locations
(a) Core sample; (b) Coring machine; (c) Inside view of a cored pavement.

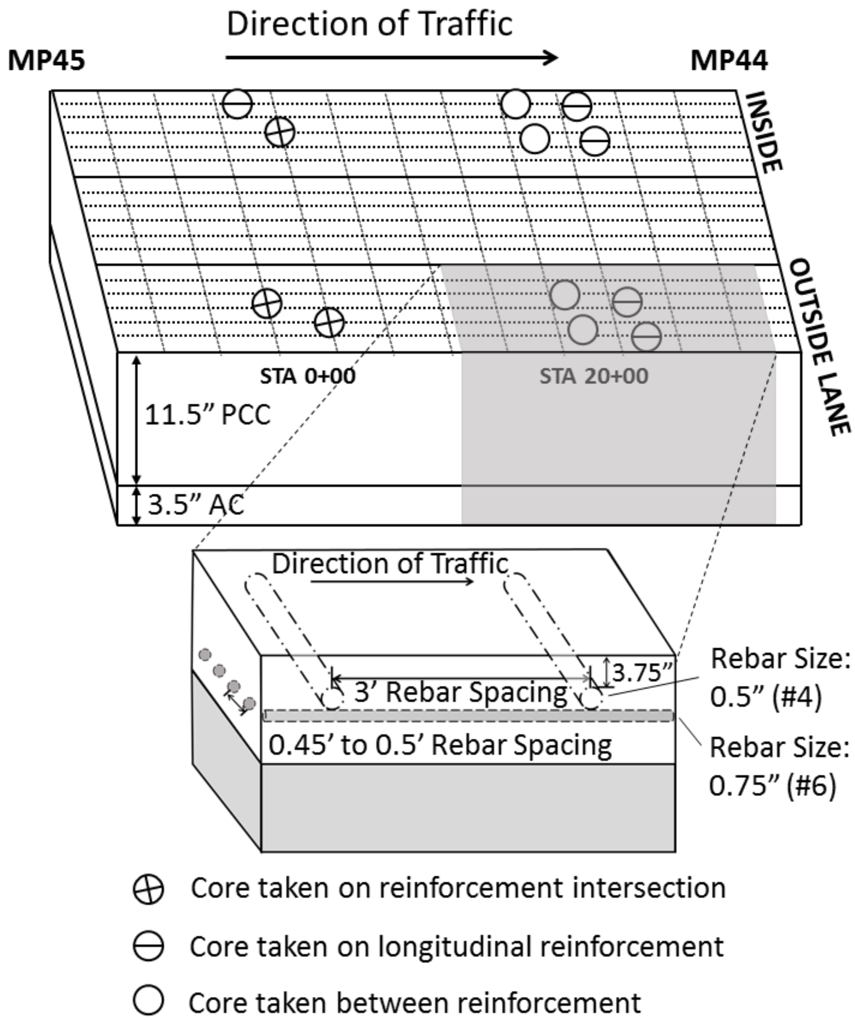


Figure 7.7 - 3D View of Pavement Design Parameters for Fair CRC (I-85 MP 45-44)

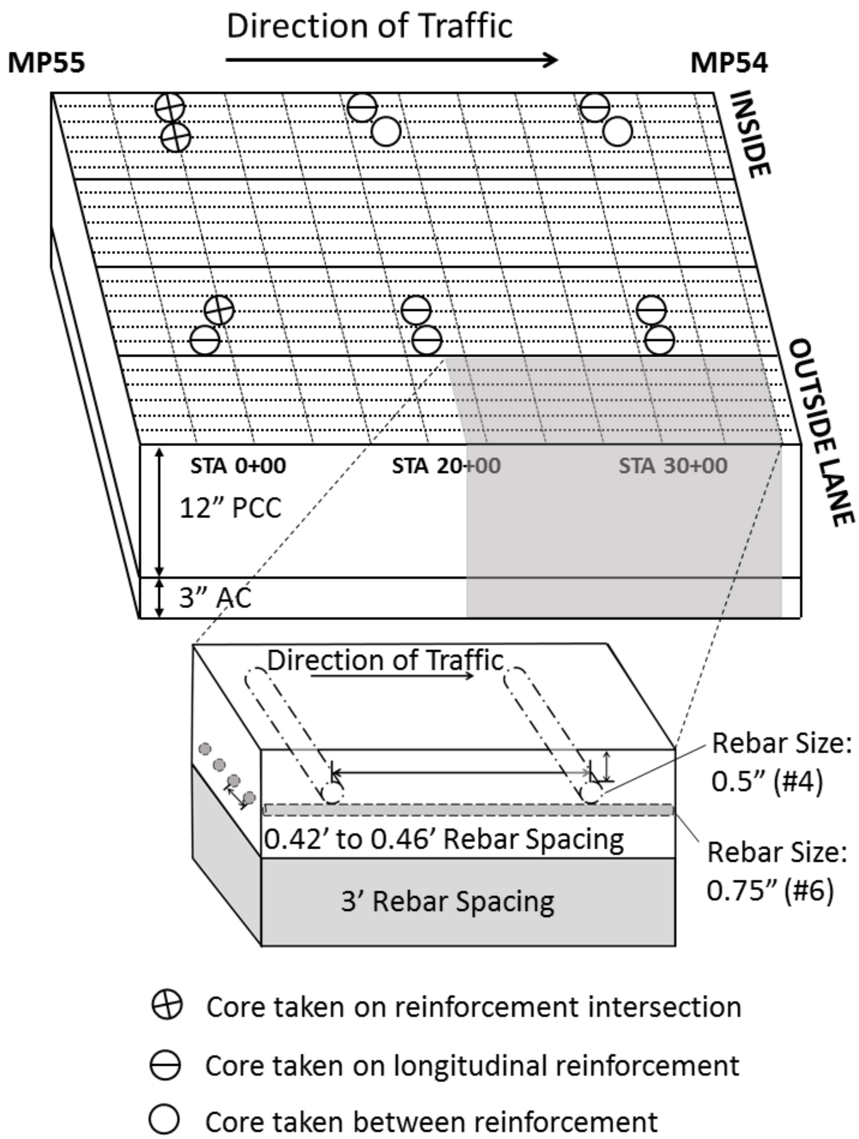


Figure 7.8 - 3D View of Pavement Design Parameters for Poor CRC (I-85 MP 55-54)

Photos of all cores extracted are shown in Figure 7.9. As seen in the figure, the reinforcement depth varied by ± 0.5 inches. MP 45 showed relatively consistent compaction with few voids. MP 55 had compaction problems in the outside lane, more specifically, sample C1MTR (Figure 7.9(b) far left). A visible difference was seen when comparing the concrete from the outside and inside lanes from MP 55. More details on these two material differences are provided in section 7.6 - Petrographic Examination.

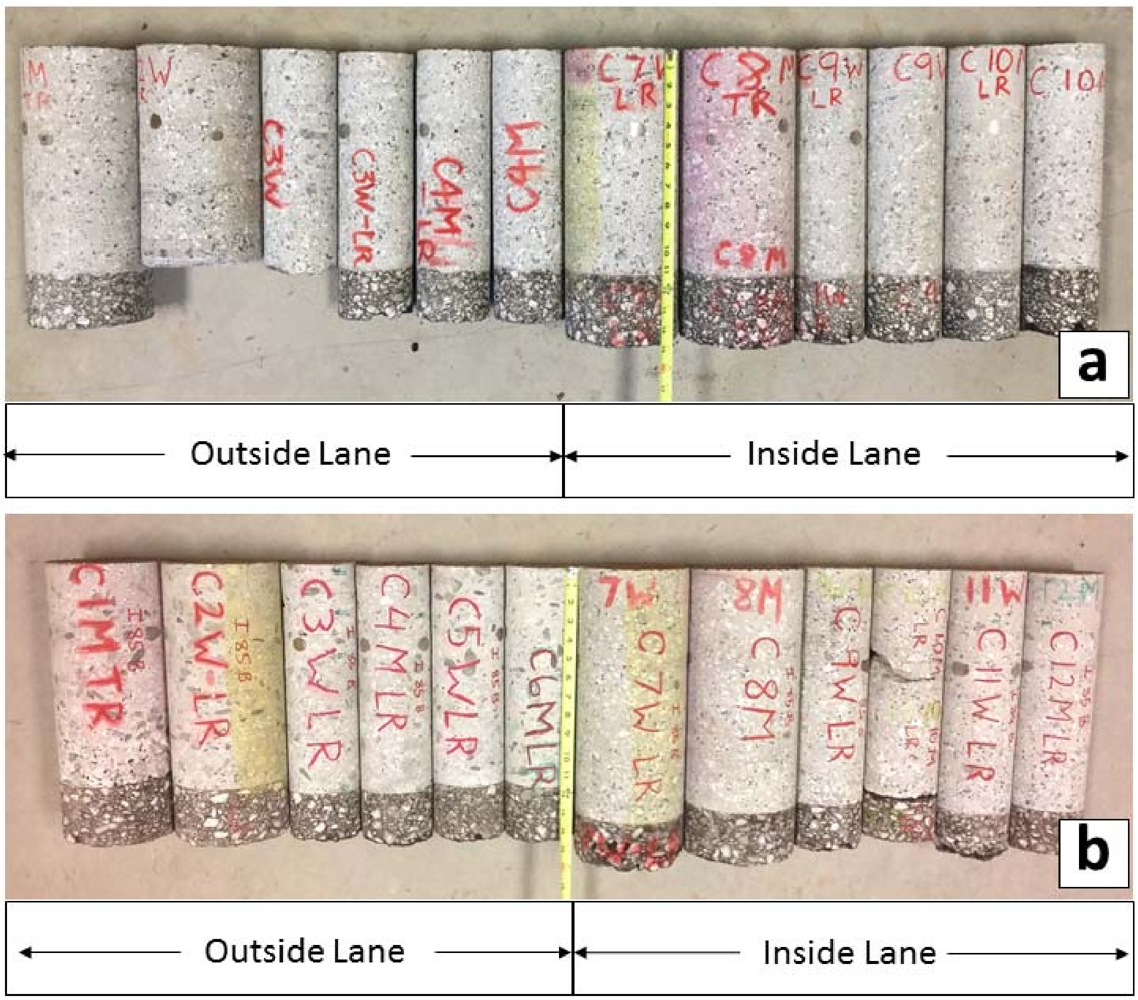


Figure 7.9 - All cores extracted
(a) Fair CRC (I-85 MP 55-54) (b) Poor CRC (I-85 MP 45-44)

7.4.1 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 ASR using a chemical testing kit. Both test results showed a negative reaction, which means no carbonation or ASR 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 are often caused by leveling on pavement surface. The reference level for placing dowels appeared to have been taken from the ground up. It was also observed that neither longitudinal nor transverse reinforcement were epoxy coated.

7.4.2 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 and the adjacent lane. It was noticed that the seal was missing between the outside and lane next to the outside. 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.

7.5 Destructive Testing – Laboratory

A summary of laboratory test results is described in this section. More information on these specific technologies is presented in Section 3.5 – Laboratory Testing Methods for Concrete Pavements.

The CTE tests were conducted using cored specimens in accordance with AASHTO T 336 (AASHTO T 336, 2011). The CTE of 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.

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 7.3, the RCP values in sections MP 45 and MP 55 were both determined to be moderate.

The MOE tests for the CRCP 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 average MOE for MP 45 was 3550 ksi. In comparison, MP 55 had a lower MOE average of 2790 ksi (Table 7.3). To investigate the possible reasons of varying material performance, petrographic analyses were performed.

Table 7.3 - Summary of CRC 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			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 is scratched by a Mohs 3 hardness point.
	Air entrained	See I-85 MP 55-54 Inside Lane		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.	

7.6 Petrographic Examination

Two cores from this site were selected for petrographic analysis, one each in the outside and inside lane of MP 55. The outside lane of the MP 55 had 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 3/4 inch NMAS 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 was attributed to poor consolidation, as shown in Figure 7.10(a). Although the paste was reported to be of fair quality, it was much softer than the aggregate, as seen in Figure 7.10(b). Also, segregation issues are often attributed to excessive vibration, as seen in Figure 7.10(c).

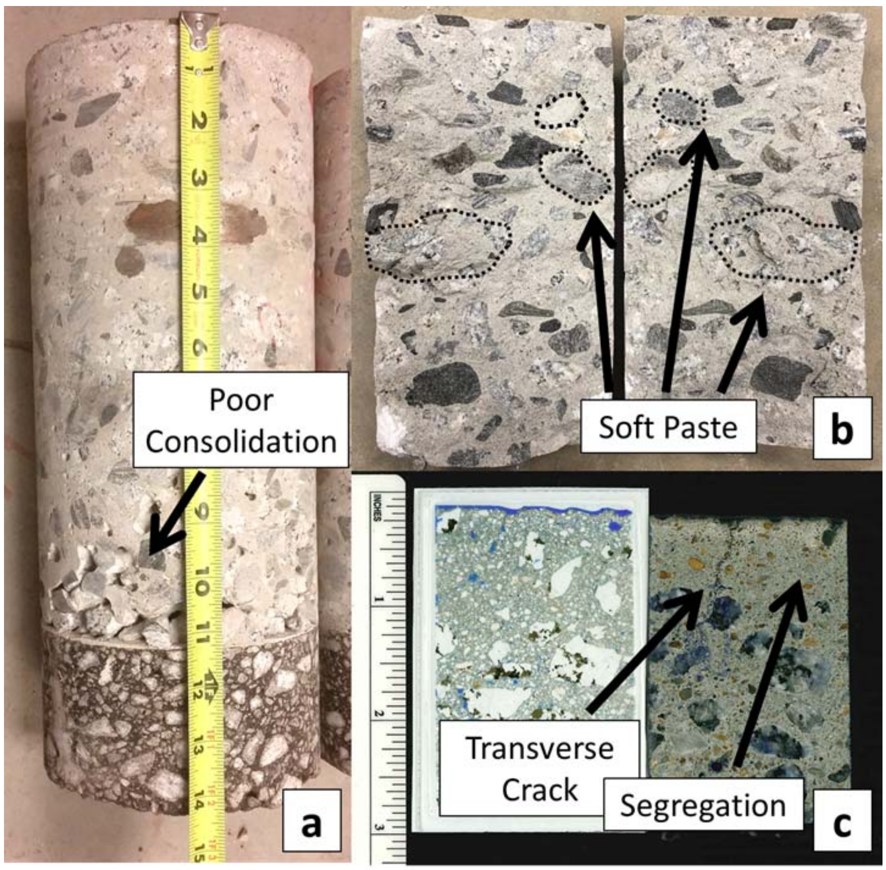


Figure 7.10 - Construction Signs of Distress* on I-85 MP 55-54 (Poor Condition)

7.7 Analysis of Testing Results

A summary of test results is shown in Table 7.3. 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 based on the GDOT RP 10-04 report (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 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 3900 for the inside lane and 3400 for the outside 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 result from "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 are 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 infiltrates the pavement (Pavement Interactive, 2012). A full report of conclusions and recommendations is included in Section 10 – Conclusions.

CHAPTER 8

SUPERPAVE ASPAHLT PAVEMENT

8.1 Introduction

The NCHRP Report 747 Guideline reports that Asphalt-Concrete (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 lead to various types of distress within asphalt pavements.

To investigate how HMA pavements behave in Georgia, a forensic investigation was conducted using Report 747 report as a guideline. Two HMA sites have been investigated, SR-38 in 'fair' condition and SR-54 in 'poor' condition (Figure 8.1). SR-38 shows less severe signs of longitudinal cracking and raveling (Figure 8.2(a)). Similarly, SR-54 shows visual signs of severe distress, mainly longitudinal cracking and raveling (Figure 8.2(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.

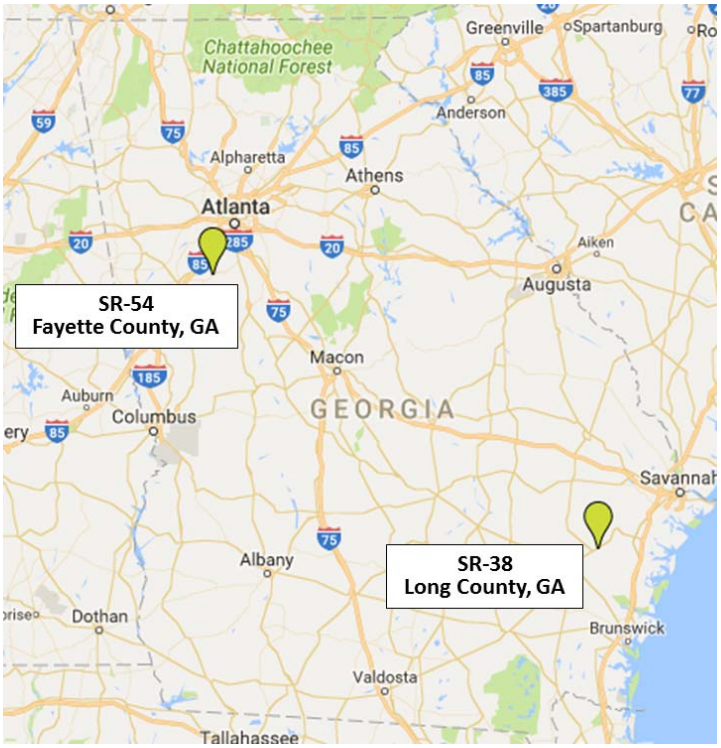


Figure 8.1 - AC Site Locations

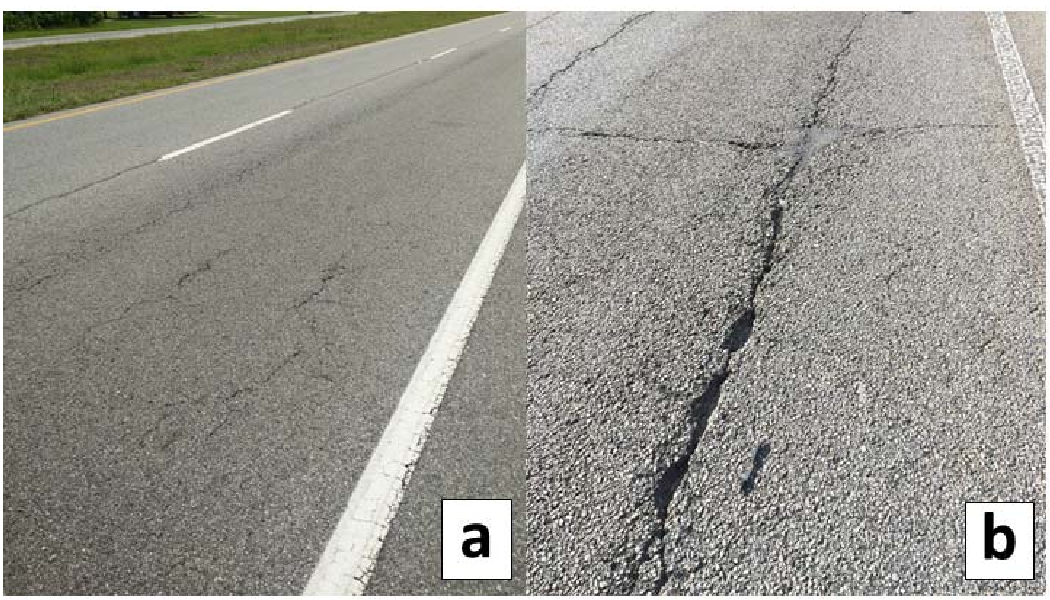


Figure 8.2 - Site Photos (AC pavements)
(a) SR 38 (fair condition) (b) SR 54 (poor condition)

8.2 Visual Inspection and Non-Destructive Testing

SR-38 in Long County was composed of AC with a densely packed Superpave (SP) surface layer. In addition, soil cement was used instead of GAB. The state route consists of four-lane divided 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 throughout most of the pavement. These surface cracks consisted primarily 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 surface weathering was observed in the left wheel path of the outside lane (Figure 8.3(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.

SR-54 (Eastbound) in Fayette County was composed of AC with a densely packed (SP) surface layer. The road was a four-lane divided highway (two lanes in a single direction), with moderate levels of traffic. In addition, SR-54 contained many stoplights and frequently experienced traffic congestion. A section between MP 5 to MP 4 was selected for this evaluation as it showed poor pavement performance in both inside and outside lanes. The test section exhibited severe longitudinal cracking along the wheel-paths of the road that extended throughout a majority of the section. Occasional transverse cracking was observed in both inside and outside lanes. Severe longitudinal (or linear) cracking and raveling were the most commonly observed signs of distress (Figure 8.3(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 was considered to be the result of pavement rehabilitation in the form of patches the filling of potholes. A comparison of site conditions and pavement profiles of the two HMA pavement sections is shown in Table 8.1.

Table 8.1 - SuperPave Site Conditions and Pavement Profile

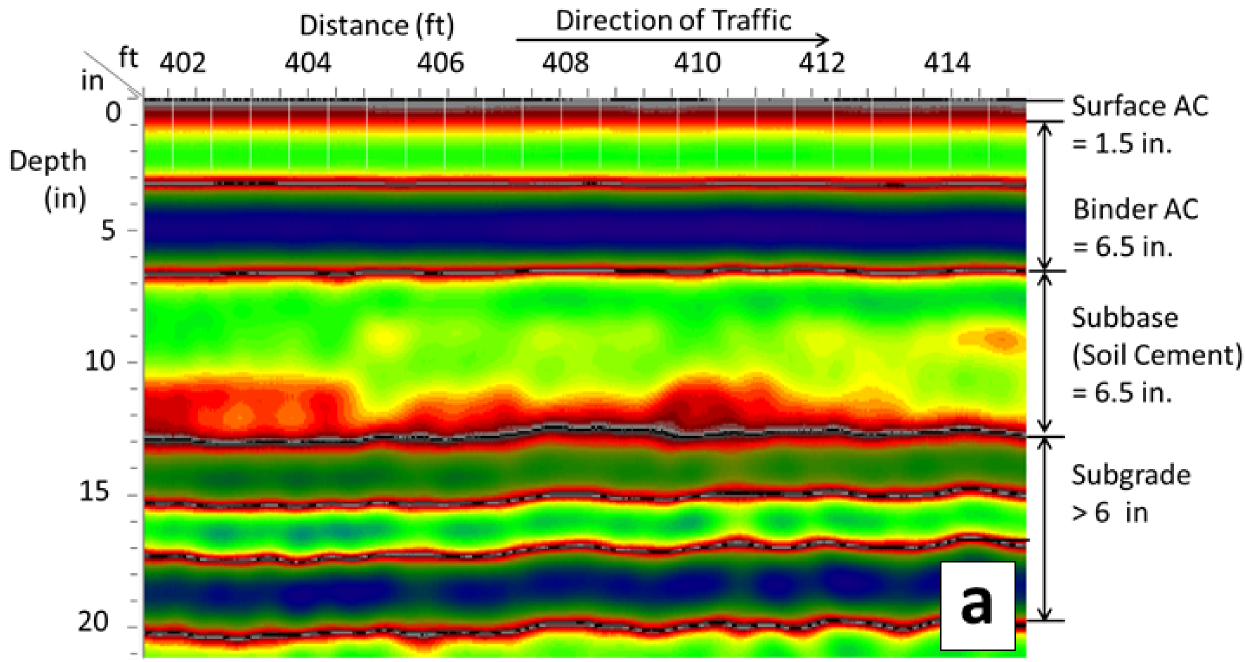
	Parameters	**SR 38 (Fair Condition)**		**SR 54 (Poor Condition)**	
		Outside	**Inside**	**Outside**	**Inside**
General	Condition	Poor	Good	Poor	Poor
General	Age (years)	N/A		26 (1990)	
General	Total Pavement Structure (inches)	1.75 (12.5mm SP)/ 6.5 (19mm SP)/ 2 (25mm SP)/ 0.75 (12.5mm SP)/ 0.75 (19mm 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)
General	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	
Traffic	ADT (% Trucks)	5,860 (9.5% Trucks)		21,680 (5% Trucks)	

Figure 8.3 - Typical Distress
(a) SR-38 (b) SR-54

8.3 Non-Destructive Testing

Non-destructive testing was carried out by using FWD and GPR units. More information on these technologies is included in Section 3.3 - Review of Pavement Forensic Technologies – Non-destructive. A representative scan from SR-38 and SR-54 is shown in Figure 8.4. The scan of SR-38 (Figure 8.4(a)) showed surface, base, and subgrade profiles with level layers. However, the GPR scan of SR-54 showed an irregular subgrade layer (Figure 8.4(b)). Variability of subgrade density and moisture level was observed, which results 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 8.5. 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 8.5(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 mixtures, with no GAB below. 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).



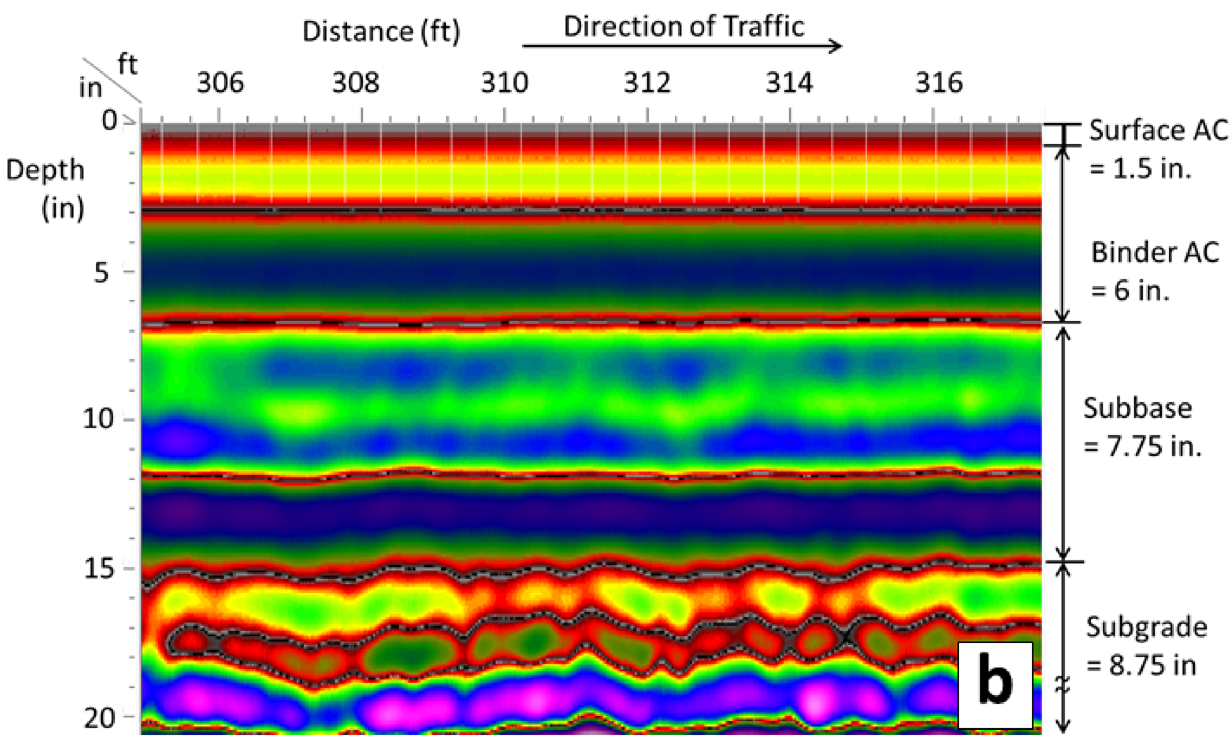


Figure 8.4 - SuperPave pavement scan
(a) SR-38 and (b) SR-54

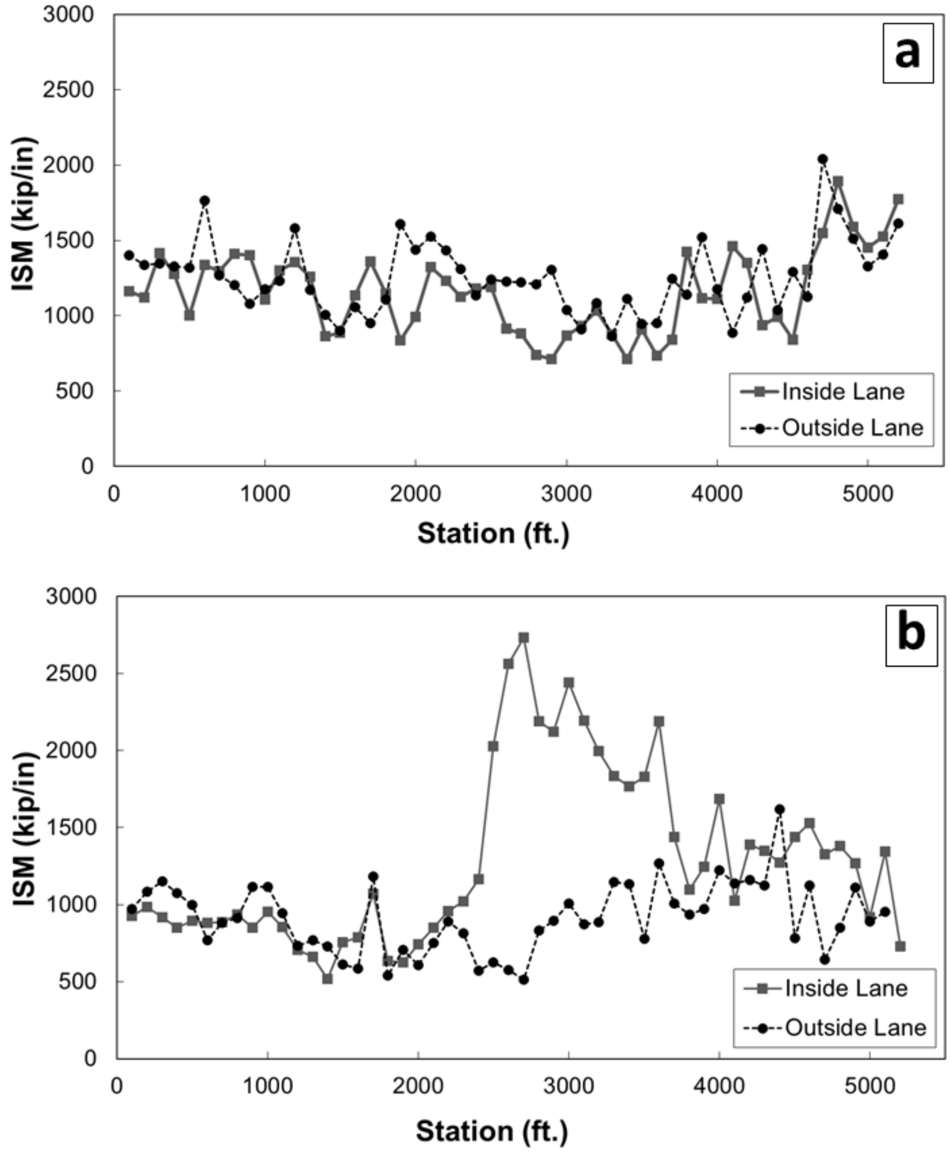


Figure 8.5 - ISM Plots for AC pavements
(a) SR-38 and (b) SR-54

The subgrade modulus for the subgrade layer of each pavement section was back-calculated 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 8.2. SR-38 met the requirements for SN, meaning that the pavement is structurally sound to hold its loading. However, SR-54 did not meet the SN requirements, meaning that the pavement needs to be rehabilitated.

Table 8.2 - 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 indicated 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).

8.4 Destructive Testing – Coring and Field Testing

8.4.1 SR-38 Coring and On-site Testing

A summary of laboratory test results is described in this section. More information on these technologies is written in Section 3.6 – Laboratory Testing Methods for Hot Mix Asphalt Pavements.

The cores samples from SR-38 contained compacted soil cement below their respective AC layer. The use of soil cement was typical in South Georgia to save GAB material and haul costs as most of quarries for GAB were located in North Georgia. Figure 8.6 shows typical cores taken at joints from SR-38. Figures 8.6(a) and 8.6(b) show a core sample before and after extraction. As shown in Figure 8.6(c), there were several AC lifts within the cored specimen. This could be attributed to a series of milling and overlay rehabilitation. All cores extracted from SR-38 and SR-54 are shown in Figure 8.8. Cores from SR-38 (Figure 8.8(a)) appeared very similar in length and layer density.

As previously mentioned, SR-38 exhibited moderate raveling and fatigue cracking on the left side of the outside lane (Figure 8.3(a)). Due to the unique location of longitudinal cracking, it was suspected that the distress was initiated by reflective cracking and deteriorated further due to traffic. Observation of cored specimens from SR-38 confirmed that cracking was reflected from soil cement. Reflective cracking is a common distress among pavements when a soil cement is used, because the soil cement typically experiences shrinkage cracks, which is reflected upward to the pavement surface (Adaska, 2004).

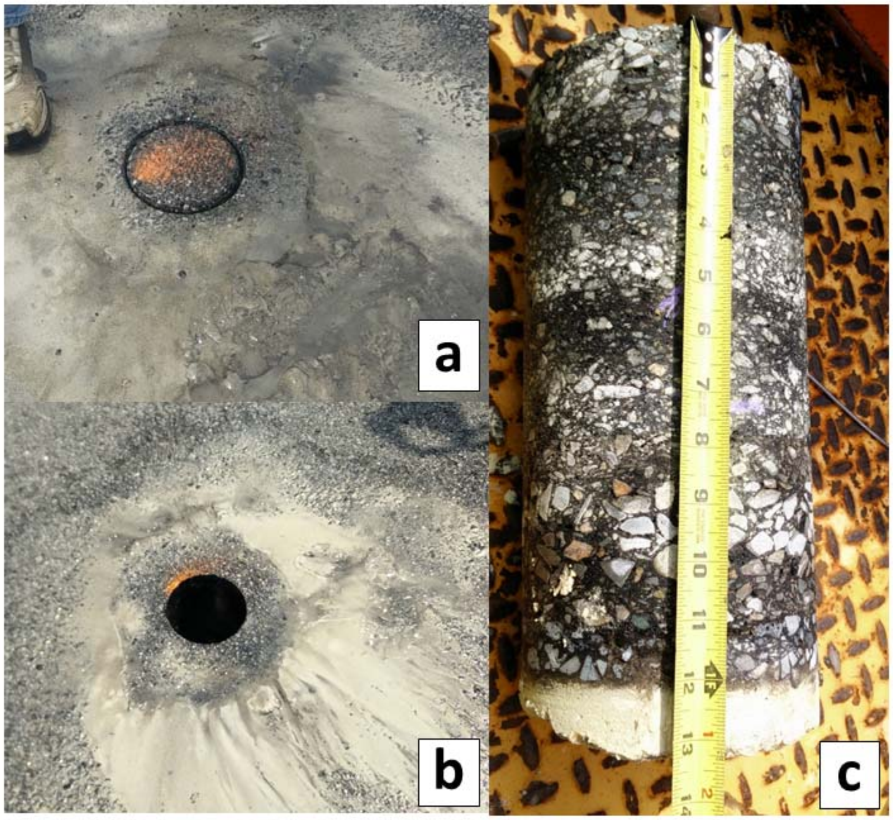


Figure 8.6 - Typical Cores at Joints (SR-38)
(a) Unremoved core sample (b) Pavement with core sample removed (c) Core sample

8.4.2 SR-54 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 8.7 shows images from the coring process on SR-54. A large, irregularly shaped void was discovered when coring sample C-7 (Figure 8.7(b)). The void was also part of the extracted sample, as seen in Figure 8.7(c). The void was 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 8.8. The core samples from SR-54 had a consistent thickness in the outside lane. However, cores from the inside lane had irregular lengths (Figure 8.7(c)). As mentioned previously, the 4th, 6th, and 7th samples in

Figure 8.8(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 lead to the conclusion that, the inside lane was composed of an existing pavement that contained an asphalt base (Figure 8.7c). 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. The large cracks shown in Figure 8.3 (b) initiated from the two pavement layers.

Figure 8.7 - Typical Cores at Joints (SR-54)

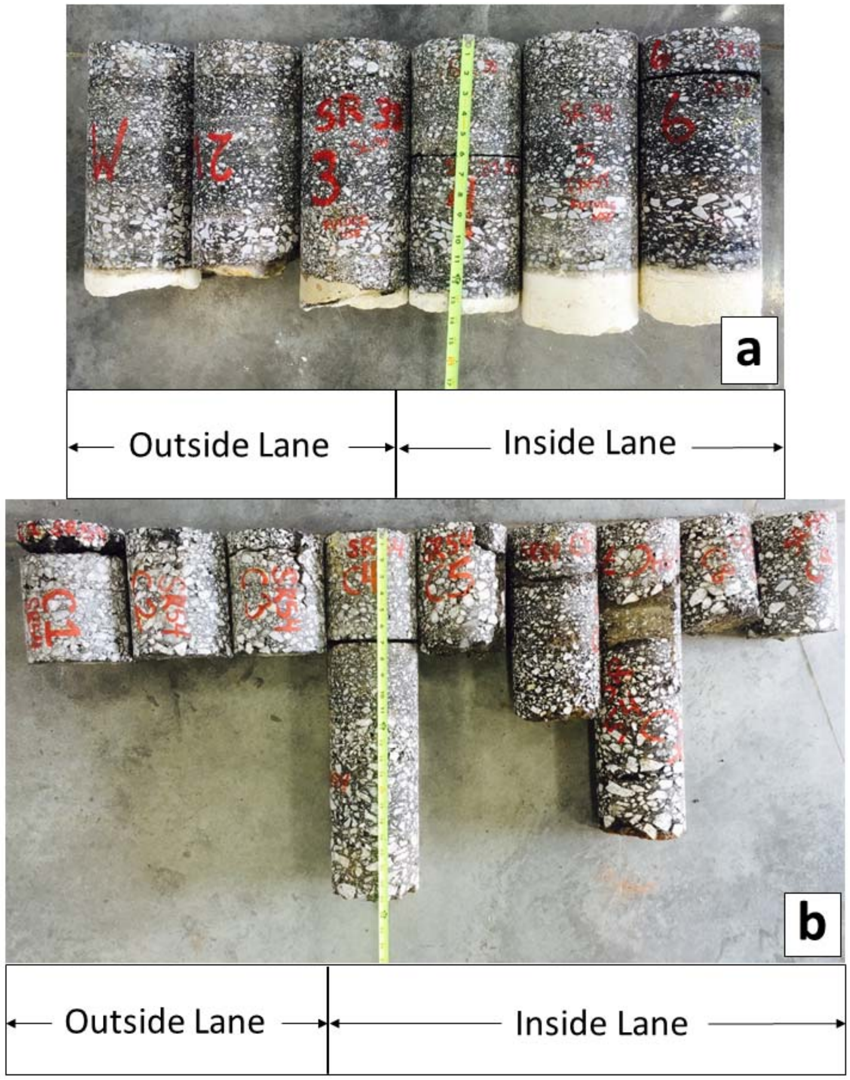


Figure 8.8 - All Cores (Superpave Pavement)
(a) SR 38 and (b) SR 54

8.5 Analysis of Testing Results

8.5.1 Air Content Analysis

The air content observed in sample SR-38 was highly irregular (Table 8.3). The air void content of the surface lift is 7.3%, which was close to the general guideline of 4 to 7%. The binder lifts each had varying air contents, which are not representative of the sample. SR-54 sample C-2 was taken from the outside lane, and shows a very high air content (11.3%), which indicative of raveling distress (Table 8.3). SR 54 sample C-2, taken from the inside lane, exhibited a high air content (7.8%), which was indicative of raveling distress. However, the binder lift below the surface had a low air content (4.0%), which may be the result of compaction caused by traffic loading.

8.5.2 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 1000°F until the internal scale reached a constant weight (approximately 60 to 90 minutes). Figure 8.9 shows how samples looked after burning in the ignition furnace. The binder content is calculated by the ignition oven and this information is listed in Table 8.3. The percentage of air voids is included in this table.

Figure 8.9 - Asphalt Sample after Ignition Burning

Table 8.3 - Summary of Pavement Information for Selected SP Sites
Note: LC=Load Cracking, BC=Block Cracking, RV=Raveling, RT=Rutting

Crack Type	Site	Condition /Lane	Pvmt Layer	Material Type	Material Sub-Type	Thickness, (in.)	G_{mb}	G_{mm}	Air Void (%)	Asphalt Content (%)
LC BC RV RT	**SR 38 C-4**	Fair/ Inside	1	AC	1/2 in NMSA	2	(Not tested)	2.49	(Not tested)	5.12%
			2	AC	3/8 in NMSA	1.5	2.26	2.52	10.45	5.29%
			3	AC	3/8 in NMSA	2	2.37	2.44	2.73	6.51%
			4	AC	1/2 in NMSA	3	2.43	2.45	0.77	6.07%
			5	AC	3/4 in NMSA	2.5	2.25	2.49	9.79	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 tested)	2.52	(Not tested)	4.58%
			2	AC	3/4 in NMSA	2	2.38	2.52	5.86	4.64%
			3	AC	1 in NMSA	4	2.43	2.52	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 tested)	2.50	(Not tested)	4.38%
			2	Binder	3/4 in NMSA	2.25	2.41	2.51	3.97	4.82%
			3	Base	3/4 in NMSA	3	2.36	2.52	6.44	4.50%
			4	GAB	GAB	10	--	--	--	--

8.5.3 Sieve Analysis

After ignition, 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 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 are listed in Tables 8.4 through 8.6. 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 outside the

GDOT specification range. These test results are based on a limited number of core samples and may not reflect the pavement's structural behavior as the gradation was out of range by a small margin. A full report of conclusions and recommendations is shown in Section 10 – Conclusions.

Table 8.4 - SR 38 C-4 (Outside Lane) Sieve Analysis

Sample	Layer 1	GDOT Spec	Layer 2	GDOT Spec	Layer 3	GDOT Spec	Layer 4	GDOT Spec	Layer 5	GDOT Spec
NMAS (in.)	1/2	1/2	3/8	3/8	3/8	3/8	1/2	1/2	3/4	3/4
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 8.5 - SR 54 C-2 (Outside Lane) Sieve Analysis

Sample	Layer 1	GDOT	Layer 2	GDOT	Layer 3	GDOT
NMAS (in.)	1/2	1/2	3/4	3/4	1	1
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 8.6 - SR 54 C-4 (Inside Lane) Sieve Analysis

Sample	Layer 1	GDOT	Layer 2	GDOT	Layer 3	GDOT
NMAS (in.)	1/2	1/2	3/4	3/4	3/4	3/4
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%	--

CHAPTER 9

POROUS EUROPEAN MIX ASPHALT PAVEMENT

9.1 Introduction

To investigate how Porous European Mix (PEM) pavements behave in Georgia, a forensic investigation was conducted using the NCHRP Report 747 report as a guideline. Two PEM sites were investigated, I-95 in ‘good’ condition and I-75/I-85 in ‘poor’ condition (Figure 9.1). I-95 does not show any visible deficiencies (Figure 9.2(a)). However, the I-75/I-85 section shows extreme raveling along the wheel paths (Figure 9.3(b)).

This investigation was intended to be completed in three phases, as Report 747 recommends. A non-destructive investigation was performed on both sites. Destructive testing was scheduled and core samples were taken from I-95. However, the pavement from I-75/I-85 was rehabilitated before coring samples could be taken. Thus, this chapter presents a partial analysis of PEM pavements.

Figure 9.1 - PEM Site Locations

Figure 9.2 - Site Photos (PEM pavements)
(a) I-95 (good condition) (b) I-75/I-85 (poor condition)

9.2 Visual Inspection and Non-Destructive Testing

Interstate 95 in McIntosh County is composed of a PEM surface layer, AC base and subbase, and GAB as foundation. The interstate consists of six-lane divided highway (three lanes in a single direction). The surface course was composed of PEM, and the base and subbase are composed of a SuperPave mix. Visual inspection revealed no visible signs of distress.

Interstate 75/85 in Fulton County is a twelve-lane divided highway (six lanes in each direction). The section was located where the I-75 and I-85 interstates merge towards Atlanta, and extremely high volumes of traffic are located. This section was selected for this evaluation as it showed poor pavement performance in all lanes, with an emphasis on the outside lane and lane adjacent to the outside lane. For the sake of organization within this report, these two lanes will be referred to as Lanes 5 and 6, with Lane 6 representing the outside lane. A summary of site conditions from both sections is listed in Table 9.1.

Table 9.1 - PEM Site Conditions and Pavement Profile

	Parameters	I-95 (Good Condition)		I-75/I-85 (Poor Condition)	
		Outside	**Inside**	**Lane 6 (Outside)**	**Lane 5**
General	Condition	Good	Good	Poor	Poor
	Age (years)	Not Investigated		Not Investigated	
	Total Pavement Structure (inches)	1 (PEM)/ 2(12.5mm SP)/ 5.5 (19mm SP)/ 5.25 (25mm SP)/ 12 (GAB)	1 (PEM)/ 3.5 (12.5mm SP)/ 3.5 (19mm SP)/ 12 (GAB)	Cores not taken	Cores not taken
	Visual Distress Observed	No deficiencies		Severe Raveling, Load Cracking	

9.3 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 is shown in Section 3.3 - Review of Pavement Forensic Technologies – Non-destructive. A representative scan from I-95 and I-75/I-85 with labels denoting PEM surface, base, and subgrade profiles is shown in Figure 9.3. Both scans showed consistently level layers with no air voids.

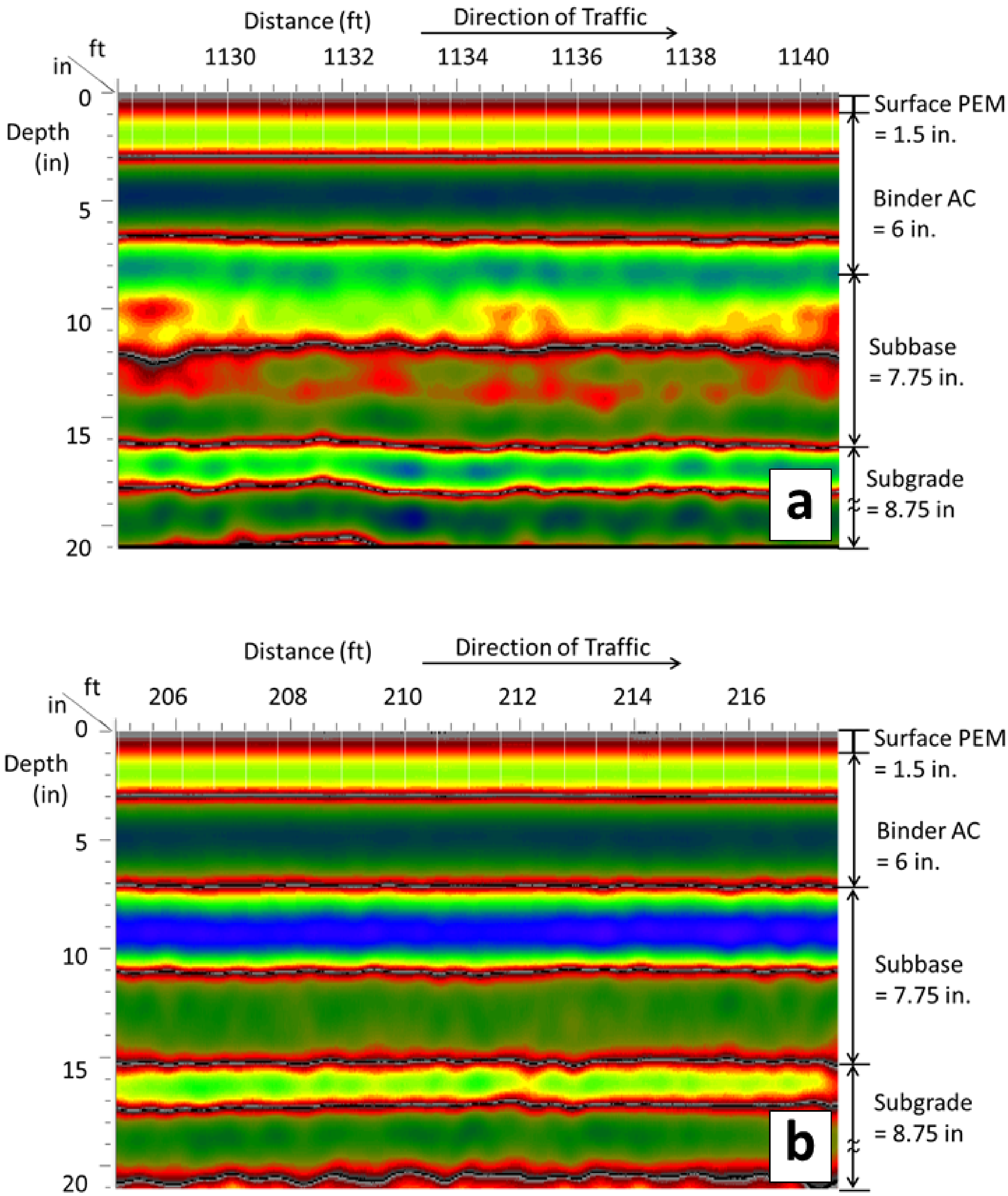


Figure 9.3 - PEM pavement scan
(a) I-95 and (b) I-75/I-85

CHAPTER 10

CONCLUSIONS

A forensic investigation was performed on JPC, CRC, and SP pavements. The conclusions formed from the investigations are presented in this chapter.

10.1 Jointed Plain Concrete Pavement

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. Therefore, a more extensive study is recommended to investigate the observed distresses in further detail.

10.2 Continuously Reinforced Concrete Pavement

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 in in 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.

10.3 SuperPave AC Pavement

A forensic investigation of two SP 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.

CHAPTER 11

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, SP) 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 747 Report, 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 1).
- 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 doesn't reflect the functional condition evaluation of CRCP properly.
- 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

Appendix 1 - Table of Abbreviations

Abbreviation	Full Name
AASHTO	American Association of State Highway and Transportation Officials
AC	Asphalt Content
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
JOR	Joint Repair
JPC	Jointed Plain Concrete
NCHRP	National Cooperative Highway Research Program
NDT	Non-Destructive Testing
PDR	Partial Depth Repair
PEM	Porous European Mix
RCP	Rapid Chloride Permeability
RWD	Rolling Wheel Deflectometer
SP	Superpave
SSD	Saturated Surface Dry
STI	Stitching
Gmm	Theoretical Maximum Specific Gravity
TRB	Transportation Research Board
UCP	Unbonded Concrete Overlay

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)

Appendix 2 - Standard International Conversion Factors

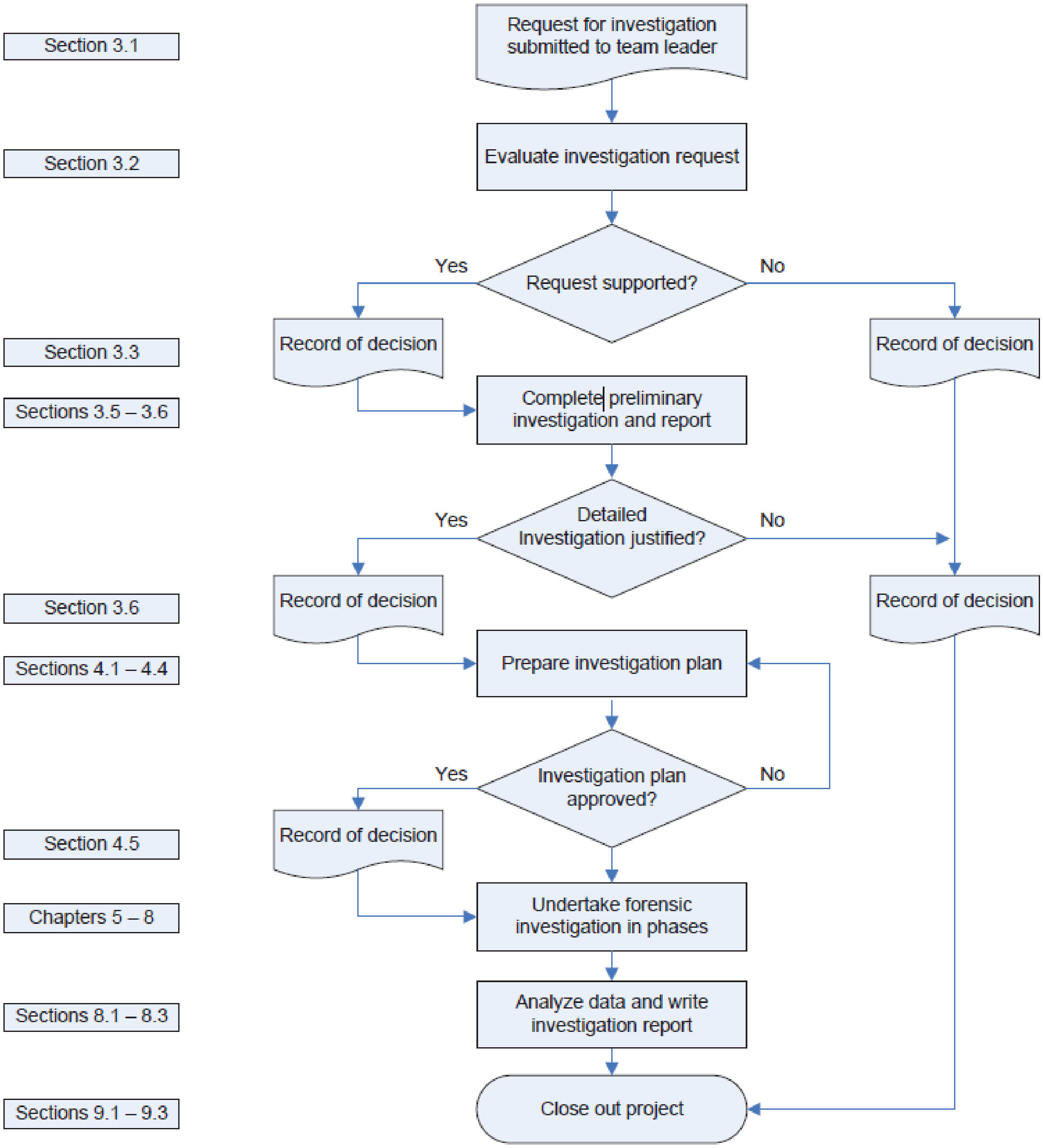

Figure 1.1. Recommended general approach to forensic investigations.

Appendix 3 - Report 747 - Recommended General Approach to Forensic Investigations (Rada, 2013)

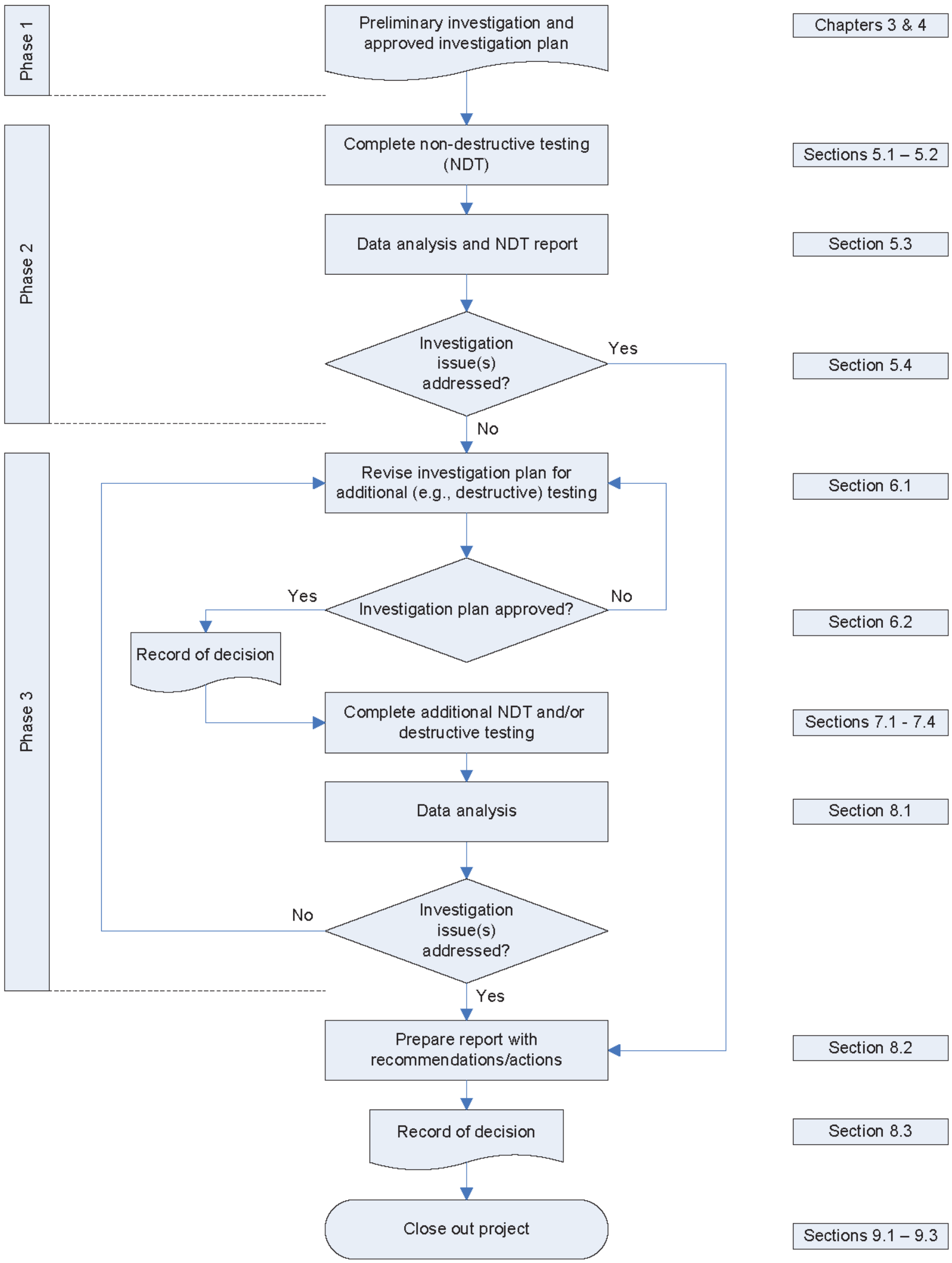

Figure 4.1. Phased approach to forensic investigations.

Appendix 4 - Report 747 - Phased Approach to Forensic Investigations (Rada, 2013)

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: Slight					Severe	Extent: <5				>80	Length	Width	Number	Location
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: Slight					Severe	Extent: <5				>80	Narrow (% area)	Wide (% area)	Position	Location
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: Good				Poor	Influencing factors									
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 5 - Report 747 - Visual Assessment Form for AC Pavements
(Rada, 2013)

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						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	Narrow	Wide	Position	
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						Extent					Narrow (% area)	Wide (% area)	Position	Location
	Slight				Severe		<5				>80				
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					Influencing factors									
	Good				Poor										
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 6 - Report 747 - Visual Assessment Form for PCC Pavement
(Rada, 2013)