INVESTIGATING PREMATURE PAVEMENT FAILURE DUE TO MOISTURE

FINAL REPORT

SPR 632

Oregon Department of Transportation

INVESTIGATING PREMATURE PAVEMENT FAILURE DUE TO MOISTURE

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by

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16. Abstract

This report details the forensic investigations conducted to identify the causes of pavement failures shortly after a rehabilitation activity on five interstate highway projects in Oregon, and the research efforts conducted to develop guidelines to minimize the risk of premature failures on future projects. One of the principal objectives of this research effort was to identify sources of moisture and other conditions that led to the early rutting problems observed along the five projects. Overall, improper tack coat or failure, permeable dense-graded layers, inadequate drainage, and, possibly, inadequate compaction of dense-graded material, were all identified as the likely root causes of the observed moisture damage and consequential rutting problems. The other principal objective was to evaluate design, construction, and materials requirements that will minimize the risk of such failures for future rehabilitation projects so that guidelines could be developed for these processes. In this respect, this report contains guidelines for the following:

- Pre-construction site investigations to identify the potential for moisture-related problems.
- Pavement structural design techniques that have been effective in reducing the risk of failures related to moisture damage.
- Construction techniques that can reduce the risk of failure due to moisture damage.
- Materials selection and testing to assist in reducing the risk of failure due to moisture damage.

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Symbol	When You Know	Multiply By	To Find	Symbol	Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH					**LENGTH**		
in	inches	25.4	millimeters	mm	mm	millimeters	0.039	inches	in
ft	feet	0.305	meters	m	m	meters	3.28	feet	ft
yd	yards	0.914	meters	m	m	meters	1.09	yards	yd
mi	miles	1.61	kilometers	km	km	kilometers	0.621	miles	mi
		AREA					**AREA**		
in^2	square inches	645.2	millimeters squared	mm^2	mm^2	millimeters squared	0.0016	square inches	in^2
ft^2	square feet	0.093	meters squared	m^2	m^2	meters squared	10.764	square feet	ft^2
yd^2	square yards	0.836	meters squared	m^2	m^2	meters squared	1.196	square yards	yd^2
ac	acres	0.405	hectares	ha	ha	hectares	2.47	acres	ac
mi^2	square miles	2.59	kilometers squared	km^2	km^2	kilometers squared	0.386	square miles	mi^2
		VOLUME					**VOLUME**		
fl oz	fluid ounces	29.57	milliliters	ml	ml	milliliters	0.034	fluid ounces	fl oz
gal	gallons	3.785	liters	L	L	liters	0.264	gallons	gal
ft^3	cubic feet	0.028	meters cubed	m^3	m^3	meters cubed	35.315	cubic feet	ft^3
yd^3	cubic yards	0.765	meters cubed	m^3	m^3	meters cubed	1.308	cubic yards	yd^3
NOTE: Volumes greater than 1000 L shall be shown in m^3.									
		MASS					**MASS**		
oz	ounces	28.35	grams	g	g	grams	0.035	ounces	oz
lb	pounds	0.454	kilograms	kg	kg	kilograms	2.205	pounds	lb
T	short tons (2000 lb)	0.907	megagrams	Mg	Mg	megagrams	1.102	short tons (2000 lb)	T
		TEMPERATURE (exact)					**TEMPERATURE (exact)**		
°F	Fahrenheit	(F-32)/1.8	Celsius	°C	°C	Celsius	1.8C+32	Fahrenheit	°F

*SI is the symbol for the International System of Measurement

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INVESTIGATING PREMATURE PAVEMENT FAILURE DUE TO MOISTURE

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

APPENDICES

The appendices are located online at:
http://www.oregon.gov/ODOT/TD/TP_RES/docs/Reports/2009/Moisture_Damage_Appendices.pdf

1.0 INTRODUCTION

1.1 PROBLEM STATEMENT

In the last several years a number of major interstate and smaller projects have exhibited pavement distress that appeared to be associated with moisture damage within months following a rehabilitation activity. The Oregon Department of Transportation (ODOT) Research Section contracted with Oregon State University (OSU) to conduct a thorough forensic investigation of five projects that have exhibited such distress. These projects were investigated to determine appropriate site investigation methods and testing to identify sources of moisture and other conditions that have led to premature failures; and, to evaluate design, construction, and materials requirements that will minimize the risk of premature failures related to moisture damage for future rehabilitation projects.

1.2 BACKGROUND AND SIGNIFICANCE OF WORK

Four projects on Interstate 5 and one on Interstate 84 (Figure 1.1) exhibited significant early distress that appeared to be associated with moisture damage. These were identified by the ODOT personnel as projects warranting detailed investigation. The following sections provide brief summaries of the rehabilitation activities and observed problems that occurred shortly after the rehabilitation activities for each of the projects.

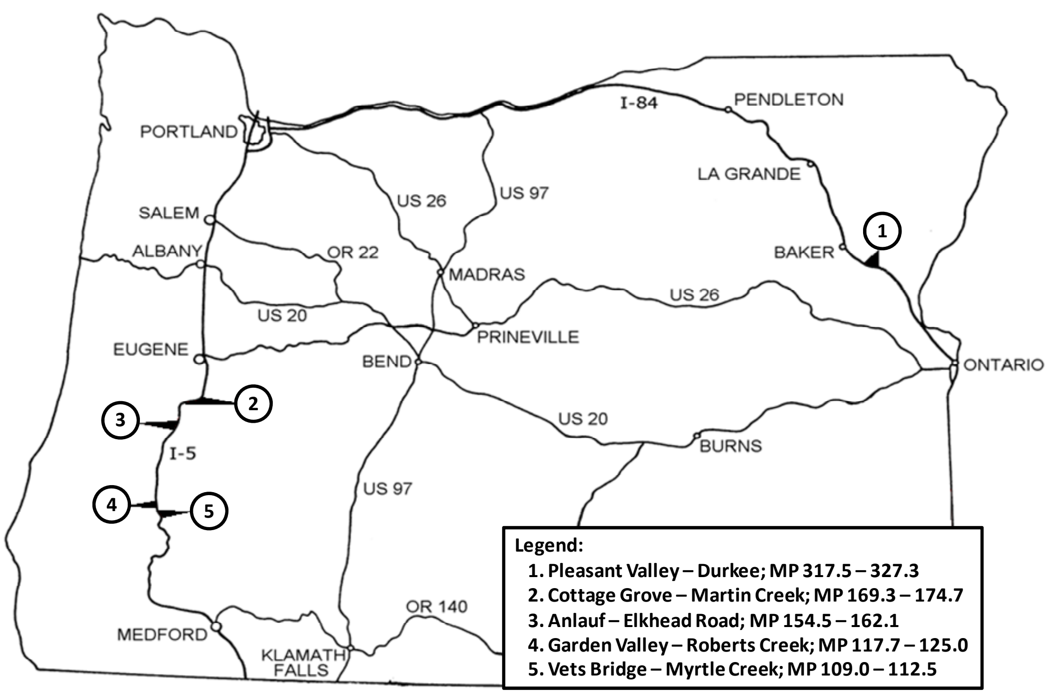


Figure 1.1: Project Locations.

1.2.1 Pleasant Valley – Durkee, Interstate 84 MP 317.5 – 327.3, Baker County; Year of Rehabilitation: 1999

Rehabilitation Activity. Originally it was planned to mill 2 inch (50 mm) of hot mix asphalt (HMA) and replace it with 2 inches (50 mm) of a 3/4 inch (19 mm) nominal maximum aggregate size (NMAS), dense-graded, lime-treated HMA, and then overlay this with 1½ inches (40 mm) of a 1/2 inch (12.5 mm) NMAS, lime-treated stone matrix asphalt (SMA).

Observed Problems. Within months of the initial rehabilitation activity (while the project was still under construction) and before the SMA wearing course was placed, several areas exhibited rutting to depths of 1 inch (25 mm) and greater. Trenching operations revealed stripping of the material and the presence of free water under the new inlay to a depth of about 5 inches (125 mm). Five inches (125 mm) of material from the outside lane was removed and replaced with a 3/4 inch (19 mm) NMAS, dense-graded, lime-treated HMA. Due to low truck traffic volumes on the inside lane, only 2 inches (50 mm) of material was removed and replaced on the inside lane. A 1½-2 inch (40-50 mm) SMA wearing course was placed over the entire roadway. Figure 1.2 illustrates the as-built profile of the bound layers. No effort was made at the time of these activities to determine the source(s) of water that contributed to the stripping. Within 6 months of final construction activities, water was observed to be leaking from longitudinal construction joints and in the wheel paths in several areas of the project. Rutting and potholes began forming in several of these areas within a short period thereafter. ODOT maintenance forces have ground and patched the failed areas. Several of the patches also failed.

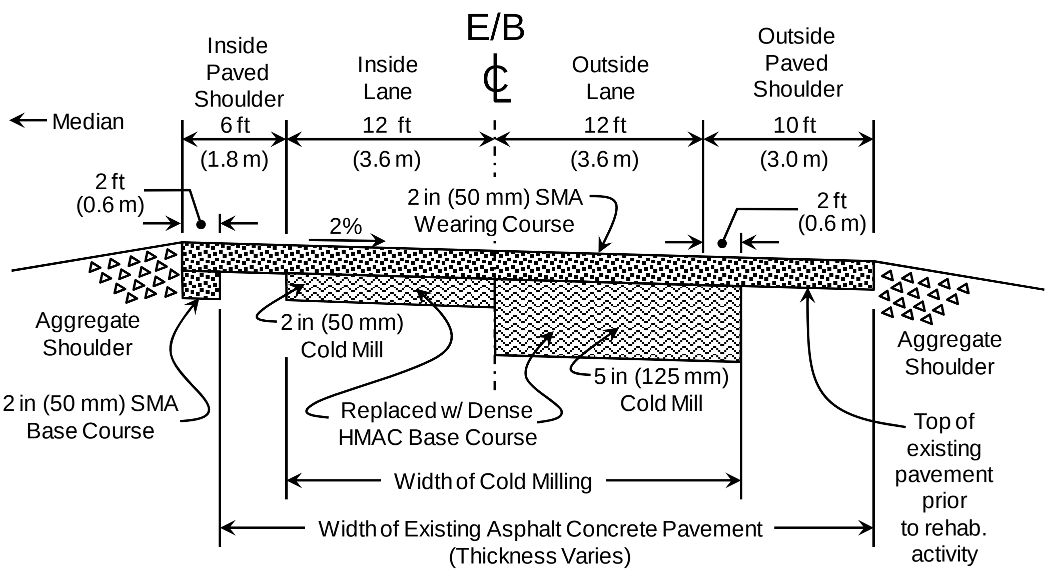


Figure 1.2: As-Built Profile of the Bound Layers for the Pleasant Valley – Durkee Project.

1.2.2 Cottage Grove – Martin Creek, Interstate 5 MP 169.3 – 174.7, Lane County; Year of Rehabilitation: 2000

Rehabilitation Activity. Two inches (50 mm) of the existing HMA surface was milled and replaced with a 3/4 inch (19 mm) NMAS, lime-treated, open-graded HMA as illustrated in

Figure 1.3. It should be noted that inclusion of lime in the mixture was not a requirement specified in the contract documents.

Observed Problems. Beginning in 2002, rutting to depths of 1 inch (25 mm) and greater began forming in isolated locations throughout the project. ODOT maintenance forces have ground and patched the distressed areas and observations during these activities suggested that the underlying material was the source of the failures.

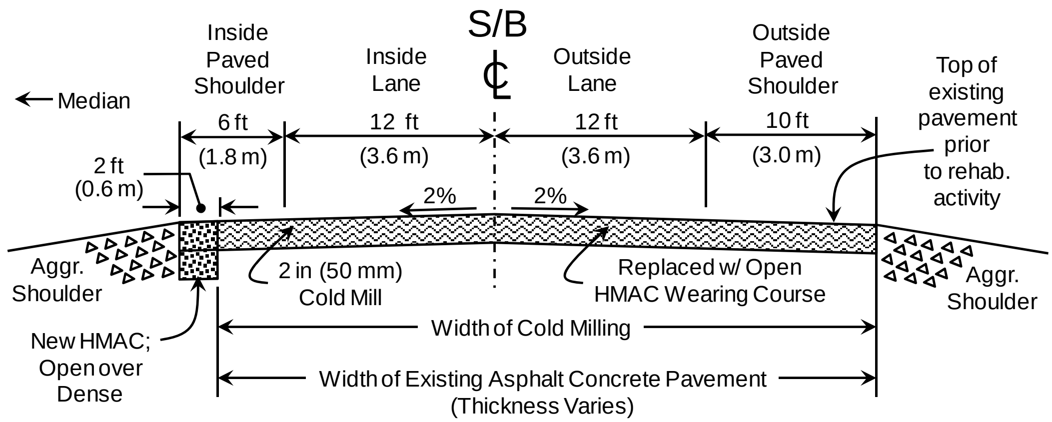


Figure 1.3: As-Built Profile of the Bound Layers for the Cottage Grove – Martin Project.

1.2.3 Anlauf – Elkhead Road, Interstate 5 MP 154.5 – 162.1, Douglas County; Year of Rehabilitation: 1997

Rehabilitation Activity. Two to three inches (50- 75 mm) of the existing HMA was milled from the outside (slow) lane and replaced with of a 3/4 inch (19 mm) NMAS, lime-treated, dense-graded HMA base course. The entire surface was then overlaid with 2 inches (50 mm) of a 3/4 inch (19 mm) NMAS, lime-treated, open-graded HMA. Figure 1.4 illustrates the as-built profile of the bound layers.

Observed Problems. Rutting to depths of 1 inch (25 mm) and greater began to form within 2 years following rehabilitation, principally within the inside lanes, but also in some isolated areas within the outside lanes. ODOT maintenance forces have ground and patched the distressed areas and observations during these activities suggested that the material under the newer material was failing.

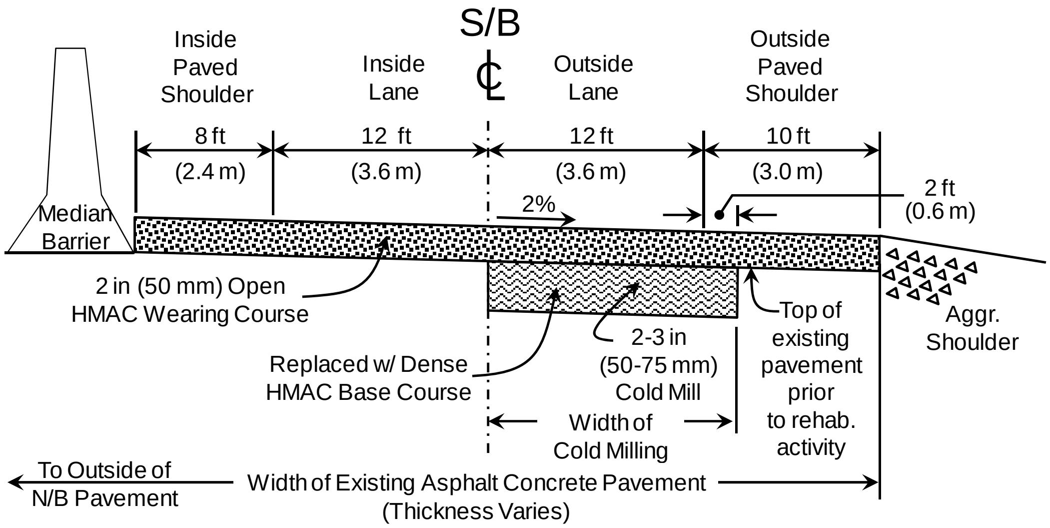


Figure 1.4: As-Built Profile of the Bound Layers for the Anlauf – Elkhead Road Project.

1.2.4 Garden Valley – Roberts Creek, Interstate 5 MP 117.74 – 125.0, Douglas County; Year of Rehabilitation: 2002

Rehabilitation Activity. Two inches (50 mm) of the existing HMA surface was milled from both travel lanes and replaced with 2 inches (50 mm) of a 3/4 inch (19 mm) NMAS, dense-graded HMA. The entire surface was then overlaid with 2 inches (50 mm) of a 3/4 inch (19 mm) NMAS, open-graded HMA as illustrated in Figure 1.5. It should be noted that inclusion of lime in the mixture was not a requirement specified in the contract documents

Observed Problems . Within approximately 6 months of the rehabilitation activity, rutting to depths of 1 inch (25 mm) and greater began forming in isolated areas. Additional rutting continued to form in isolated areas thereafter. No determination of the causes of the distress was made at the time the distress was first observed. ODOT has retained backup samples of the 3/4 inch (19 mm) NMAS, dense-graded HMA obtained during the production of the mixture.

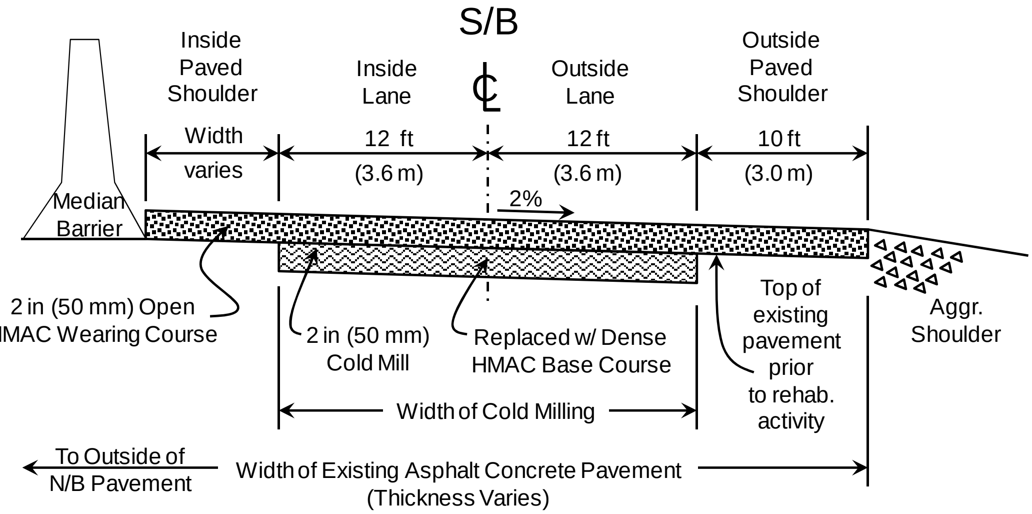


Figure 1.5: As-Built Profile of the Bound Layers for the Garden Valley – Roberts Creek Project.

1.2.5 Vets Bridge – Myrtle Creek, Interstate 5 MP 109.0 – 112.5, Douglas County; Year of Rehabilitation: 2003

Rehabilitation Activity. Three inches (75 mm) of the existing HMA surface was milled and replaced with 3 inches (75 mm) of a 3/4 inch (19 mm) NMAS, open-graded HMA as illustrated in Figure 1.6. Full depth HMA was placed where the pavement abuts the bridge ends. It should be noted that inclusion of lime in the mixture was not a requirement specified in the contract documents.

Observed Problems. Within approximately 9 months following the rehabilitation activity rutting to depths of 1 inch (25 mm) and greater began to form. Rutting continued to form in additional areas thereafter. Trenching operations at several locations revealed that the material immediately under the new 3/4 inch (19 mm) NMAS, open-graded HMA appeared to be stripping.

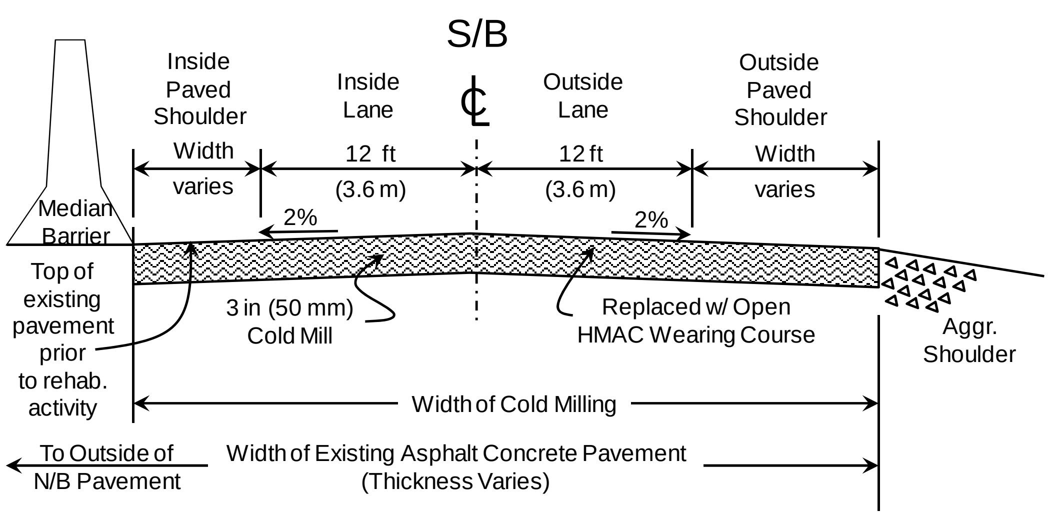


Figure 1.6: As-Built Profile of the Bound Layers for the Vets Bridge – Myrtle Creek Project.

Understanding the cause(s) of the distress exhibited by the above projects is significant in that such knowledge can be used to avoid similar problems occurring on future projects. The findings from this research project will potentially provide the requisite knowledge and guidelines for avoiding early rutting distress as a direct result of moisture damage.

1.3 OBJECTIVES OF THE STUDY

The principal objectives of this research effort were to identify sources of moisture and other conditions that led to the rutting problems in the five projects listed above; and, to evaluate design, construction, and materials requirements that will minimize the risk of such failures for future rehabilitation projects. More specifically, the objectives of this research effort are listed below:

1. Identify the potential cause(s) for the failure of each project.
2. Develop guidelines for pre-construction site investigations to identify the potential for moisture-related problems.
3. Develop guidelines for pavement structural design, construction techniques, and materials selection and testing when the potential for moisture-related problems exist.

1.4 BENEFITS

Several potential benefits are likely to be realized through implementation of the guidelines developed through this project. These include, but are not limited to, the following:

- Reduced risk of early failures due to moisture-related damage.
- Improved guidelines for investigating projects during the pre-construction phase to identify when the conditions contributing to moisture-related failures are present.
- Improved pavement structural design techniques and elements that can reduce the risk of moisture-related failures.
- Improved construction and materials specifications and testing that can reduce the risk of moisture-related failures occurring.
- Substantial savings in the cost of maintaining and/or rehabilitating pavements that would have failed due to moisture damage through implementation of measures to preclude the likelihood of moisture damage.

1.5 IMPLEMENTATION

The Pavement Design Unit will implement the recommended guidelines for cost-effective investigation techniques and structural design techniques immediately following acceptance of this report. Similarly, ODOT will implement the recommended guidelines for materials selection and testing and construction specifications immediately following acceptance of this report.

1.6 RESEARCH TASKS

Several specific work tasks were developed for this project in order to meet the objectives listed above. Briefly, these included the following:

1.6.1 Task 1 – Literature Review

A literature review was conducted to address the following:

- Moisture damage in HMA pavements.
- Site investigation techniques to identify potential conditions leading to moisture damage.
- Pavement structural design techniques to mitigate the potential for moisture damage.
- Best practices for construction techniques, and materials selection and testing to mitigate the potential for moisture damage.

Section 2.0 of this report provides a synopsis of the findings from the literature review.

1.6.2 Task 2 – Site Visits and Maintenance Personnel Interviews

Each project was visited to obtain information to aid in determining the cause of distress as well as to assist in the development of a more comprehensive field investigation methodology. Section 3.0 of this document provides a summary of this effort.

1.6.3 Task 3 – Records Review

A records review was conducted, with the assistance of ODOT contract administration personnel, to obtain available information for each of the five projects. Section 3.0 of this report provides a summary of the records review.

1.6.4 Task 4 – Interim Report

An interim report was developed to summarize the information obtained in Tasks 1, 2, and 3 as well as to propose an experiment plan for field and laboratory investigations to verify the cause(s) of failure in the five projects listed above.

1.6.5 Task 5 – Conduct Experimental Plan

Section 3.0 and 4.0 of this document provide details regarding the field and laboratory investigations.

1.6.6 Task 6 – Final Report

This document constitutes the final report for the study.

2.0 LITERATURE REVIEW

An extensive literature review was undertaken to gather information pertaining to forensic investigations of pavement failures as well as current materials requirements and design and construction practices utilized to minimize the risk of moisture-related damage in HMA pavements. This section provides a synthesis of the findings, whereas Appendix A provides a more comprehensive summary of the literature. The findings, together with the field investigations (Section 3.0) and laboratory study results (Section 4.0), formed the basis for the guidelines presented in Section 6.0 of this report.

2.1 MOISTURE DAMAGE IN HOT MIX ASPHALT PAVEMENTS

In a general sense, moisture damage in hot mix asphalt pavements can be defined as the loss of strength and durability due to the effects of moisture (*Little and Jones 2003*) caused by loss of cohesion (strength) of the asphalt film, failure of the adhesion (bond) between the aggregate and asphalt, and degradation of the aggregate particles subjected to freezing (*Terrel and Al-Swailmi 1994*). Moisture damage is commonly manifested in the form of stripping as a result of detachment, displacement, spontaneous emulsification, pore pressure, and hydraulic scour (*Majidzadeh and Brovold 1968; Fromm 1974; Taylor and Khosla 1983; Kiggundu and Roberts 1988; Tarrer and Wagh 1991; Terrel and Al-Swailmi 1994; Cheng et al. 2002; Birgisson et al. 2005*). Table 2.1 summarizes the numerous factors that affect the susceptibility of HMA mixtures to moisture damage (*Stuart 1990, Hicks 1991*).

Moisture damage generally starts at the bottom of an asphalt base layer or at the interface of two asphalt layers (*Khosla et al. 1999*). Eventually, localized potholes are formed or the pavement ravels or ruts. With hardened binders, localized fatigue cracking (longitudinal cracking that progresses to alligator cracking) may occur resulting in a weakened pavement structure. Subsequent water intrusion into these localized water-damaged areas, coupled with traffic loading, further degrades the structural integrity of the pavement layer, and possibly the underlying layers which, if not repaired, can lead to substantial localized failure of the pavement structure (*Scholz 1995*). Surface raveling or a loss of surface aggregate can also occur, especially with chip seals. Occasionally, binder from within the pavement will migrate to the pavement surface resulting in flushing or bleeding (*Stuart 1990*).

2.2 PRE-CONSTRUCTION SITE INVESTIGATIONS

The principal goal of a pre-construction investigation is to determine the mechanism(s) related to observed pavement distresses as well as the events that may have led to the problems. Such an investigation should find which mechanisms caused the distress and rule out any unrelated mechanisms (*Crampton 2001*). The important tasks involved in a pre-construction investigation are shown below in Table 2.1.

Table 2.1: Factors Influencing Moisture Damage

Major Factors	Description
Aggregate Properties	▪ Composition (degree of acidity or pH, surface chemistry, type of minerals, source of aggregate) ▪ Physical characteristics (angularity, surface roughness, surface area, gradation, porosity, and permeability) ▪ Dust and clay coatings ▪ Moisture content ▪ Resistance to degradation
Asphalt Binder Properties	▪ Grade or stiffness ▪ Chemical composition ▪ Crude source and refining process
HMA Mixture Characteristics	▪ Air void level and compaction ▪ Type of HMA (dense-graded, gap-graded, open-graded)
Environmental Factors	▪ Temperature ▪ Freeze-thaw cycles ▪ Moisture vapor ▪ Dampness ▪ Pavement age ▪ Micro organisms ▪ Presence of ions in the water
Traffic	▪ Percent of trucks ▪ Gross vehicle weight of trucks ▪ Truck tire pressure
Construction of HMA Pavements	▪ Compaction ▪ Drainage ▪ Weather ▪ Segregation ▪ Contractor experience
Design of HMA Pavements	▪ Air void content ▪ Subsurface drainage ▪ HMA mix selection ▪ Designer experience ▪ Designer site visit

2.2.1 Records Review

Records review refers to the data that are be collected from office files and historical records that can be an invaluable aid in the investigation process. Based on the literature, data that were considered essential for identification of probable failure of pavements due to moisture damage (stripping) included (*ASCE 1986; Victorine 1997; Scullion 2001; Crampton 2001; Kandhal 2001; Zhang 2002; and Caltrans 2003*):

- Pavement history: date of construction of pavement overlays, repairs, and maintenance information; past distress data; and non-destructive test results.
- Pavement structure: layer thickness and material properties.
- Pavement material information: HMA mix design; aggregates; asphalt binder; and base layer material information.

- Traffic information: Average daily traffic (ADT); and equivalent single axle loads (ESALs)
- Description of distress: type; severity; and extent.
- Relevant construction records: drawings; specifications; schedule; as-built drawings; QC/QA records; lab material test results; and drainage.
- Weather records: rainfall; weather during construction; and temperature data.
- Soil and geologic information: classification; geologic origin; and terrain.

2.2.2 Site Visit and Condition Survey

The principal goal of the initial site visit and condition survey is to inspect the pavement section under investigation to obtain information about the distresses present in the pavement. This should include type, severity, and extent of each distress and these should be drawn on a plan view map (or straight-line chart) of the pavement section, preferably with other features such as drainage systems, structures and culverts, and surrounding topography and hydrology. Once the distresses have been identified, the potential locations and methods for field investigations and sampling can be determined. The onsite investigation also gives the investigative team additional support in developing alternate rehabilitation strategies by considering the existing local conditions and restrictions that will influence the final decision (*Victorine 1997*).

2.2.3 Field Testing

The principal objective of a field investigation is to determine the in situ properties of pavement layers, which may differ from the expected (designed) properties. Scullion (*2001*) identifies two broad categories field investigation methods: 1) non-destructive testing, and 2) destructive testing. Non-destructive testing is used to examine a pavement without impairing its future usefulness. Non-destructive methods include conditions surveys, falling weight deflectometer (FWD) surveys, ground penetrating radar (GPR) surveys, Portable Seismic Pavement Analyzer (PSPA) surveys, and Automated Road Analyzer (ARAN) surveys. Destructive testing involves destruction of part or all of the pavement section, necessitating repair of the affected pavement. Coring, dynamic cone penetration (DCP) testing, and trenching are examples of destructive testing used for pavement investigations. Table 2.2 summarizes several field evaluation methods that have been suggested to investigate pavements with distresses related to moisture damage (*Scullion 2001*).

2.2.4 Laboratory Testing

Samples to be tested in the laboratory should be obtained at the time the field investigations are undertaken. For evaluating moisture-damaged pavements, Kandhal (*1994*) recommends obtaining at least seven 4 inch (100 mm) diameter cores from random locations within two 500 ft sections with one representing a typical “distressed area”, and the other representing a relatively “good area.” Kandhal emphasizes the necessity of using CO_2 or compressed air, rather than water, to cool the core barrel so that moisture content tests can be performed on the cores. If dry coring cannot be accomplished, Kandhal recommends obtaining samples using a jackhammer.

Irrespective of the type of sample, it is recommended that they be sealed in air-tight containers for transport to the laboratory.

Table 2.2: Field Investigation Methods to Evaluate Water Damage (*Scullion 2001*)

Pavement Distress	Evaluation Method
Rutting (probable outcome of stripping)	▪ Condition survey, extent and severity of problem ▪ Drainage ▪ GPR survey (moisture in base, stripping in HMA, layer thickness) ▪ FWD survey (layer moduli) ▪ DCP survey (strength profile in base and subgrade) ▪ Samples (from rutted and non-rutted areas, cores and/or bag samples) ▪ Trenching (trench to identify problem layer)
Alligator cracking (probable outcome of stripping)	▪ Condition survey, extent and severity of problem ▪ Drainage ▪ GPR survey (moisture in base, stripping in HMA, layer thickness, layer bonding) ▪ FWD survey (layer moduli) ▪ DCP survey (strength profile in base and subgrade) ▪ Samples (from areas with and without cracks, cores and/or bag samples) ▪ Bonding between layers (observation from coring, slab removal, Seismic Pavement Analyzer Test (SPA) test)
Longitudinal cracking (probable outcome of stripping)	▪ Condition survey, extent and severity of problem ▪ Drainage ▪ Undisturbed samples of fill and subgrade foundation soils ▪ Geometric factors: lane, paved or unpaved shoulder, side slopes ▪ Other: presence of trees close to pavement edge
Raveling (probable outcome of stripping)	▪ Condition survey, extent and severity of problem ▪ HMA cores for laboratory evaluation

In a case histories study of premature failures of asphalt overlays (three in the US and one in Australia) Kandhal and Richards (*2001*) found that saturation of the pavement and asphalt layers was the root cause of the stripping problems. They determined that forensic investigations should include an assessment of the moisture conditions in failed areas and in areas without failure. Kandhal (*1994*) recommends determining the in situ moisture conditions of the asphalt layers by weighing the cores before and after air-drying the cores. Once the moisture conditions are determined, he recommends sawing the cores so that additional testing can be performed on individual layers of lifts, including bulk specific gravity, indirect tensile strength, and visual assessment of stripping performed on the split specimens.

Scullion (*2001*) compiled a list of recommended tests based on the type of observed distress. Table 2.3 summarizes some of the laboratory evaluation methods suggested in the literature to investigate distress due to moisture damage.

Table 2.3: Lab Evaluation Methods to Investigate Pavement Distress

Rutting (probable outcome of stripping)	▪ HMA properties	▪ Hveem stability, water susceptibility, condition ▪ Asphalt content, asphalt penetration, air void content ▪ Aggregate properties (gradation, absorption, shape, surface texture) ▪ Wheel tracker performance (Asphalt Pavement Analyzer, Hamburg Wheel Tracker) ▪ Repeated load test
	▪ Base, Subbase, Subgrade	▪ Gradation, field moisture content ▪ Tri-axial classification and tube suction test (moisture susceptibility)
Alligator cracking (probable outcome of stripping)	▪ HMA properties	▪ Moisture susceptibility ▪ Asphalt content, asphalt penetration, air void content ▪ Aggregate properties (gradation, absorption, shape, surface texture)
	▪ Base, Subbase, Subgrade	▪ Gradation, field moisture content ▪ Tri-axial classification and tube suction test (moisture susceptibility)
Longitudinal cracking (probable outcome of stripping)	▪ HMA properties (segregation of HMA near crack)	
Raveling (probable outcome of stripping)	▪ HMA Properties ▪ Air voids	▪ Asphalt content ▪ Asphalt properties (penetration, viscosity) ▪ Aggregate properties (gradation, absorption, shape, surface, texture, mineralogy) ▪ Moisture susceptibility

2.2.5 Data Analysis

This step involves reviewing all evidence relating to the project in order to come up with the most reasonable explanation for the failure. Some of the questions that should be asked in order to compare the data obtained from the field to form evidence on the probable causes of failure are (*Crampton 2001*):

- What did the industry standard call for?
- What did the design documents call for?
- What was actually constructed?
- What changed after construction?

However, even after the completion of all testing and analyses are completed, uncertainties often remain. In that case, through a combination of previous experience and engineering principles, the most likely cause of the problem must be determined (*Victorine 1997*).

2.2.6 Summary Report

Reports should include items such as the project history and background, a description of pavement structure, and a description of material types. A detailed description of the pavement condition, the types of distress involved, and the failure modes should also be included. Environmental conditions, soil conditions, traffic history data, and traffic projections must be included. A summary of the evaluation and testing strategies used for the investigation, as well as the findings of these tests, should also be presented. Finally, a prioritized summary of possible corrective strategies and their associated costs should be included (*Victorine 1997*).

2.3 DESIGN PRACTICES

The design and construction of HMA overlays are the most widely-used method for rehabilitation of HMA pavements in the United States. They provide a relatively fast, cost-effective means of correcting existing surface deficiencies, restoring user satisfaction and (depending on the thickness) adding structural load-carrying capacity (*Sebaaly et al. 1997; NHI and FHWA 2001*). Some of the most important steps to be considered while designing an HMA overlay include (*Sebaaly et al. 1997a*):

1. Identify the complete history of the pavements section;
2. Identify the traffic requirement;
3. Survey the conditions of the project;
4. Conduct nondestructive testing;
5. Conduct overlay design analysis; and
6. Evaluate alternatives and make final recommendations.

The first four steps were covered previously in Sections 1.2.1 and 1.2.3. The following paragraphs provide an overview of the key considerations found in the literature for designing HMA overlays including:

- Cold milling depth,
- Overlay design procedures,
- Drainage design, and
- HMA mix design.

2.3.1 Cold Milling Depth

For mill and overlay (inlay) rehabilitation strategies, the depth of milling is an important factor to consider in the design process. Based on a research by Wu et al. (*2000*) for Kansas Transportation Authority and Kansas Department of Transportation, it was concluded that in order to achieve higher fatigue life, the mill-and-inlay thickness should be at least 1.25 times the thickness of the remaining milled pavement layer. The literature search did not reveal other research on the selection of milling depth on HMA pavements.

2.3.2 Overlay Design Procedure

Although several methods exist for determining overlay thickness including simple engineering judgment or policy decisions and designed based on structural deficiency or limiting deflection (*Crovetti 2005*), Oregon is currently in the process of transitioning to the new mechanistic-empirical design procedure developed under NCHRP Project 1-37A (*ARA 2004*).

2.3.3 Drainage Design

To prevent moisture damage in rehabilitated pavements, one of the most important steps in HMA overlay design is the design of drainage systems. To obtain adequate pavement drainage, the designer should consider providing three types of drainage systems: 1) surface drainage, 2) groundwater drainage, and 3) structural drainage (*AASHTO 1993)*. Failure of any one of these will allow the moisture to stay in the pavement and lead to moisture-related distress such as stripping, rutting, potholes, etc. The pavement design engineer should conduct a detailed drainage survey to evaluate the existing drainage conditions. Drainage evaluation requires investigation of the problem site, preferably during a wet weather period. According to AASHTO (*1993*), the following is a partial list of questions to ask during the site investigation:

- Where and how does water move across the pavement surface?
- Where does water collect on and near the pavement?
- How high is the water level in the ditches?
- Do the joints and cracks contain any water?
- Does water pond on the shoulder?
- Does water-loving vegetarian flourish along the roadside?
- Are deposits of fines or other evidence of pumping (blowholes) visible at the pavements edge?
- Do the drainage system inlets contain debris or sediment buildup?
- Are the joints and cracks sealed effectively?

If visual observations suggest a significant drainage deficiency may exist, more intensive inspection may be conducted. If edge drains are present, their effectiveness should be evaluated by observing their outflow either after a rainfall or after water is released from a water truck over pavement discontinuities. The flow from each outlet should be examined and any outlets that are flowing at a much lower rate than the others should be noted on a strip map (*NHI and FHWA 2001*).

Another way of assessing the effectiveness of edge drains is through the use of video inspections (*Christopher 2000*). This inspection technique uses a high-resolution, high-sensitivity color video camera attached to a pushrod cable approximately 15 mm (0.6 in) in diameter and 150 m (500 ft) long (*Daleiden 1998*). The device is inserted into the drainage system at the outlets, and as the camera is pushed along, it records the inspection in progress and simultaneously notes the distances that the camera has advanced (*Daleiden 1998*). In this way, any blockages, rodents' nests, or areas of crushed pipes can be located. Several states have now adopted video edge drain inspection as part of new drainage construction (*NHI and FHWA 2001*).

In addition, the following drainage-related items should be noted as part of the drainage survey (*NHI and FHWA 2001*):

- Topography of the project.
- Transverse slopes of the shoulder and pavement.
- Condition of the ditches.
- Geometrics of the ditches.
- Condition of drainage outlets (if present).
- Condition of drainage inlets (if present).

All of the information collected from the drainage surveys should be marked and noted on strip maps, and then examined together to obtain a visual picture of what moisture is doing to the pavement, whether any moisture damage is occurring, and what factors are present that allow the moisture damage to occur. In particular, AASHTO (*1993*) suggests the following questions:

- Is the original drainage design adequate for the existing road?
- What changes are necessary to ensure drainage inadequacies, which may contribute to structural distress, are corrected?
- If the original drainage system design was adequate, have any environmental or structural changes taken place?
- Does the present or projected land use in areas adjacent to the road indicate any change in flow pattern of surface drainage or likely to change, thus rendering existing drainage facilities inadequate?

Subsurface drainage systems should be installed at locations suitable for easy removal of water the pavement. If there is no groundwater to be removed, shallow trenches are adequate. If frost heave is a problem, the drainage system should be deep enough to keep groundwater at least 0.9 meters below the pavement structure (*Terrel 1990*). In HMA overlay projects where road widening is necessary, it is essential to provide uninterrupted drainage of the base layer under the pavement. If the widening is to extend to the edge of the shoulder, an open-graded asphalt treated permeable material may be used as drainage layer under the widened portion (*Terrel 1990*).

2.3.4 HMA Mix Design

A vital component in the flexible pavement overlay process is the design of the HMA mixture. The mixture design consists of selecting and proportioning the aggregates and asphalt binder to produce a mixture that will provide high durability, workability, and stability. The majority of state highway agencies in the US utilize the Superpave mix design method in an attempt to accomplish this. However, some state highway agencies still allow the use of either the Marshall or Hveem mix design methods.

2.4 CONSTRUCTION PRACTICES

The production of HMA mix and the subsequent construction of the pavement play a major role in the prevention of moisture sensitivity of HMA pavements. The major construction steps involve:

- Surface preparation of existing pavement,
- HMA production,
- Paving operations, and
- Compaction.

2.4.1 Surface Preparation

The amount of repair and treatment that is performed to a pavement prior to overlay is probably the single most important factor that affects the future performance of the overlay (*NHI and FHWA 2001*). Surface preparation (or pre-overlay treatment) of existing pavements can include activities such as removing a top layer (fully or partially) through milling, applying a leveling course, repairing localized areas, surface leveling (rutting), control of reflection cracking, drainage improvements, etc. Irrespective of the type of treatment, the surface should be thoroughly cleaned by sweeping or washing, and allowed to dry, prior to placement of the tack coat and overlay.

Cold milling the pavement surface is commonly used prior to placement of an overlay, particularly where it is necessary to maintain vertical clearance under structures or remove distressed pavement. The literature recommends the following practices when cold milling is performed as part of the rehabilitation activity (*NHI and FHWA 2001; Roberts et al. 1996; WAPA 2002*):

- It is recommended that failed pavement areas be patched prior to cold milling, as cracking becomes difficult to locate on the milled surface.
- The forward speed of the milling machine, the rotational velocity of the rotating drum, the spacing of the carbide bits, and the grade control of the cutting head should be closely controlled to produce a uniform texture throughout the project.
- The longitudinal profile should be held to the same tolerance as new base course construction if the pavement is to be overlaid.
- After a pavement has been milled, the resulting surface is quite dirty and dusty. The surface should be cleaned by sweeping or washing before any overlay is placed; otherwise, the dirt and dust will decrease the bond between the new overlay and the existing pavement. When sweeping, more than one pass is typically needed to remove all the dirt and dust. If the milled surface is washed, the pavement must be allowed to dry prior to paving.

- Transverse slope of the pavement is also an important factor and should be specified to obtain proper drainage. Generally, a transverse slope of 2 percent, or 64 mm (2.5 in) over 3.66 m (12 ft), is recommended.
- Grinding limits and transitions or stop lines at bridges and ramps should be clearly marked on the plans.

2.4.2 HMA Production

Aggregate Stockpiling. Segregation of aggregates and moisture absorption during stockpiling can lead to a moisture sensitive HMA mix. Hence, it is necessary to take certain measures during aggregate stockpiling to achieve an HMA mix that meets the design criteria. Some of the best practices to prevent aggregate segregation and moisture absorption as reported by St. Martin et al. (*2003*) include the following:

- The foundation for aggregate stockpiles should be stable so that the construction equipment can efficiently build the stockpiles and remove material from the stockpiles.

- The foundation for aggregate stockpiles should be clean to ensure that foreign materials, such as roots, soil, or grass, are not picked up during aggregate hauling. Vegetation, soft particles, clay lumps, excess dust and vegetable matter may affect performance through loss of structural support and/or prevents binder-aggregate bonding (*WAPA 2002*).

- Foundations should be constructed such that water does not pond underneath the stockpile which would increase the moisture content of the aggregates near the bottom of the stockpile.

- There should be sufficient space between the stockpiles so that cross-contamination between stockpiles does not occur.

- Stockpiles should be built to ensure that the moisture content within the stockpile stays as low and consistent as possible. A method of preventing water from infiltrating into the stockpile is to cover the stockpile using some type of a roof structure. Tarps are generally not recommended for covering stockpiles because moisture tends to collect under the tarp.

- Proper handling/hauling techniques should be used to minimize segregation. Excessive handling of the aggregates can also cause degradation of the aggregates, which causes a change in the gradation of the stockpile.

Cold Feed System. The cold feed system includes cold feed bins, a collecting conveyor, and a charging conveyor. To produce a uniform, high-quality HMA, it is imperative that the entire cold feed system needs to be properly calibrated.

Loading the Mixture. Two important considerations while loading the HMA mix in the trucks are: 1) proper charging of HMA mix into trucks, and 2) truck bed cleanliness and lubrication. Improper charging of the truck bed can lead to mixture segregation, which can lead to increased permeability within the completed pavement (*Roberts et al. 1996*). To minimize the risk of segregation, WAPA (*2002*) recommends loading the HMA into the trucks beds in several masses (three drops are typical) at different locations in the truck bed. For truck bed lubrication, non-petroleum based products such as lime water, soapy water or other suitable commercial products have been recommended (*Roberts et al. 1996*).

Transportation. The two major concerns during transport of HMA mixtures are cooling of the mixture and asphalt binder drain-down. If the HMA mix cools below a certain temperature, it will be difficult to achieve proper density on the roadway, potentially allowing water to permeate into the finished pavement following compaction (*St. Martin et al. 2003*). The exposed, exterior mass of the mixture tends to lose heat first resulting in a thin crust on the surface that insulates the hotter, interior core. The resulting temperature differential can cause isolated areas of inadequate compaction resulting in decreased strength, accelerated aging/decreased durability, rutting, raveling, and moisture damage (*Hughes 1984*). Generally, temperature differentials greater than about 25°F can potentially cause compaction problems (*WAPA 2002*). Minimizing haul distances, utilizing trucks with insulated beds, and covering the mixture with a tarpaulin have been suggested by WAPA (*2002)* to minimize cooling of the mixture. A material transfer vehicle (MTV) can also be used ahead of the paver to remix the cooler crust of mix with the hotter, interior mass to achieve a more uniform temperature of the mix prior to paving.

The other major concern during transport involves drain-down of the asphalt binder, typically associated with open-graded and stone matrix asphalt mixtures, which can result in insufficient coating of the coarser aggregate. As a result, moisture damage can occur owing to displacement, detachment, or hydraulic scour in the presence of water (*St. Martin et al. 2003*).

Paving Operations. Some of the best practices identified in the literature include the following:

- The HMA mix should be unloaded quickly when it arrives at the paving site. This will minimize the mix cooling before it is placed. The supply of mix to the paving train should not be such that there are an excessive number of trucks waiting to empty. As the trucks wait, the mixture cools.

- Before HMA is loaded into the paver, the inspector and/or foreman should make sure that it is the correct mix. Occasionally, paving jobs require more than one mix design (i.e., one for the leveling course and one for the wearing course) and these mixes should not be interchanged.

- The hopper should never be allowed to empty during paving. This results in the leftover cold, large aggregate in the hopper sliding onto the conveyor in a concentrated mass and then being placed on the mat without mixing with any hot or fine aggregate. This can produce aggregate segregation or temperature differentials, which will cause isolated low densities in the mat. If there are no transport vehicles immediately available to refill the hopper, it is better to stop the paving machine than to continue operating and empty the hopper (*TRB 2000*). A recent study by Gilbert (*2005*) reported that dump trucks used without material transfer vehicles are prone to temperature segregation. The author also suggested that windrow elevators appear to work on par with MTVs.

Compaction. Once placed on the roadway, the mix is compacted in an attempt to achieve a desired (specified) in-place density. Hughes (*1989)* indicated that inadequate compaction results in a pavement with decreased stiffness, decreased durability (accelerated aging), and increased likelihood of rutting, raveling, and moisture damage. For dense-graded mixes, numerous studies have shown that initial in-place air void content should not be below approximately 3% or above approximately 8% (*St. Martin et al. 2003*).

Excessive rolling during mix compaction and/or using a heavier roller than needed may also result in moisture damage. Either factor may cause fracturing of the aggregate. In the presence of water, the fractured aggregate can absorb water and lead to displacement of the asphalt film (*St. Martin et al. 2003*).

The longitudinal joint is a highly susceptible location to allow water into the pavement structure. In the field, longitudinal joints are usually constructed to a lower density than the interior portion of the mat. This lower density at the joints is a result of compacting unconfined edges, not properly pinching the joint with the roller, and so forth (*St. Martin et al. 2003*).

Pavement Drainage. NCHRP Report 96 (*Ridgeway 1982*) and more recently NCHRP Report 239 (*Christopher and McGuffey 1997*) discuss in detail the importance of subsurface drainage of pavements. Some of the common drainage problems identified during the construction process include:

- poor control of grades, which leaves water pooled in the pipes;
- guide and guardrail posts driven through drains and outlet pipes;
- pipes and other parts of the facility crushed and collapsed by construction traffic;
- altered drainage outlet spacing;
- headwalls that tilt backward;
- bad or poor headwall connections;
- improper use of connectors;
- high ditch lines that do not allow proper drainage from outlets; and
- outlets that have been left out altogether.

NCHRP Report 239 indicated that most problems with subsurface drainage facilities originate in the construction phase. It also reported that proper training of the construction staff may avoid the problems identified above. The study reported some best practices on the installation of edge drains including the following:

- Proper pipe grade control is essential for edge drains to be effective. Undulating drain lines are not acceptable.
- Drains should be properly connected to the outlets.
- Drain lines are to be carefully marked and proper care should be given throughout construction to avoid crushing the pipe with construction equipment.

- When drains are installed after constructing the pavement, temporary drainage is required for the permeable base to prevent a bathtub effect from water trapped in the porous base.
- Backfill material for edge drains needs to be at least as permeable as the permeable base. The drainage backfill should be placed below the invert of the pipe and compacted to better support the pipe.
- The edge drain system should be inspected and tested for proper operation toward the end of construction, before final acceptance. Acceptance criteria based on performance parameters must be established, otherwise signs of poor construction practices most likely will not be identified until major structural damage occurs.

QC/QA. Quality control/quality assurance is one of the most important aspects of HMA production and construction process. The three major objectives of a quality control/quality assurance system are to:

- Produce a quality product,
- Assure that the final product meets job specification, and
- Satisfy the customer's needs, as economically as possible.

In order to meet these objectives, a typical QC/QA program will have various sampling and testing procedures are presented in Table 2.4.

Table 2.4: QC/QA Sampling and Testing Procedures

Testing and Sampling Stage	Testing required
Pre-production	• Plant considerations • Aggregate, asphalt cement, and additives for mix design • Consider anticipated process adjustments • Cause and effect • Economics
Job mix formula approval and verification	• Aggregate gradation • Aggregate physical properties • Asphalt content • Volumetric analysis • Stability or strength testing, where applicable • Moisture susceptibility
QC during production by contractor	• Aggregate gradation • Asphalt content • Volumetric analysis • In-place density
Production and in-place acceptance by owner	• Random production and in-place acceptance testing by the owner is similar to contractor testing • Measurements of thickness, smoothness, overall profile and workmanship

Some of the tools used to ensure the quality of HMA in the field include: 1) QC plans, 2) checklists, 3) daily dairies, and 4) feedback systems (*Russell et al. 2001*).

2.5 MATERIALS REQUIREMENTS

Proper material selection and testing are critical to obtain a desirable HMA mix that will resist the moisture damage in pavements. Before production in the hot-mix plant, component materials are often tested to ensure that they have the same physical properties desired in the mix design (*Russell et al. 2001*). The literature was reviewed to document the best practices in material selection and testing that has been reported from previous studies.

2.5.1 Aggregate Selection and Testing

Aggregates compromise approximately 92% to 96% of the volume of HMA (depending to mix type). Hence, the aggregate properties are very important to the performance of flexible pavements. Often, pavement distress such as rutting, stripping, surface disintegration, and lack of adequate surface frictional resistance can be directly attributed to improper aggregate selection and use (*Kandhal et al. 1997*). Some of the properties/characteristics of aggregates that have been reported in literature that contributed to moisture damage include (*Khosla et al. 1999; Birgisson et al. 2005; Stuart 1990; Parker 1989; Kandhal 1994*):

- Degradation of aggregate.
- High moisture contents in the mineral aggregates before mixing with the asphalt binder.
- Excessive dust coating on the aggregate can prevent thorough coating of asphalt binder on the aggregate.
- Siliceous aggregates which often have slick, smooth areas, may give rise to stripping, while roughness may help to promote bonding.
- Interlocking properties of the aggregate particles, which include individual crystal faces, porosity, angularity, absorption, and surface coating, are also believed to improve the bond strength in an asphalt mixture.
- Aggregates that impart a low pH value to water.

Aggregates have to pass a stringent series of mechanical, chemical, and physical tests in order to demonstrate that they will perform satisfactorily, and meet or exceed specifications in a HMA mixture. A study conducted by Kandhal (*1997*) reported the variety of tests used by the various state highway agencies in the US. The author listed the aggregate tests for HMA as follows:

- Particle Shape and Surface Texture (Coarse Aggregate).
- Particle Shape and Surface Texture (Fine Aggregate).
- Porosity or Absorption.
- Cleanliness and Deleterious Material.
- Toughness and Abrasion Resistance.
- Durability and Soundness.
- Expansive Characteristics.
- Polishing and Frictional Characteristics.

Kandhal et al. (*1998*) conducted a study to determine the best test method for aggregates to study the presence of detrimental plastic fines in the fine aggregate, which may induce stripping in HMA mixtures. The study concluded that AASHTO TP57 (methylene blue test) is the best test for fine aggregate in determining the propensity for stripping in HMA.

2.5.2 Anti-Stripping Agents

Aggregates that are susceptible to moisture damage can be treated with anti-stripping agents to improve the moisture resistance of the HMA mixture. If a HMA mix is inherently prone to stripping based on the results of the methodological investigations and moisture susceptibility tests, then anti-stripping agents are warranted (*Kandhal 1992*). Some of the anti-stripping agents include hydrated lime, liquid anti-stripping agents, portland cement, fly ash, and polymers (*Epps et al. 2003*). A survey was conducted by Aschenbrener (*2002*) to identify the current practice of anti-stripping agent usage among highway agencies revealed that, of the 55 agencies (50 state departments of transportation, 3 FHWA Federal Land offices, the District of Columbia, and 1 Canadian province) surveyed, 25 respondents use a liquid anti-stripping agent, 13 use hydrated lime, seven use either a liquid or hydrated lime, and 10 reported that they did not use any treatment for moisture damage problems in HMA pavements.

2.5.3 Asphalt Binder Selection and Testing

The most important character of asphalt that relates to stripping resistance is the viscosity of the asphalt binder in service. Several studies have documented that high viscosity asphalt cement resists displacement by water better than asphalt cements that have low viscosity (*Hicks 1991*). Most states in the US utilize the Superpave performance-grade specification (AASHTO MP1), which stipulates a variety of tests, for binder selection.

2.5.4 Moisture Susceptibility Testing

Laboratory tests are commonly used to determine the effectiveness of different types of anti-strip treatments. Aschenbrener (*2002*) reported, based on a survey, that a majority (44 of the 55 respondents) of the highway agencies conducted test(s) for moisture susceptibility. The most common test was the indirect tensile test (AASHTO T283, ASTM D4867), five agencies used a compressive test (AASHTO T165), two performed retained a stability test, and two used both a wheel tracking and indirect tensile test. The timing of the test was also reported. Thirty agencies conducted the tests only during the mix design process and eighteen performed the tests both during the mix design process and during field acceptance.

3.0 FIELD INVESTIGATIONS

Field investigations included ODOT personnel interviews, initial site visits, records reviews, obtaining pavement cores, excavating trenches, and conducting ground penetrating radar surveys. This section provides details regarding each of these activities.

3.1 ODOT PERSONNEL INTERVIEWS

Oregon Department of Transportation maintenance personnel were interviewed to obtain their thoughts on the possible cause or causes of pavement distress on the projects being investigated. These interviews were conducted with the help of a detailed survey questionnaire (Appendix B) developed prior to the interviews and modified as warranted with successive interviews. The questionnaire gave the researchers a structured way of recording the maintenance personnel responses.

The OSU researchers were successful in conducting interviews of maintenance personnel for all projects except for the Anlauf-Elkhead Road project. Appendix C summarizes the outcomes of the interviews.

3.2 INITIAL SITE VISITS

In conjunction with the personnel interviews, initial site visits were made to visually examine the pavement conditions of the five projects being investigated. Appendix D summarizes the observations made during these visits. Some of the information the investigators collected during the initial site visits included:

- Visual evaluation of the pavements surface.
- Photographs of existing pavement conditions.
- Visual assessment and photographs of the topography and geographic features in the vicinity of the project.

The visual examination of the pavement surface was conducted by windshield survey. A detailed distress survey was not a feasible method for the following reasons:

- Heavy traffic volume on the pavement sections under study and a lane closure would have disrupted the traffic flow considerably.
- The distresses were located in numerous isolated locations. A detailed condition survey at each of these locations would have merely provided similar information to what was observed at the intermittent locations actually observed.

The windshield survey was performed by the investigators from the front passenger car seat, driving at the speed limit, but frequently driving at a slower speed on the shoulders along the distressed locations. Frequent stops were made near locations where severe distresses were witnessed. Drainage features at and/or near these locations were inspected to determine if improper drainage was a contributing factor to the distress. In addition, several photographs of the distressed pavements were taken at these locations.

3.3 RECORDS REVIEW

Review of records is an important process in the investigation of pavement failure. It helps to identify any deficiencies in the design and/or construction of the pavement or any other factors that might have influenced the failure. A detailed records review was conducted with the help of the ODOT Construction Section to obtain the following information:

- Pavement design information.
- Existing pavement structure immediately before rehabilitation.
- Geotechnical and bridge design information related to soil, aggregate, and moisture conditions on the project.
- Topography and geographic features in the vicinity of the project.
- Environmental conditions immediately before, during, and immediately after rehabilitation.
- HMA aggregates source test results.
- HMA mix design information.
- HMA production test results.
- Type of milling equipment used to remove existing HMA, depth of cut per pass, and percent of total depth of existing HMA removed.
- Whether or not traffic was allowed on the milled surface and, if so, for how long?
- Pre- and post-construction pavement performance derived from ODOTs pavement management system database as well as from observations made by the maintenance personnel.
- Maintenance activities performed prior to and following rehabilitation.
- Forensic evaluation information already obtained by ODOT personnel.

Table 3.1 summarizes the findings from the records review, while Appendix E provides additional details.

Table 3.1: Summary of the Findings from Records Review

Project Demographic Records	**Vets Bridge – Myrtle Creek**	**Garden Valley – Roberts Creek**	**Cottage Grove – Martin Creek**	**Anlauf – Elk head Road**	**Pleasant Valley - Durkee**
Year of Rehabilitation	2003	2002	2000	1997	1999
County	Douglas	Douglas	Lane	Douglas	Baker
Highway type/ Section	I-5 MP 109.0 – 112.5	I-5 MP 117.74 – 125.0	I-5 MP 169.3 – 174.7	I-5 MP 154.5 – 162.1	I-84 MP 317.5 – 327.3
Direction	SB and NB	SB and NB	SB and NB	SB and NB	EB and WB
Number of lanes	4	4	4	4	4
Year of Orig. Construction	1955	Variable (1953 earliest)	1959	1954	Late 60s
Number of contracts let on this project	3 (1955, 1964, 1976)	Variable (1976 last Rehab)	3 (1959, 1962, 1980)	3 (1954, 1965, 1975-76)	No data available
Percent cut / percent Fill	No data available	No data available	No data available	No data available	No data available
Creeks/river along project?	Yes (100% of project length)	Yes (100% of project length)	Yes (100% of project length)	Yes (100% of project length)	Yes (100% of project length)
Pavement Rehab Design Records	**Vets Bridge – Myrtle Creek**	**Garden Valley – Roberts Creek**	**Cottage Grove – Martin Creek**	**Anlauf – Elk head Road**	**Pleasant Valley - Durkee**
Existing structure (before Rehab)	40 mm Open graded E-mix	1” E-mix WC; 1.5 – 5.5” C-mix BC	1” E-mix WC; 5” B-mix BC	25 mm E-mix WC, 75 mm B-mix BC	No data available
Subgrade soil type (classification)	No data available	Clayey gravel; Inorganic clay with low to medium plasticity	Silty clay with high plastic and soft material	Dry to damp, stiff, silty-clay subgrade materials	Silty clay with medium plastic slightly moist
Design Subgrade modulus (Average/range)	37.9 MPa (5500 psi)	42.7 MPa	34 MPa (5000 psi)	55.16 MPa	No data available
Subgrade soil moisture content	No data available	5.48 to 22.64; top 1000 mm dry followed by highly moist soil	No data available	No data available	No data available
Deflection testing performed?	Yes	Yes	Yes	Yes	
Pavement Design Recommended	75 mm grind and overlay, with 75 mm of Level 4, 19 mm Open mix	WC: 50 mm of 19 mm Open graded HMAC BC: Inlay 50 mm of 19 mm dense HMAC	2” inlay F-mix AC wearing course; two areas of base failure, full dig out and replacement with B-mix	25 mm and 75 mm mill from inside and outside lane resp; 50 mm 19 mm lime treated leveling dense mix BC, followed by 50 mm of 19mm lime treated Open mix WC	50 mm of HMA milled and inlayed with 19 mm dense HMA mix, then overlaid with 40 mm of 12.5 mm SMA

Pre-Design Pavement Condition Survey Records	Vets Bridge – Myrtle Creek	Garden Valley – Roberts Creek	Cottage Grove – Martin Creek	Anlauf – Elk head Road	Pleasant Valley - Durkee
Was a detailed condition survey performed?	Yes	Yes	Yes	Yes	Yes
Was a visual examination of pavement surface conducted? If so, in what year?	Yes (2000)	Yes (1998)	Yes (1995)	Yes (1995)	Yes (1998)
What was the predominant distress noted in the project?	Rutting (both directions)	Rutting (both directions)	Rutting (both directions)	Rutting (both directions)	Rutting (both directions)
What are the other types of surface distress noted?	Fatigue cracking, few longitudinal cracks, and few ruts rich with asphalt	SB - Fatigue cracks and patches; NB – Patches, raveling, and longitudinal cracks	SB – blade and pothole patches; NB – fatigue cracking with low to high severity	Patches (inlay and blade), raveling, transverse and longitudinal cracking near ramps	Rutting and patches
Was coring performed?	Yes	Yes	Yes	Yes	Yes
Was stripping evident in core? If yes, at what depth/layer?	No	No	SB – Yes (MP 170.39, 170.43, 171.39); NB - None	Yes (few cores, initial stages)	Yes (few cores @ EB MPs 314.85 and 315.38 and at ramps WB MP 313)
Was acker/exploration hole drilled?	No	Yes	Yes	Yes	Yes
Were rutting measurements taken? If so, what was the average/range of rutting?	Yes; Average: 0.5”, ranging from 0” to 1”	Yes; Average: 3/8”, ranging from 0” to 1 1/8”	Yes; Average: 0.75”, ranging from 0” to 1 5/8”	Yes; ranging from 0” to 1.4”	Yes; raging from 0.1” to 0.8” EB and 0” to 0.8” WB
What was the condition of shoulders? (Poor, Fair, Good)	Fair	Fair	Fair	No data available	No data available
Was a detailed drainage survey conducted? If so what was the condition of the drains? (Poor, Fair, Good)	Unknown; Unknown	Unknown; Unknown	Unknown; Unknown	No data available	No data available

Traffic Records	**Vets Bridge – Myrtle Creek**	**Garden Valley – Roberts Creek**	**Cottage Grove – Martin Creek**	**Anlauf – Elk head Road**	**Pleasant Valley - Durkee**
Design Equivalent Single Axle Loads	15 year; Truck lane: 29,200,000 Inside lane: 3,240,000	10 year – 18 million	10 year - 6,100,000	10 year; SB-20,600,000 NB-23,800,000	10 year – 20.6 million
Average Daily Traffic	NB-11700, SB-10800	NB-14600, SB-13400	NB-11000, SB-10000	NB-9100, SB-8400	6700
Level of truck traffic (%)	NB-22.7%, SB-25.4%	NB-21%, SB-23%	NB-21.8%, SB-24.3%	NB-22.4, SB-21.8%	42%
Year of traffic counts	1997	1994	1993	1994	1996
Environmental Condition Records	**Vets Bridge – Myrtle Creek**	**Garden Valley – Roberts Creek**	**Cottage Grove – Martin Creek**	**Anlauf – Elk head Road**	**Pleasant Valley - Durkee**
Inches of rainfall in previous 7 days (before rehab)	No data available	No data available	Dry (Maintenance personnel)	No data available	No data available
Mean Daily Temperature in previous 7 days (before rehab)	No data available	No data available	No data available	No data available	No data available
Minimum and maximum temperature in previous 7 days (before rehab?)	No data available	No data available	No data available	No data available	No data available
Was there rainfall during construction?	No	No	Yes	No data available	No data available
What was the average rainfall during the period?	No data available	No data available	No data available	No data available	No data available
What was the average temperature during paving?	No data available	No data available	No data available	No data available	No data available
Minimum and maximum temperature in 30 days following rehab?	No data available	No data available	No data available	No data available	No data available
Inches of rainfall in 30 days following rehab	No data available	No data available	No data available	No data available	No data available
Mean Daily Temperature in 30 days following rehab	No data available	No data available	No data available	No data available	No data available

HMA Mix Design Records	Vets Bridge – Myrtle Creek	Garden Valley – Roberts Creek	Cottage Grove – Martin Creek	Anlauf – Elk head Road	Pleasant Valley - Durkee
Mix Design Existing AC surface (before rehab)					
• Mix type	E-mix	E-mix	E-mix WC over B-mix BC	E-mix WC over B-mix BC	No data available
• Target AC	No data available	No data available	No data available	No data available	No data available
• Actual AC	No data available	No data available	No data available	No data available	No data available
• Target P200	No data available	No data available	No data available	No data available	No data available
• Actual P200	No data available	No data available	No data available	No data available	No data available
• Moisture content	No data available	No data available	No data available	No data available	No data available
Mix Design New AC base course					
• Mix type	Not Applicable for mainline, only inlay was involved	19 mm Dense HMA mix	Standard duty HMA B-mix	Heavy duty HMA B-mix	19 mm dense HMA mix
• Target AC		5.2	5.7	5.5	
• Actual AC		Avg-5.17	Avg-6.02, Stddev-0.154	Avg-5.55, Stddev-0.1531	
• Target P200		4.8	5.0	5.0	
• Actual P200		Avg-5.57, Stddev-0.5649	Avg-3.93, Stddev-0.5465	Avg-5.93, Stddev-0.475	
• Moisture content		0.21	0.47	0.30	
Mix Design New AC wearing course					
• Mix type	Level 4, Open mix	19 mm Open mix	Heavy duty F-mix	Heavy duty F-mix	
• Target AC	5.7 (was initially 5.8, then raised to 6, then reduced to 5.7)	5.8	5.4	5.6	
• Actual AC	No data available	Avg-5.365	Avg-6.02, Stddev-0.21	Avg-5.67, Stddev-0.0516	
• Target P200	2.3	2.5	3.9	3.9	
• Actual P200	No data available	Avg-3.02, Stddev-0.4134	Avg-4.03, Stddev-0.5366	Avg-4.109, Stddev-0.5381	
• Moisture content	No data available	0.44	0.81	0.51	

Material Records	Vets Bridge – Myrtle Creek	Garden Valley – Roberts Creek	Cottage Grove – Martin Creek	Anlauf – Elk head Road	Pleasant Valley - Durkee
Materials - Existing AC surface (before rehab)	No data available	No data available	No data available	No information	
• Aggregate type	No data available	No data available	No data available	No information	
• Was lime used? If yes, what amount?	No data available	No data available	No data available	No information	
• Asphalt type	No data available	No data available	No data available	No information	
• Asphalt grade	No data available	No data available	No data available	No information	
• Any additives used?	No data available	No data available	No data available	No information	
Materials - new AC base course					
• Aggregate type	Not applicable for mainline, only inlay was involved	S. Umpqua gravels	Gravel	Gravel	
• Was lime used? If yes, what amount?		No	No	Yes, 1%	
• Asphalt type		Chevron	McCall	McCall	
• Asphalt grade		PBA – 5	PBA-5	PBA-5	
• Any additives used?		No	No	No	
Materials - new AC wearing course					
• Aggregate type	S. Umpqua gravels	S. Umpqua gravels	Gravel	Gravel	
• Was lime used? If yes, what amount?	No	No	Yes; 1%	Yes, 1%	
• Asphalt type	McCall	Chevron	McCall	McCall	
• Asphalt grade	PG 70 - 28	PBA - 6	PBA-6	PBA-6	
• Any additives used?	No	No	No	No	

Quality Control/Quality Assurance Tests	Vets Bridge – Myrtle Creek	Garden Valley – Roberts Creek	Cottage Grove – Martin Creek	Anlauf – Elk head Road	Pleasant Valley - Durkee
New base					
• Gradation within specs?	Yes	Yes	Yes	Yes	
• Binder content within specs?	Yes	Yes	Yes (6 sublot over spec limit)	Yes	
• Moisture content within specs?	Yes	Yes	Yes	Yes	
New wearing course					
• Gradation within specs?	Yes	Yes	Yes	Yes	
• Binder content within specs?	Yes	Yes	Yes	Yes	
• Moisture content within specs?	Yes	Yes	Yes	Yes	

Pavement Surface Preparation Records (Milling)	**Vets Bridge – Myrtle Creek**	**Garden Valley – Roberts Creek**	**Cottage Grove – Martin Creek**	**Anlauf – Elk head Road**	**Pleasant Valley - Durkee**
What the depth of cut per pass?	No data available	No data available	No data available	No data available	
What percentage of total depth of existing surface was milled?	No data available	No data available	No data available	No data available	
What was the milling equipment used?	7.5 foot roto-mill	7.5 foot roto-mill	Self propelled machine	No data available	
Was the surface cleaned after milling? (Y/N). If yes, what was used to clean the surface?	Yes; pick up broom	Yes; pick up broom	Yes; power broom, backhoe	No data available	
Was milled surface exposed to traffic?	No (not sure)	Yes	No	No data available	
Was the milled surface exposed to rain?	Yes	Yes	No	No data available	
What was the weather during milling ?	Dry	Dry	Dry	No data available	

Pavement Surface Preparation Records (Milling)	**Vets Bridge – Myrtle Creek**	**Garden Valley – Roberts Creek**	**Cottage Grove – Martin Creek**	**Anlauf – Elk head Road**	**Pleasant Valley - Durkee**
Was any kind of distress noticed on the milled surface particularly stripping?	No	No	No	No data available	
Post Construction Records (pavement management system)	**Vets Bridge – Myrtle Creek**	**Garden Valley – Roberts Creek**	**Cottage Grove – Martin Creek**	**Anlauf – Elk head Road**	**Pleasant Valley - Durkee**
• Distress type					
Maintenance Records					
Post construction conditions – maintenance personnel comments on prime cause of distress	Rutting primary distress. Maintenance personnel have been repairing the distress with patches.	Localized areas of rutting and shoving in the HMA layers. At that time they found only one area of failure in NB compared to several in the SB of the project maintenance personnel have been repairing the distress with patches.	Rutting was found in many places. Hand patching, inlay repair and concrete patching. Also installed 3 perforated pipes. Maintenance personnel say "underground springs" were the major cause of distress	No information	Potholes were the major problem according to maintenance personnel; Water comes out of construction joints in many places. Rutting is found in lots of places. Close to 2500 tons of grind inlay every year close to 250K. Patches done in June failed in August last year.
Trenching operations records	**Vets Bridge – Myrtle Creek**	**Garden Valley – Roberts Creek**	**Cottage Grove – Martin Creek**	**Anlauf – Elk head Road**	**Pleasant Valley - Durkee**
Were trenching operations carried out? (Y/N)	Yes	Yes (Refer to Section 3.0) August of 2003; Two trenches were cut around MP 123	No information	No information	Yes (May 16, 2006)

Trenching operations records	**Vets Bridge – Myrtle Creek**	**Garden Valley – Roberts Creek**	**Cottage Grove – Martin Creek**	**Anlauf – Elk head Road**	**Pleasant Valley - Durkee**
If yes, what were the findings?	The new 19 mm open-graded HMA and material underlying the new layer appeared to be stripping. No documentation was provided	Trench 1: ODOT personnel reported that an area of unstable HMAC at about 3.5 to 4 inches below the surface was observed. It was concluded that the mix was stripping. Trench 2: The second trench was cut in an area that was not experiencing any failure. The material was in much better condition, but did show that this area is also starting to strip.	No information	No information	Two trenches. Both trenches had materials exhibiting stripping in the surface layers. Sample collected with the investigators.

Post Construction Records (initial site visits)	**Vets Bridge – Myrtle Creek**	**Garden Valley – Roberts Creek**	**Cottage Grove – Martin Creek**	**Anlauf – Elk head Road**	**Pleasant Valley - Durkee**
Distress types	SB: Patches, fat spots and bleeding NB: Rutting, bleeding, and patches	SB: Rutting and patches NB: Fat spots and patches	SB: Patches, fat spots and bleeding NB: Patches	SB: Patches, shoving and pothole NB: Pothole, bleeding, and patches	EB: Patches, potholes, water coming out of pavements WB: Lengthy patches, silt coming out of pavements

3.4 FIELD SAMPLING AND TESTING

Field sampling and testing consisted of cutting cores from the pavements, excavating trenches along three of the projects, and conducting ground penetrating radar (GPR) surveys along four of the projects. GPR surveys were conducted in the outside travel lanes in the southbound and northbound directions. Data were collected along both wheel tracks, between the wheel tracks, and along the fog line and analyzed to determine differential thickness of the pavement layers (i.e., difference in thickness of pavement layers within the wheel tracks relative to the thickness of the layers between the wheel tracks) as well as to determine the Activity Index (a measure of radar reflection amplitude relative to the average reflection amplitude over a given length of pavement and normalized to unity). The final report provided by the subcontractor (Appendix K) provides details of the GPR surveys and data analyses. This section summarizes the observations made during field sampling of the projects and a summary of results obtained from the GPR surveys.

3.4.1 Pleasant Valley – Durkee, Interstate 84, MP 317.5 – 327.3

The field investigations for this project involved coring and trenching operations (GPR testing was not conducted on this project). Coring was accomplished by ODOT personnel on four separate occasions as indicated in Table 3.2. Appendix F includes the pavement core logs for these cores.

Table 3.2: Core Locations and Dates Obtained for the Pleasant Valley – Durkee Project.

Direction	Dates	Coverage	Number of Cores Obtained	Typical Distance between Cores (miles)	Notes
Westbound	10/18/05 – 10/20/05	MP 320.25 – MP 326.25	24	0.5	Several cores obtained at intermediate locations near structures.
	3/23/06	MP 319.25 – MP 319.75	2	0.5	
	3/29/06	MP 324.00 – MP 325.00	2	1.0	
Eastbound	3/28/06 – 3/30/06	MP 323.50 – MP 327.25	18	0.5	Several cores obtained at intermediate locations near structures. One core obtained at MP 321.

Examination of the core logs revealed that, with few exceptions, most pavement layers indentified in the logs were in “Good” condition. In the eastbound direction, one of the approximately 110 individual layers (i.e., layers delineated in the core logs) was identified as being in “Fair” condition and four were identified as being in “Poor” condition. In the westbound direction, nine of the approximately 140 individual layers were identified as being in “Fair” condition and two were identified as being in “Poor” condition. Other observations include the following:

- Four logs for the cores obtained in the eastbound direction (between mile points 325.287 and 325.401 and at MP 326.5) indicated “heavy oil” in the top two inches.

- Several logs for the cores obtained in the westbound direction indicated “with voids” or “some voids” in the lower portion of the original HMA, principally in the 8 to 16 inch range. However, one log for the inside travel lane at MP 324.5 indicated “some voids” in the 2 inch HMA inlay.

In addition to the coring operation, two trenches were excavated by ODOT personnel along the project in the eastbound direction, one at MP 323.5 and one at MP 324.9, both on a moderate downhill gradient. This involved making parallel, transverse saw cuts approximately two feet apart across the shoulder and outside travel lane to the skip stripe (Figure 3.1), a longitudinal saw cut near the skip stripe to join the transverse cuts, sweeping the saw cuts (Figure 3.2), removal of the bound layers using a backhoe fitted with a fork attachment (Figure 3.3), and removal of base and subgrade materials. The cut faces of the pavement layers were then wiped reasonably clean so that observations could be made of the pavement layers.

MP 323.5 Trench

The following observations were made during and following excavation of the trench at MP 323.5:

- During the trenching operation it was observed that the stone-matrix asphalt (SMA) layer delaminated from the underlying dense-graded layer during the removal of the shoulder slab (Figure 3.4), but not during excavation of the pavement slab across the travel lane (Figure 3.5).

- The bound layers of the pavement structure consisted of 1½ inches of SMA over 4½ inches of dense-graded HMA, over 7 inches of dense-graded HMA, over 2 inches of plant mix bituminous base (PMBB) as shown in Figure 3.6. Note that this picture was taken shortly after the cut face of the upper pavement layers had been wiped clean. Note also that the PMBB appeared to be saturated.

- The unbound layers consisted of 14 inches of 1”-0” aggregate base, over 18 inches of moist sand, over compacted, sandy decomposed granite as shown in Figure 3.7. It was noted that the aggregate base consisted of mostly round rock with very few fractured faces and that it was highly contaminated with fines.

- Some stripping had occurred in all bound layers. The SMA and top 4½ inches of dense-graded HMA exhibited early stages of stripping whereas the lower 7 inches of dense-graded HMA indicated more advanced stripping as shown in Figure 3.8.

- The photos in Figures 3.9 and 3.10 were taken approximately two hours after the trench had been opened. These clearly indicate water seepage at the SMA/HMA layer interface, within the dense-graded HMA, and in the PMBB. Note that the depths of water seepage within the HMA layers correspond with the layer interfaces identified in the pavement core log (Figure 3.10) at depths of five and eight inches.

Figure 3.1: Cutting the Bound Layers for the Trenching Operation.

Figure 3.2: Sweeping the Surface over the Saw Cuts.

Figure 3.3: Using a Backhoe with Fork Attachment to Remove Bound Material.

Figure 3.4: SMA Wearing Course Delaminating from Underlying HMA Course.

Figure 3.5: Excavation of the Travel Lane Slab with SMA Wearing Course Intact.

Figure 3.6: Profile of Bound Layers at MP 323.5.

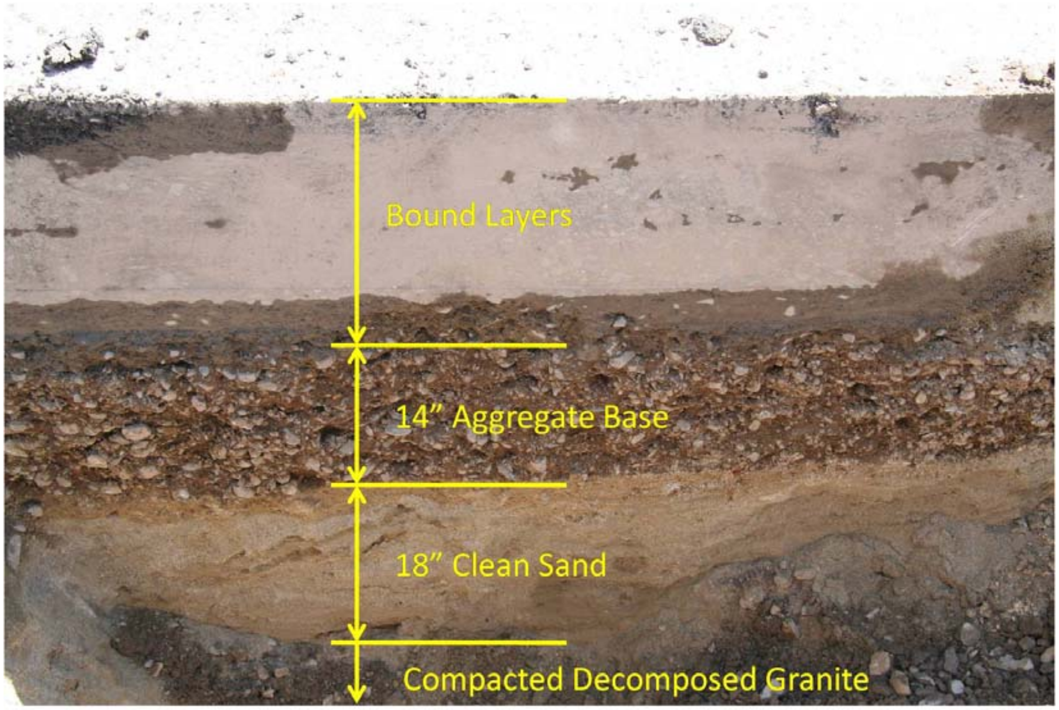


Figure 3.7: Profile of Unbound Layers at MP 323.5.

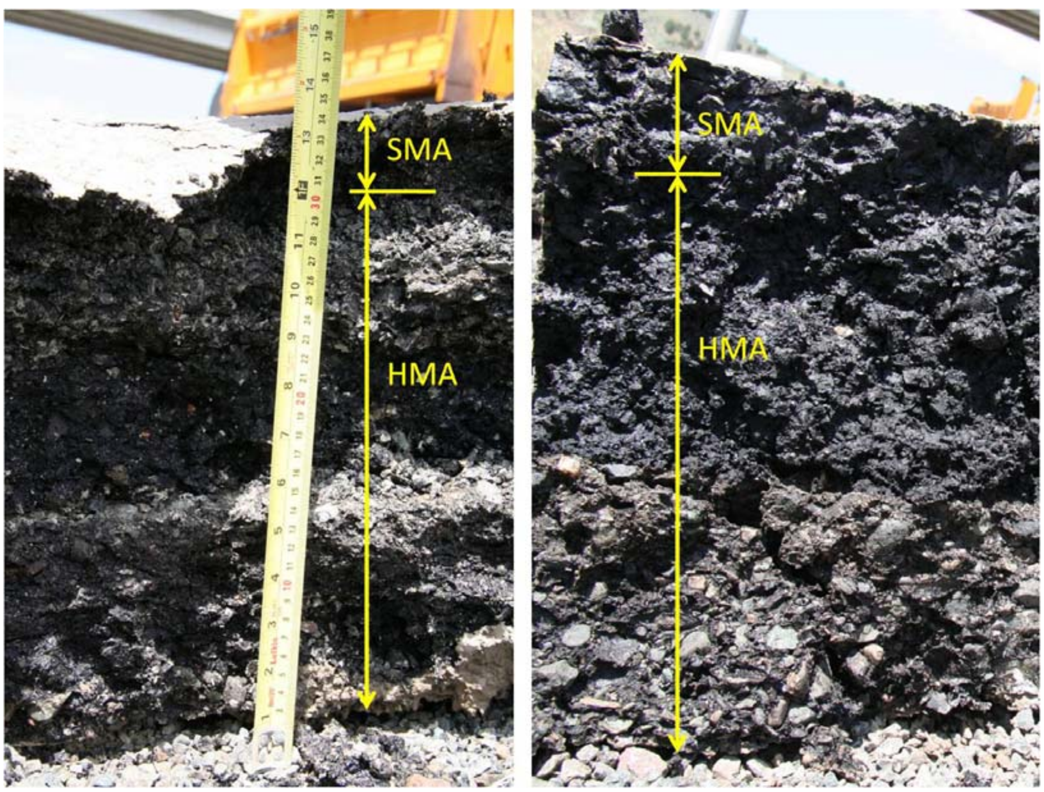


Figure 3.8: View of Stripping in the Bound Layers at MP 323.5.

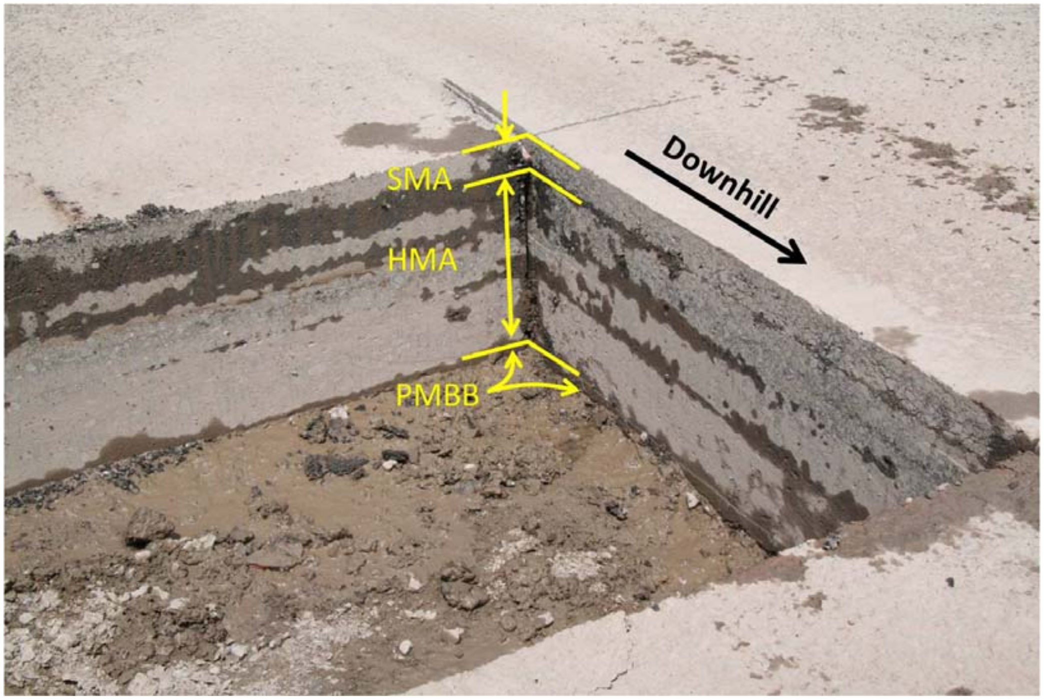


Figure 3.9: Uphill and End Faces of Trench at MP 323.5 Approximately Two Hours after Excavation.

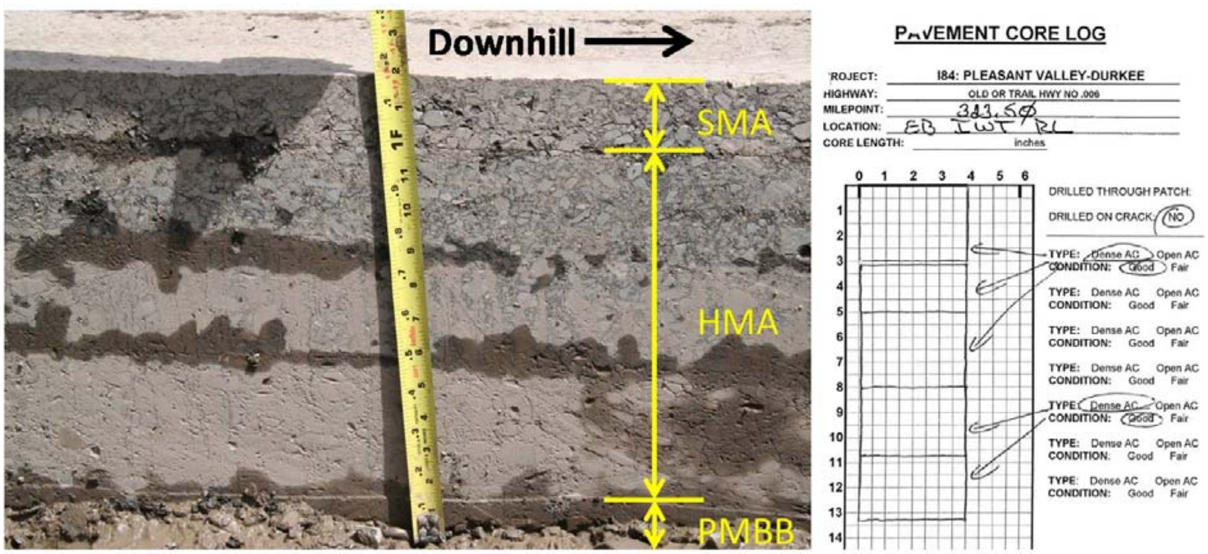


Figure 3.10: End Face of Trench at MP 323.5 Approximately Two Hours after Excavation.

MP 324.9 Trench

The following observations were made during and following excavation of the trench at MP 324.9:

- As with the first trench (at MP 323.5), the SMA layer delaminated from the underlying dense-graded HMA layer during removal of the slab at the trench at MP

324.9 as shown in Figures 3.11 and 3.12. Figure 3.11 also shows that the lifts in the lower portion of the slab also delaminated during excavation. Figure 3.13 shows the cut face at the end of the slab indicating that the SMA layer was delaminated from the underlying HMA layer near the skip stripe (i.e., near the construction joint between travel lanes).

- Figure 3.14, taken shortly after the trench was opened, shows that the depths of the bound layers were essentially the same as those at MP 323.5. Note that the PMBB layer appeared to be saturated and that some water seepage was evident between the upper and lower dense-graded HMA layers.

- Figure 3.15 shows that the depths of the unbound layers were essentially the same as those at MP 323.5.

- Figure 3.16 shows two photos of a portion of the slab removed from the trench indicating clear evidence of significant stripping in the upper and lower portions of the dense-graded HMA layers. Further, these photos indicate that the SMA layer contained a significant amount of asphalt binder, possibly indicating migration of the binder from the HMA layer into the SMA layer.

- The upper half of Figure 3.17 shows a photo of another portion of the slab removed from the trench indicating stripping had occurred in the SMA layer and throughout the HMA layers. The lower half of this figure shows a close-up photo of the upper HMA layer indicating pockets of asphalt binder and bare (stripped) aggregate particles.

- The photo in Figure 3.18, taken over two hours after the trench had been opened, clearly indicates water seepage between the SMA layer and the underlying HMA layers. It can also be seen from this photo that the PMBB layer appeared to be saturated.

- The photo in Figure 3.19, also taken over two hours after the trench had been opened, indicates water seepage at the delaminated layers as well as through the upper portion of the HMA layers. Note that the depths of water seepage within the HMA layers correspond with the layer interfaces identified in the pavement core log at depths of four and six inches. Note also the wet patch near the top of the downhill face of the trench, which indicates water seeping out of the SMA/HMA interface.

Figure 3.11: Pavement Layers Delaminating During Removal.

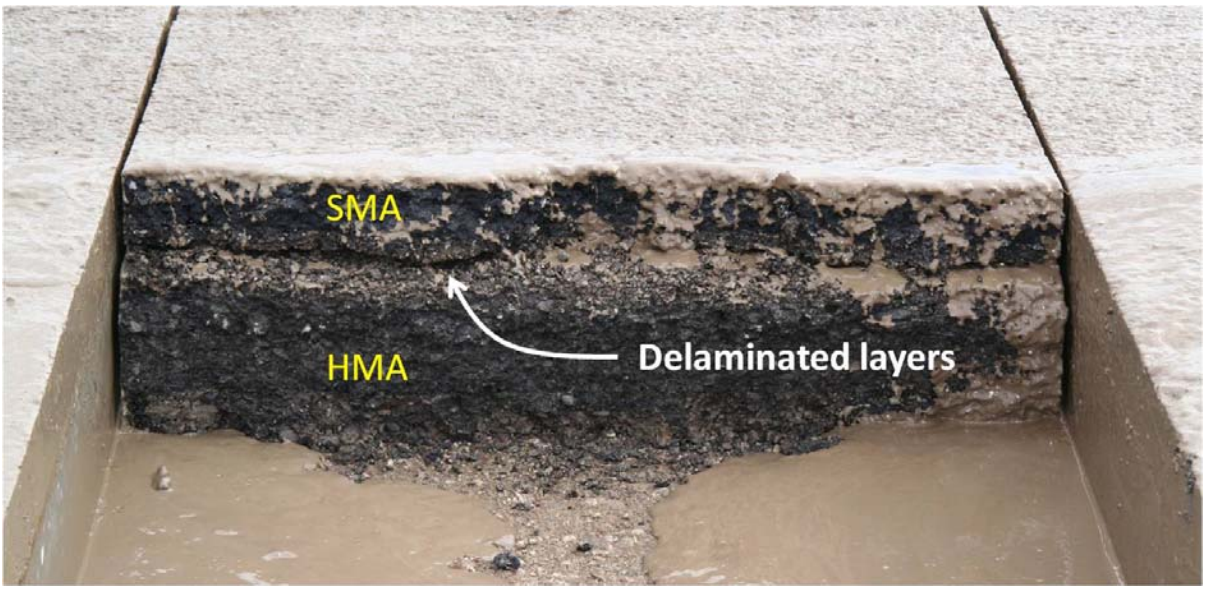


Figure 3.12: SMA Wearing Course Delaminated from HMA Layers.

Figure 3.13: Wearing Course Delaminated from the HMA Course at the End of the Slab.

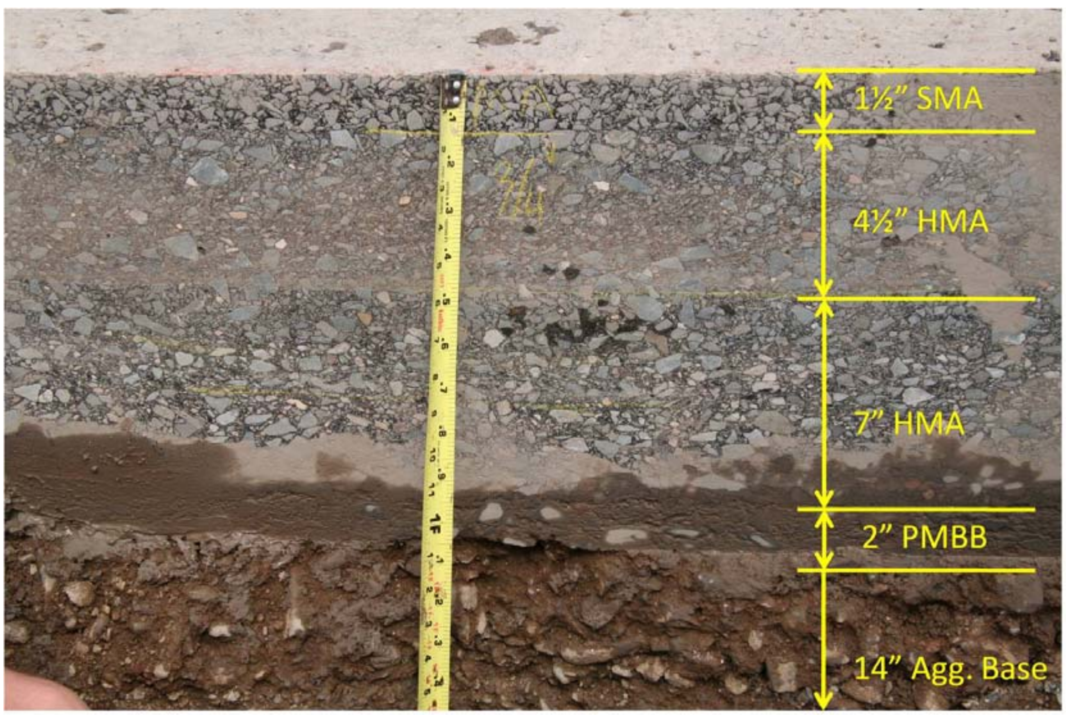


Figure 3.14: Profile of Bound Layers at MP 324.9.

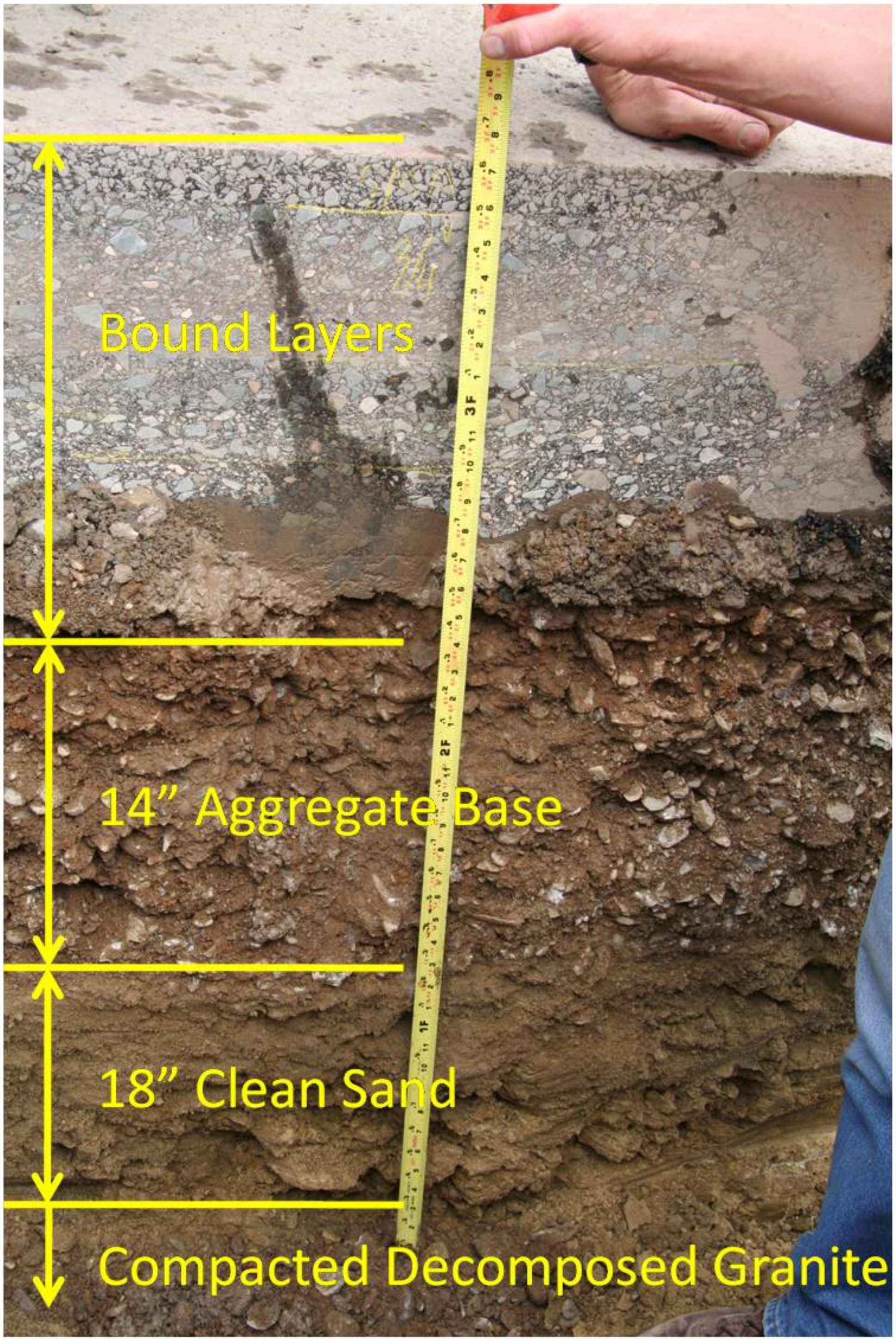


Figure 3.15: Profile of Unbound Layers at MP 324.9.

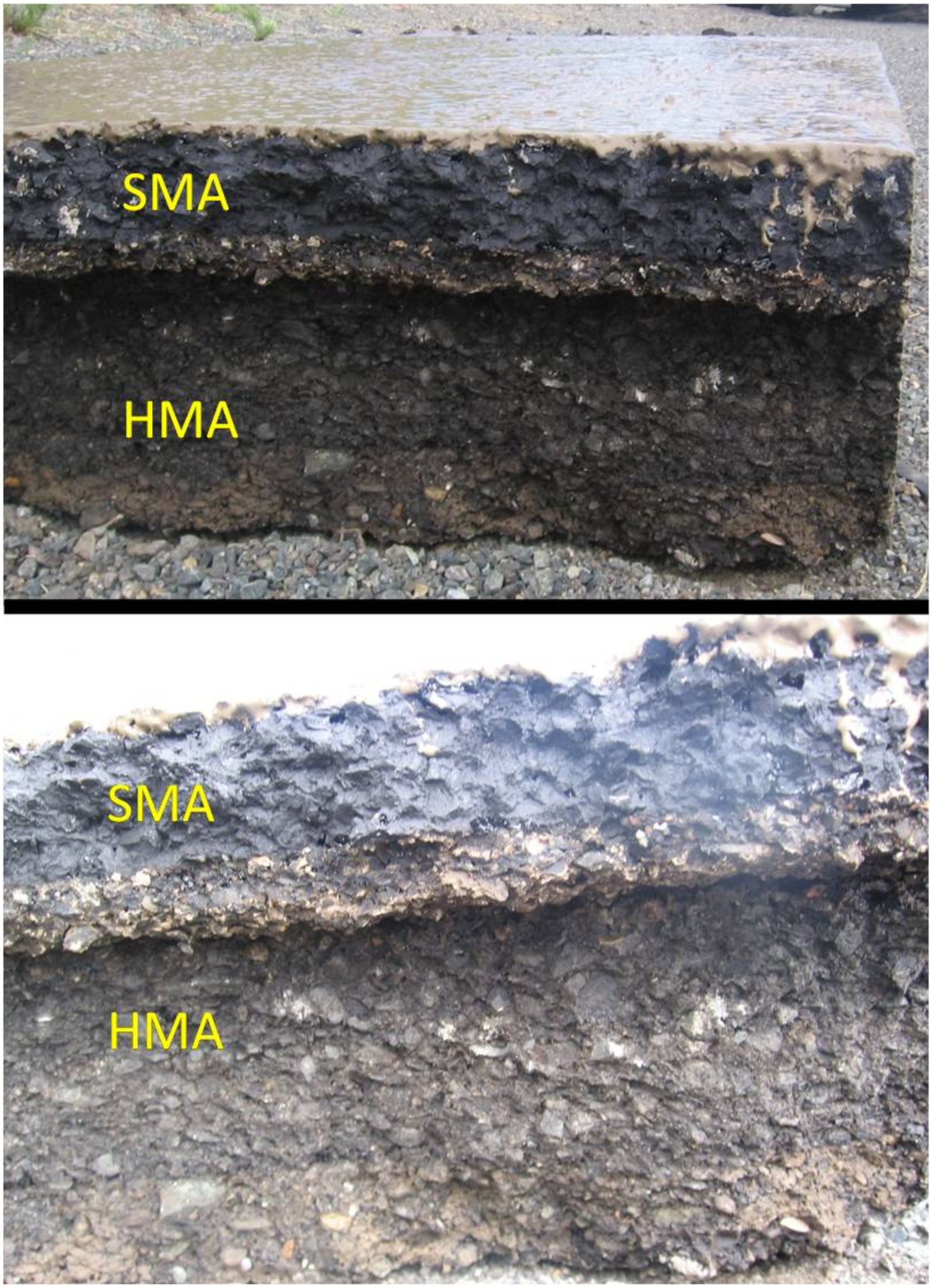


Figure 3.16: Photos of Portions of the Slab Removed at MP 324.9.

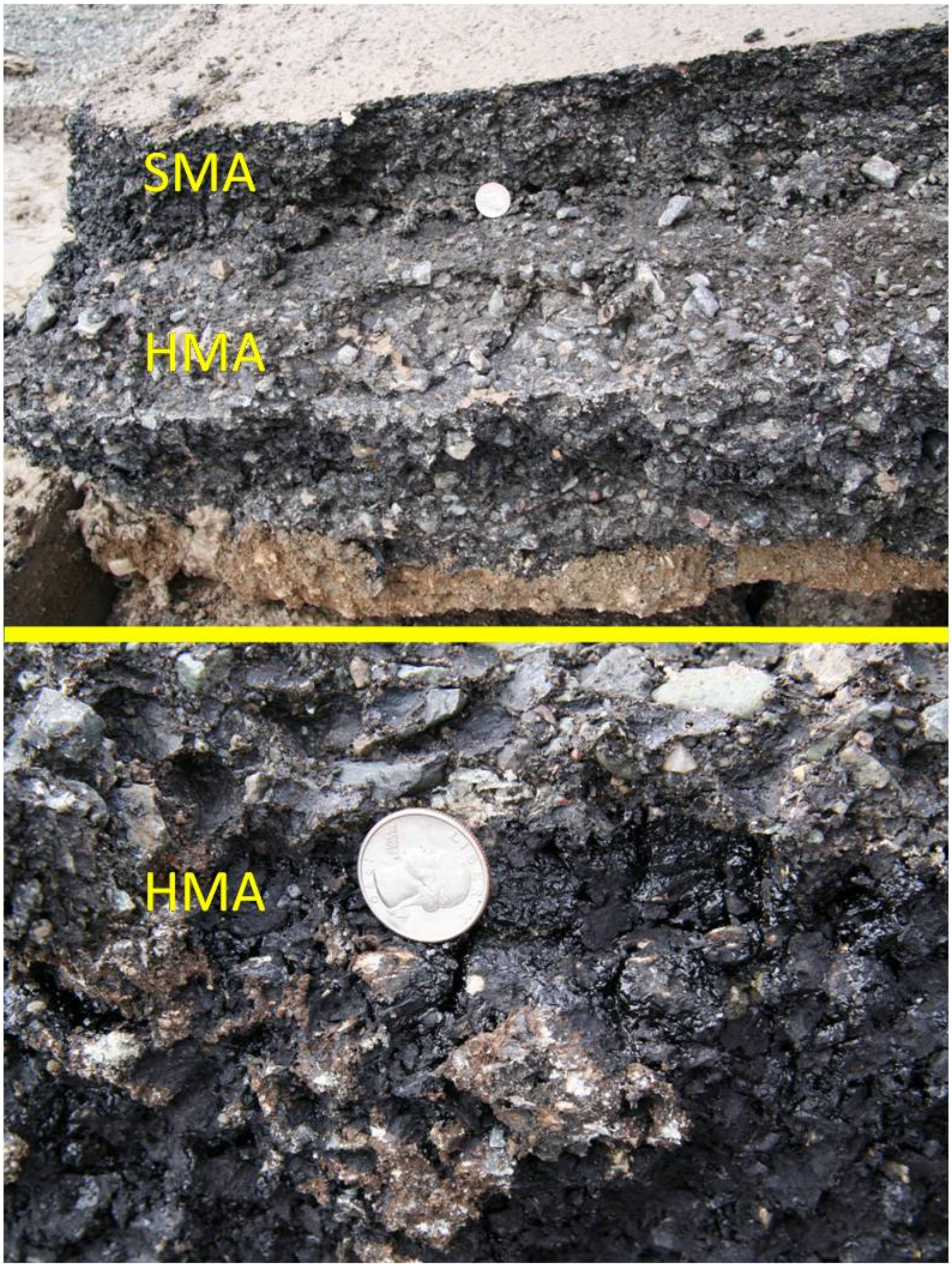


Figure 3.17: Photos of Another Portion of the Slab Removed at MP 324.9.

Figure 3.18: Uphill Face of Trench at MP 324.9 Approximately Two Hours After Excavation.

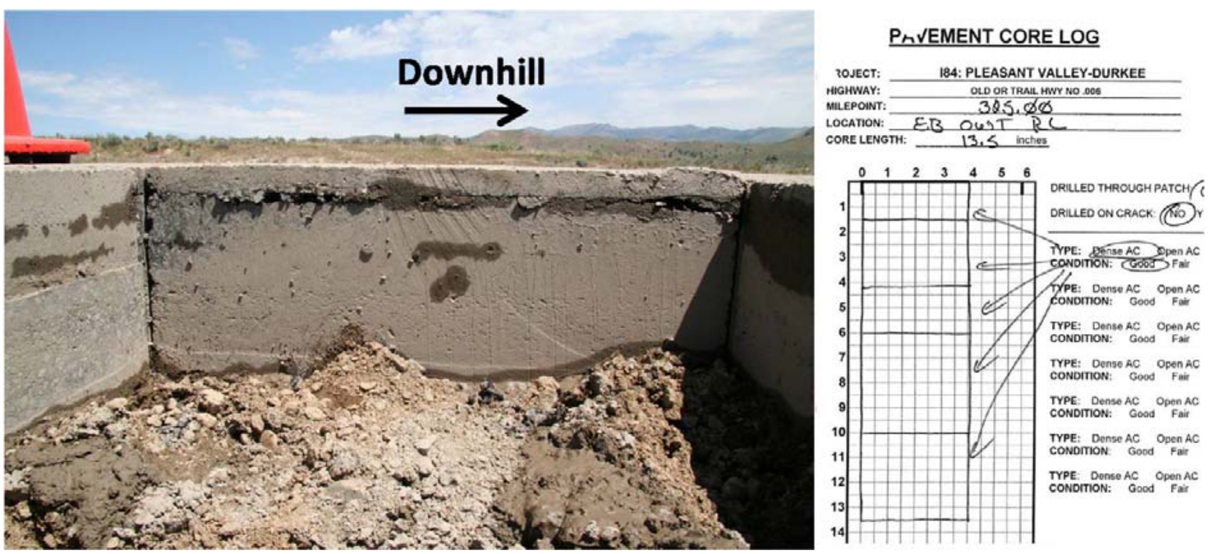


Figure 3.19: End Face of Trench at MP 324.9 Approximately Two Hours After Excavation.

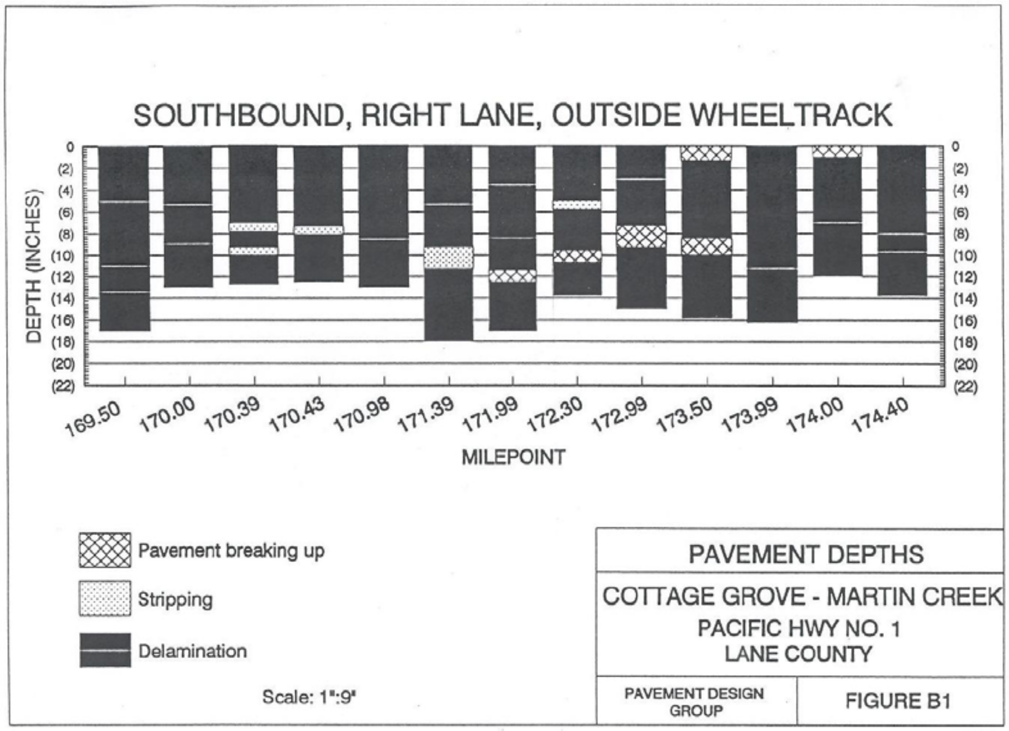
3.4.2 Cottage Grove – Martin Creek, Interstate 5 MP 169.3 – 174.7

The field investigations for this project involved GPR surveys. Inability to close a lane prevented coring and trenching activities for this study. However, cores obtained in 1995 (prior to the rehabilitation activity in 2000) are summarized below.

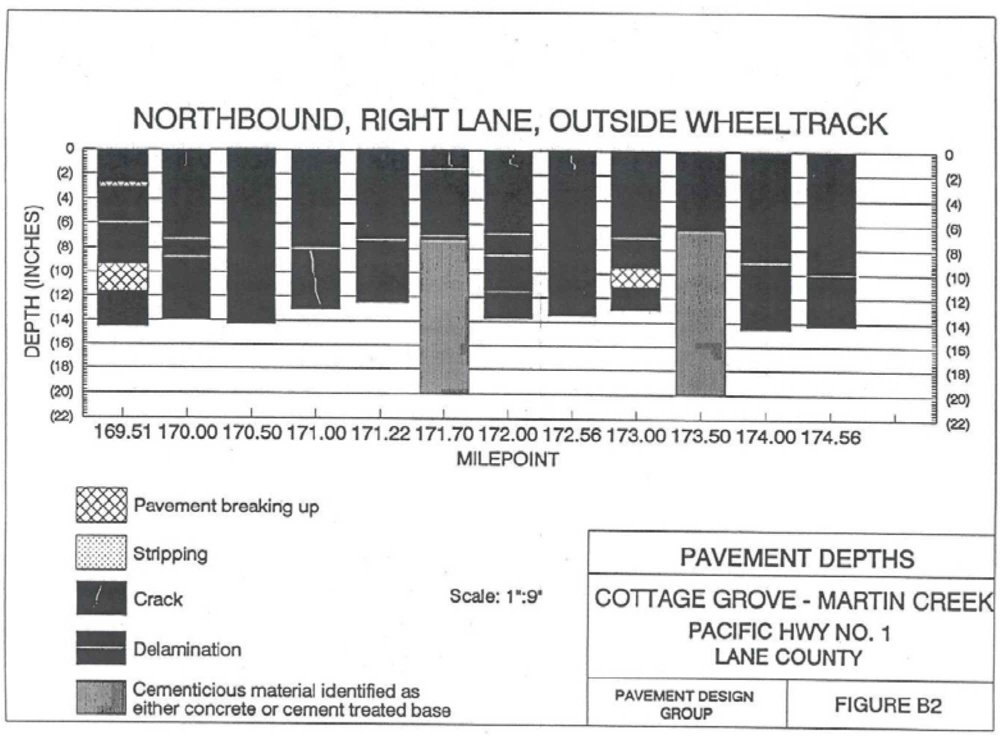
Cores Obtained in 1995

Figure 3.20 provides a summary of the condition of cores obtained from the outside wheel path of the outside lane in 1995 prior to the rehabilitation activity for this project in 2000. Ignoring the top 2 inches (50 mm), which was removed during the rehabilitation activity, it can be seen from Figure 3.20a (for the cores obtained in the southbound direction) that most of the cores had layers that delaminated during extraction. Three of the four cores obtained between mile points 170.39 and 171.39 showed signs of stripping at depths between about 7 and 11½ inches (180 to 290 mm), and all cores obtained from the stretch between mile points 171.99 and 173.50 had layers with disintegrating material.

The summary for the cores obtained from the northbound direction (Figure 3.20b) indicates that most of the cores had layers that delaminated during extraction, and two cores (from mile points 169.51 and 173.00) had layers with disintegrating material at depths between about 9½ and 11½ inches (240 and 290 mm). Additionally, the log for the core obtained from MP 171.00 indicated that the bottom layer of the pavement core from about 8 to 13 inches (200 to 330 mm) had a crack.



a) Southbound Direction



b) Northbound Direction

Figure 3.20: Summary of Core Logs for the Cottage Grove – Martin Creek Project.

Ground Penetrating Radar Survey

Data were collected from mile points 169.30 to 174.70 in both directions. These locations were selected based on the presence of surface distresses in the form of stripping, patching, and rutting along these stretches of pavement.

Figures 3.21 and 3.22, taken from the subcontractors report (Appendix K), show locations of potential moisture damage and layer thickness. This was based on analyses of the GPR data for the southbound and northbound directions, respectively.

Table 3.3, reproduced from Table 3 of the subcontractors report (Appendix K), provides a summary of results for the Cottage Grove – Martin Creek project derived from the GPR data analyses. It shows the percentage of the section length where moisture damage has likely occurred based on the magnitude of the Activity Index and "Differential Compaction" (i.e., differential layer thickness). For example, the results suggest that 19.0% of the length of pavement between mile points 169.30 and 174.70 (i.e., approximately 1.0 miles of the outside lane) in the northbound (NB) direction had moisture-induced damage. The results for the "Differential Compaction" indicator are broken down by layer, where "Layer 1" corresponds to the top layer detected, "Layer 2" corresponds to the layer detected beneath "Layer 1", and "Layer 3" corresponds to the layer detected beneath "Layer 2". The thickness of these layers varies along the length of each section as indicated in Figures 3.21 and 3.22.

Table 3.3: Summary of Results for GPR Data Moisture Damage Analysis for the Cottage Grove – Martin Creek Project.

Direction	Start MP	End MP	Predicted Moisture Damage (Percent of Section Length)			
			Activity Index	Differential Compaction		
				Layer 1	Layer 2	Layer 3
NB	169.30	174.70	19.0	6.6	3.4	4.3
SB	174.40	169.30	23.1	3.8	9.5	15.6

3.4.3 Anlauf – Elkhead Road, Interstate 5 MP 154.5 – 162.1

The field investigations for this project involved coring and GPR testing. Although it was planned to trench this project, torrential rains made it a futile endeavor. Coring was accomplished by ODOT personnel, both during the project investigation activities in 1995, and for this research study in 2006. Cores for the project investigation activities were obtained in 1995 between mile points 155.04 and 161.94 in the southbound direction and between mile points 155.64 and 162.23 in the northbound direction. Cores for this research study were obtained in 2006 at mile points 156.90 and 157.20 in the southbound direction. Appendix H includes the pavement core logs for these cores.

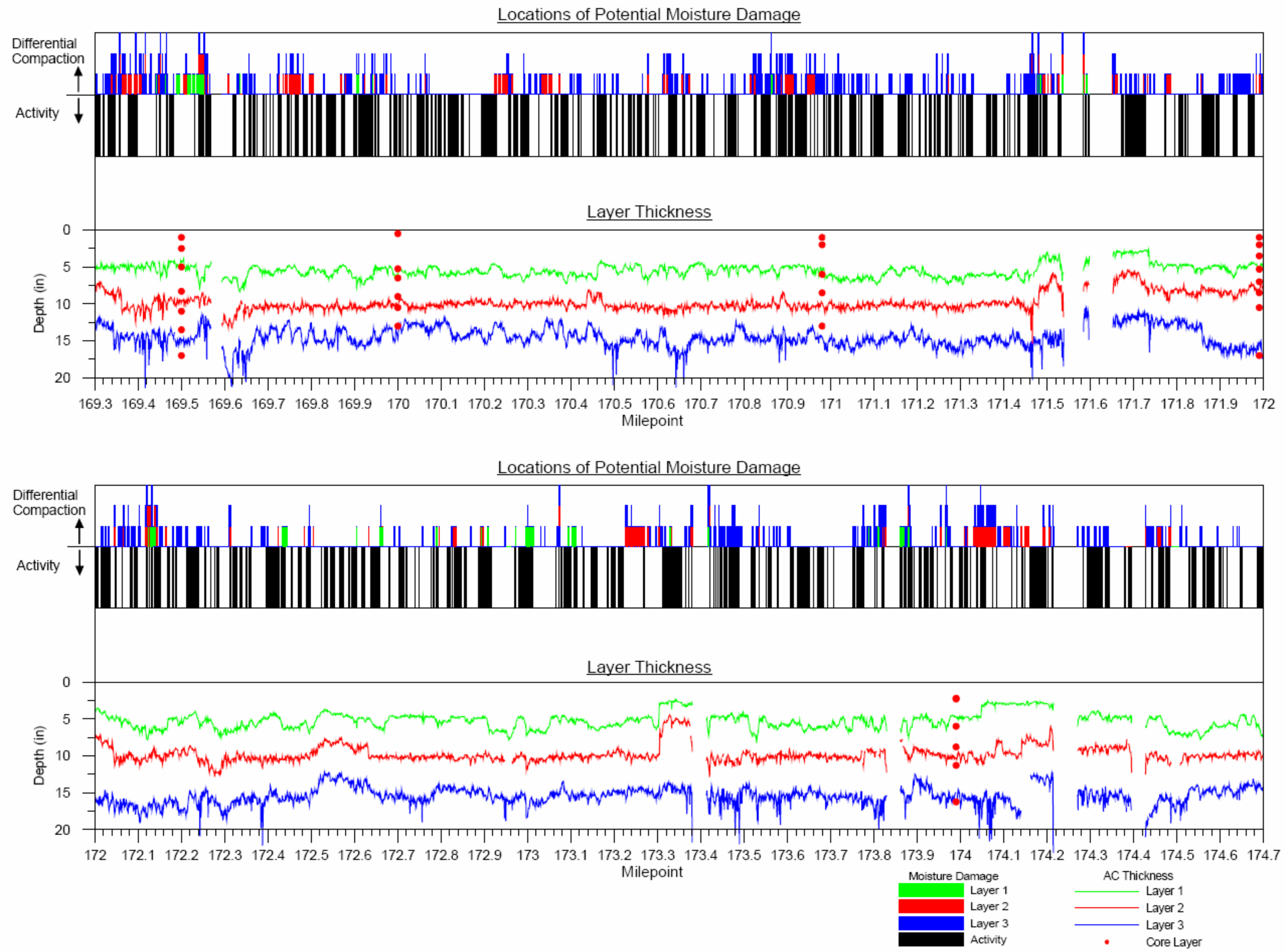


Figure 3.21: Locations of Potential Moisture Damage for the Cottage Grove – Martin Creek Project, Southbound Lanes.

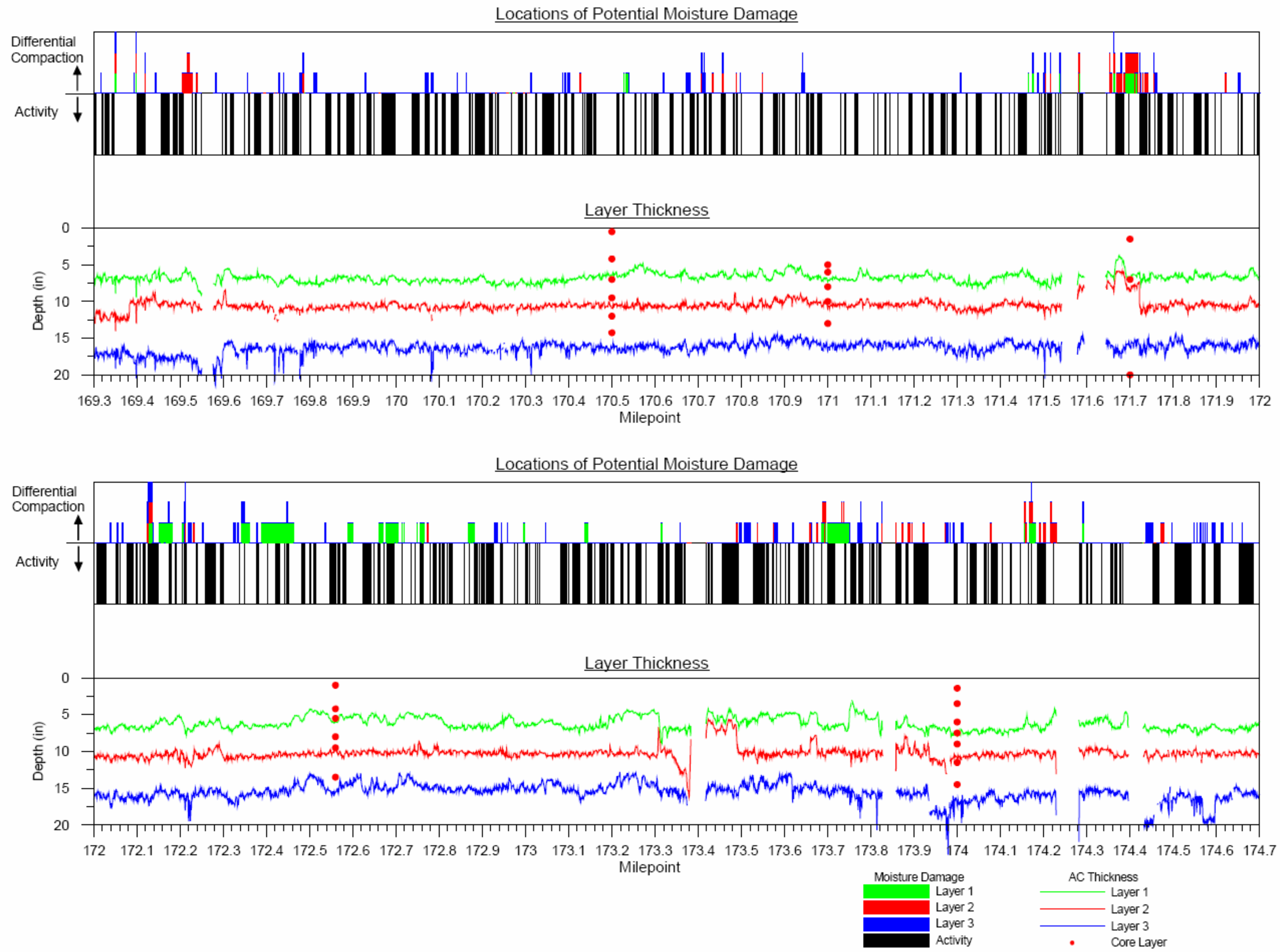


Figure 3.22: Locations of Potential Moisture Damage for the Cottage Grove – Martin Creek Project, Northbound Lanes.

Cores Obtained in 1995

Examination of the logs for the cores obtained in 1995 indicated that the pavement structure prior to the rehabilitation activity in 1997 consisted of a thin emulsified asphalt concrete (E-Mix) layer over three dense-graded (B-Mix) layers, each comprised of two lifts, over a plant mix bituminous base (PMBB) layer. The core logs also indicated that the first (lowest) dense-graded layer was placed in 1954, the second dense-graded layer was placed in 1965, and the top dense-graded layer was placed in 1975.

For the cores obtained from the southbound lanes, the following additional observations were made from the core logs:

- Most layers (lifts) identified in the core logs were reported to be in “Good” condition, with a few reported to be in “Fair” condition (mile points 155.04, 156.45, 158.83, 159.33, 160.21, 161.00, and 161.39), and only one reported to be in “Poor” condition (the surface patch at MP 160.21).

- Numerous core logs indicated “voids” or “some voids” in the dense-graded (B-Mix) layers placed in 1954 and 1965, and in the PMBB layer (mile points 155.36, 155.99, 156.45, 156.70, 157.48, 158.83, 160.21, 161.00, and 161.39).

- Stripping (“initial stages”) was identified in the top dense-graded (B-Mix) layer placed in 1975 at mile points 158.83 and 155.04.

- The presence of “free AC on side of core” was identified on the core logs for mile points 155.04 and 155.36 for the dense-graded (B-Mix) layer placed in 1975, possibly indicating bleeding or stripping.

- Several layers were identified as being a patch, namely at MP 155.04 (at 9½ inch depth), MP 156.45 (top 1½ inches), MP 156.70 (assumed to be at the surface), MP 159.33 (top 1½ inches), MP 160.21 (top 3 inches, which appear to be two patches 1½ inches each), and at MP 161.94 (top ½ inch, approximately).

For the cores obtained from the northbound lanes, the following additional observations were made from the core logs:

- Most layers (lifts) identified in the core logs were reported to be in “Good” condition, with a few reported to be in “Fair” condition (mile points 156.65, 157.31, 161.21, and 161.61), and a few reported to be in “Poor” condition (mile points 156.65 and 161.61).

- Several core logs indicated “few voids”, “some voids”, “many voids”, or “voids” mostly in the dense-graded (B-Mix) layer placed in 1965 (mile points 155.64, 156.50, 156.65, 157.00, 157.31, 158.19, 158.92, 159.43, 160.05, 161.04, 161.20, 161.61, and 162.04).

- Stripping was identified in the dense-graded layers at mile points 156.65, 157.00, 157.31, and 161.61.

- The presence of "free AC on side of core" was identified on the core logs for mile points 157.00, 157.31, 158.19, 158.92, 159.43, 160.05, 161.04, and 161.20, possibly indicating bleeding or stripping.

- Several layers were identified as being a patch, namely at MP 156.65 (top 1¾ inches), MP 157.31 (top 2¼ inches), MP 159.43 (top 1 inch), MP 161.04 (top 1 inch), MP 161.20 (two surface patches; ¾ inch and ½ inch, respectively), MP 161.61 (three surface patches; 1 inch, 1¼ inches, and 1 inch, respectively), and MP 162.04 (three surface patches; 1 inch, ½ inch, and 2¾ inches, respectively—however, the 2¾ inch "patch" was suspected to be the top dense-graded layer).

- Some core logs indicated that the core had been "drilled in dual wheel rut", namely at mile points 157. 20 and 158.19, and one log (for MP 157.31) indicated that the core was obtained from a patch with 2 inch ruts where the surface was "humped" up near the fog line.

Cores Obtained in 2006

Two sets of twelve cores (six from the outer wheel track of the outside lane and six from between the wheel tracks of the outside lane) were obtained by ODOT personnel at mile points 156.90 and 157.20 in the southbound lane on October 16, 2006. Appendix H includes the pavement core logs for these cores.

Examination of the core logs obtained from MP 156.90 indicated all layers (lifts) to be in "Good" condition, except for the lift immediately below the dense-graded HMA inlay (i.e., at a depth of approximately 3 to 4 inches), which was rated to be in "Fair" condition. It should be noted that the core logs indicate that the combined thickness of the dense-graded HMA inlay and open-graded overlay at this location was only 3 inches.

Examination of the core logs obtained from MP 157.20 indicated the pavement layers were in the same conditions as those obtained from MP 156.90, except that the combined thickness of the dense-graded HMA inlay and open-graded overlay was approximately 4 inches. As was the case at MP 156.90, the layer immediately below the dense-graded HMA inlay at MP 157.20 was identified as being in "Fair" condition with this layer being between 1½ to 2 inches thick (i.e., at a depth of approximately 4 to 6 inches).

Figures 3.23 through 3.25 provide photos of cores obtained from MP 157.20. In each case, the photos show cores obtained from the same longitudinal location, with the photo on the left showing the core obtained from between the wheel tracks and the photo on the right showing the core obtained from the outside wheel track, and with both cores taken from the outside (slow), southbound lane. The photos show the principal layers of bound material. The top two layers comprise the layers placed during the rehabilitation activity in 1997. Below these are dense-graded HMA layers over a plant mix bituminous base (PMBB) layer. An internal ODOT memorandum dated July 28, 1997 indicated that the top portion of the dense-graded layers (i.e., the 1½ to 2 inch dark layer immediately below the dense-graded HMA inlay) to be a C-Mix layer that was placed in 1977. Note that the core logs indicated this to be a B-Mix. This layer was partially milled during the rehabilitation activity in 1997. The approximately 4-inch layer (in two lifts of approximately 2 inches each) below the C-Mix layer was reported to be a B-Mix layer placed in 1965. Below this, the memorandum indicates the PMBB layer to be approximately 3 inches thick, also placed in 1965.

As expected, the presence of voids in the open-graded surface and PMBB layers were apparent as shown in Figures 3.23 through 3.25. Some voids also appeared to present in the dense-graded HMA inlay and in the top lift of the B-Mix layer as evidenced in Figures 3.23 and 3.24.

Figure 3.24 shows a break in the C-Mix layer for the core obtained from the outside wheel track, but not for the core obtained from between the wheel tracks. Four of the six cores obtained from the outside wheel track broke at approximately the same depth, whereas only one core obtained from between the wheel tracks broke at this depth as indicated in Figure 3.25.

Figure 3.25 shows that the core obtained from the outside wheel track broke at both interfaces of the C-Mix layer. It also appears that stripping had occurred in the C-Mix and underlying B-Mix layers as evidenced by the core obtained from between the wheel tracks.

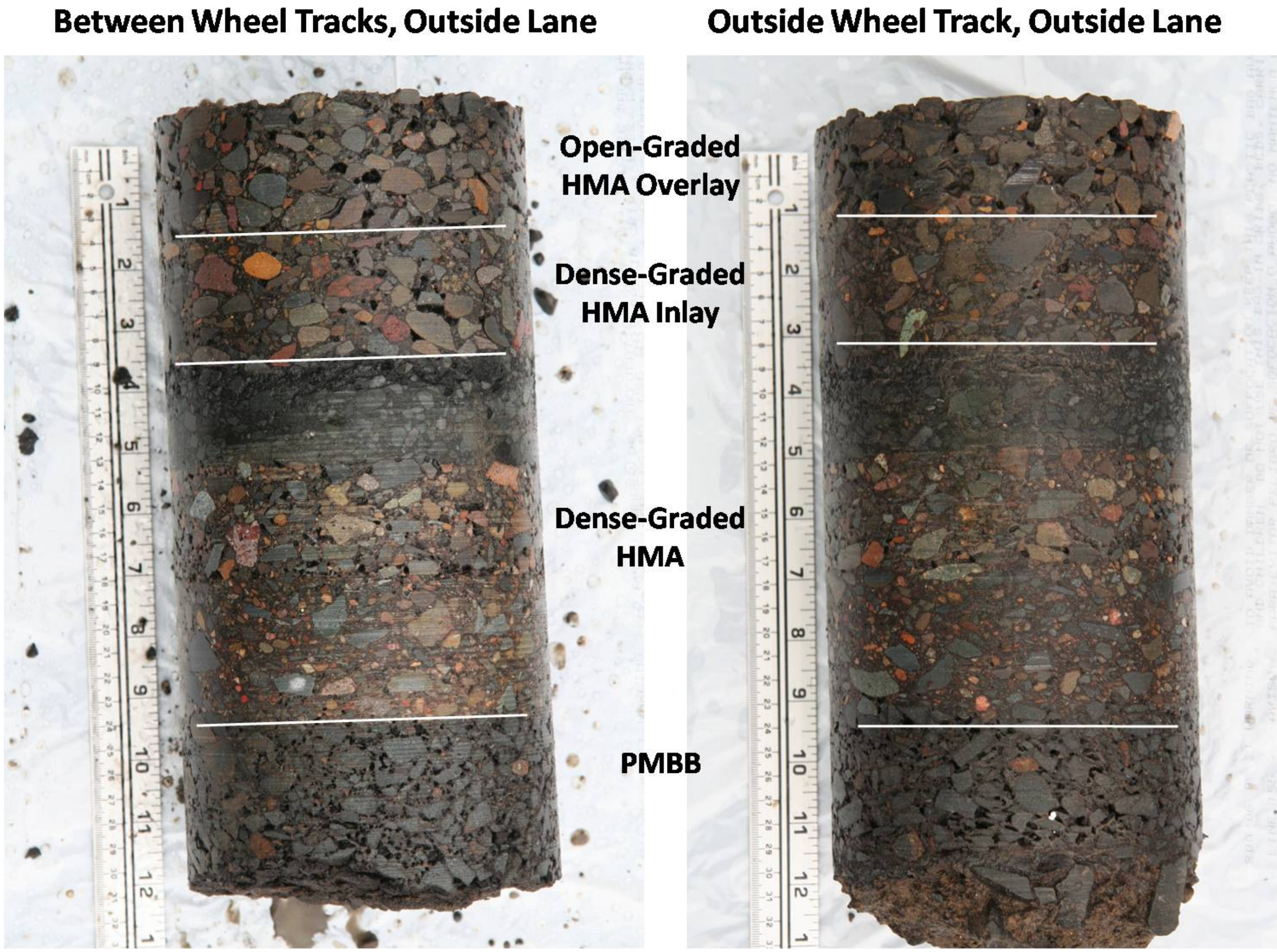


Figure 3.23: Adjacent, Intact Cores Obtained from MP 157.20 of the Anlauf – Elkhead Road Project.

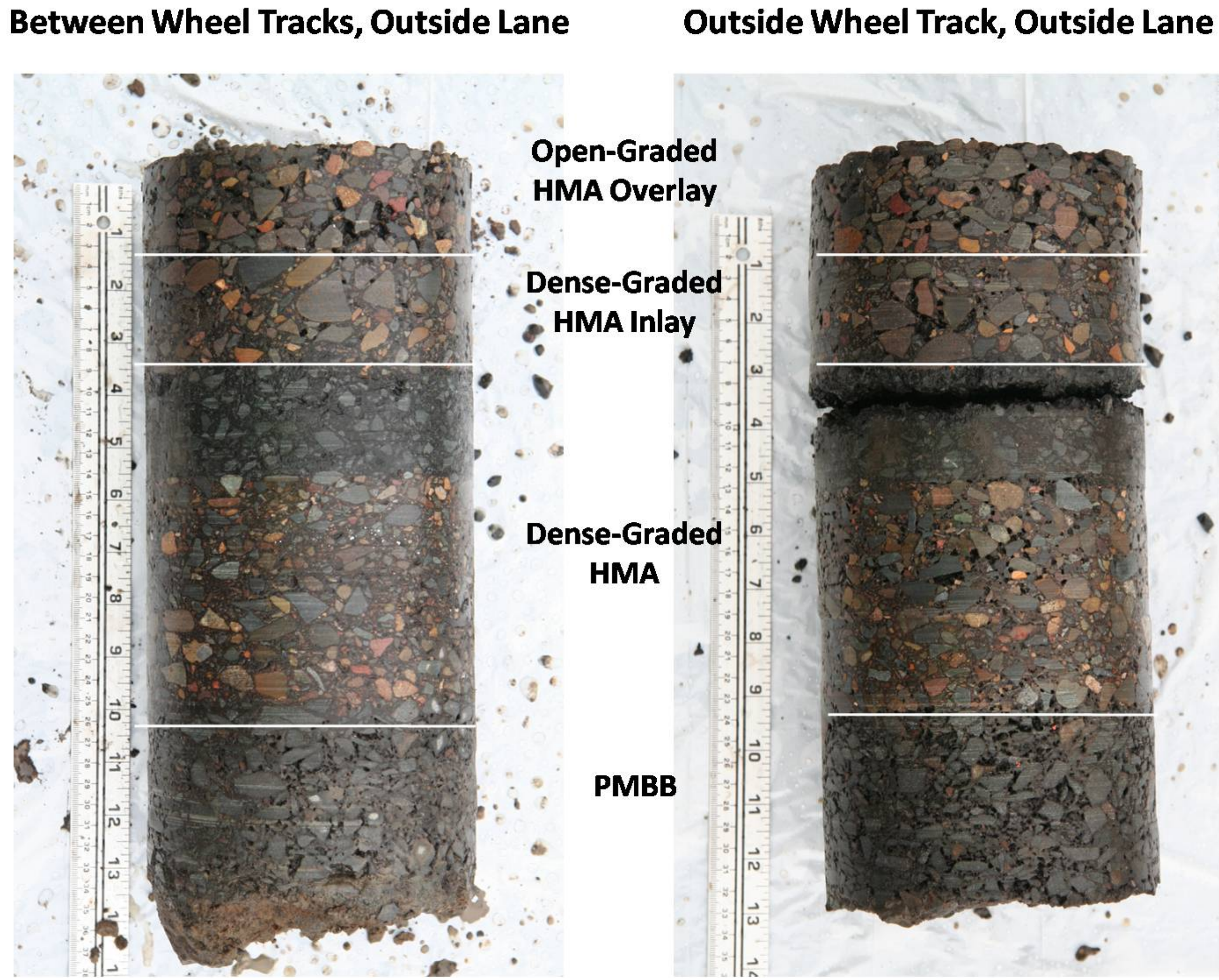


Figure 3.24: Typical Difference Between Adjacent Cores Obtained from MP 157.20 of the Anlauf – Elkhead Road Project.

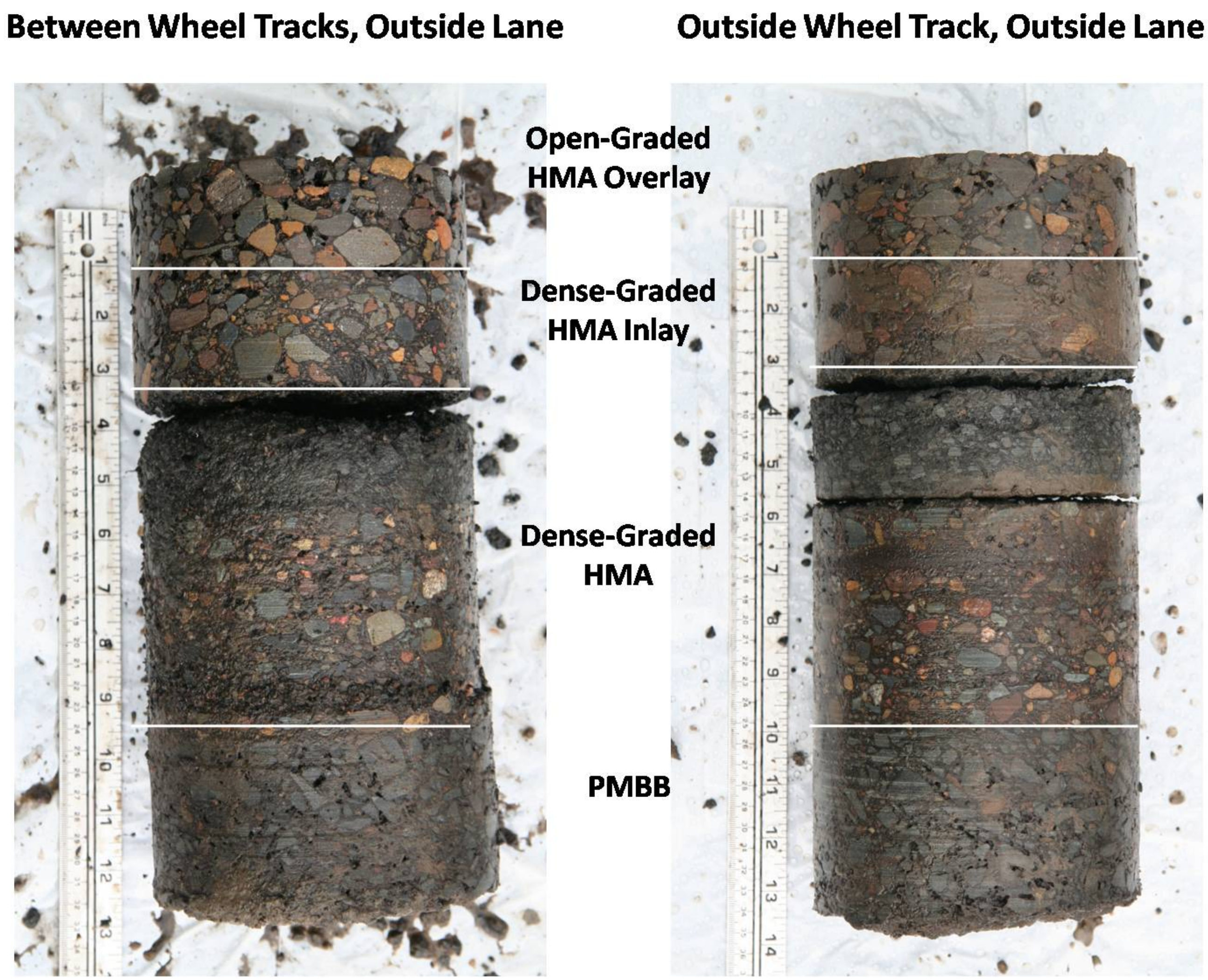


Figure 3.25: Adjacent, Broken Cores Obtained from MP 157.20 of the Anlauf – Elkhead Road Project.

Ground Penetrating Radar Survey

For the Anlauf – Elkhead Road project, data were collected from mile points 161.50 to 161.00 and from mile points 158.50 to 157.00 in the southbound direction. In the northbound direction, data were collected from mile points 156.30 to 157.50 and from mile points 161.00 to 161.50. These locations were selected based on the presence of surface distresses in the form of stripping, patching, and rutting along these stretches of pavement.

Figures 3.26 and 3.27, taken from the subcontractors report (Appendix K), show locations of potential moisture damage and layer thickness, based on analyses of the GPR data, in the southbound and northbound directions, respectively.

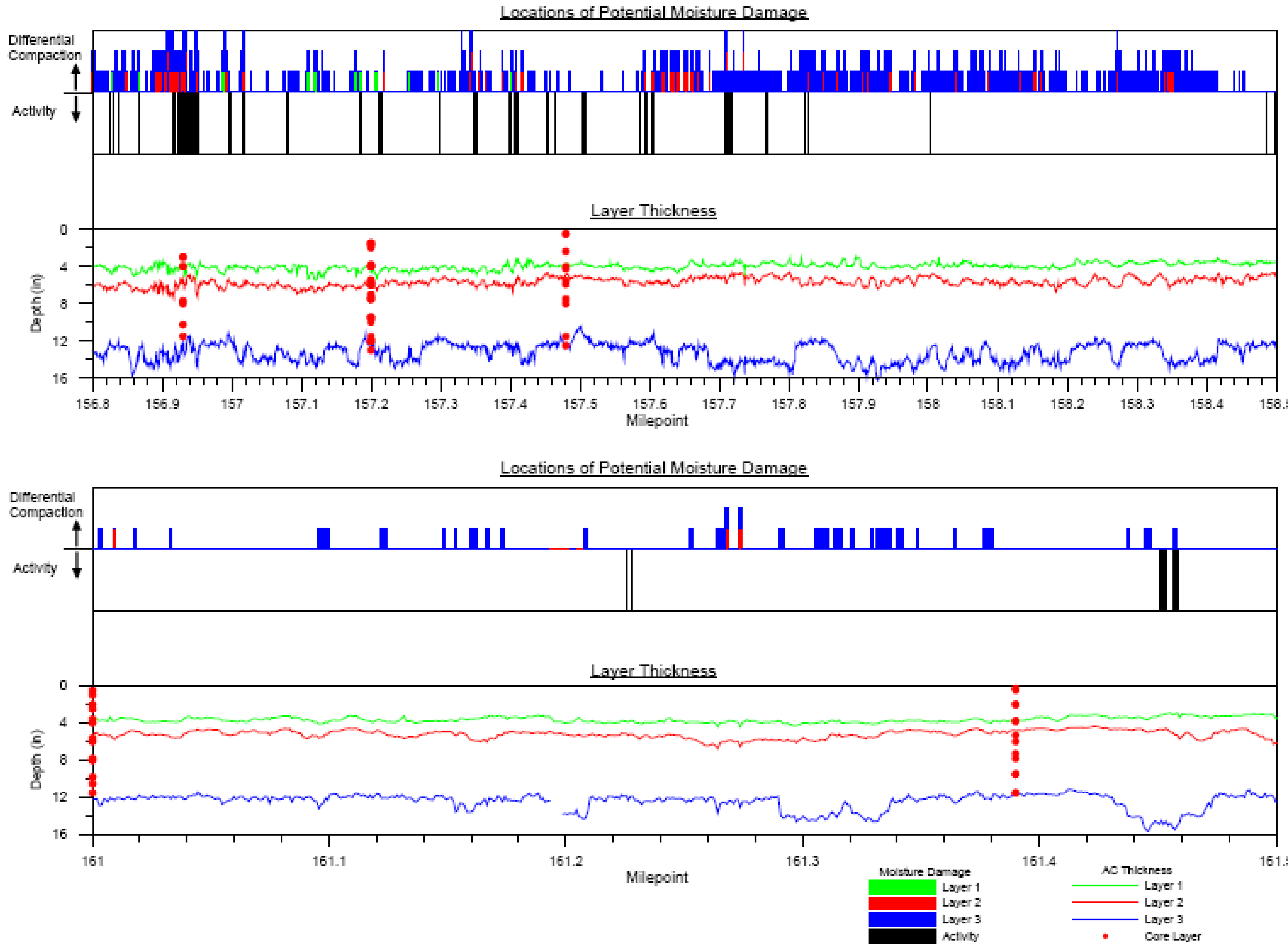


Figure 3.26: Locations of Potential Moisture Damage for the Anlauf – Elkhead Road Project, Southbound Lanes.

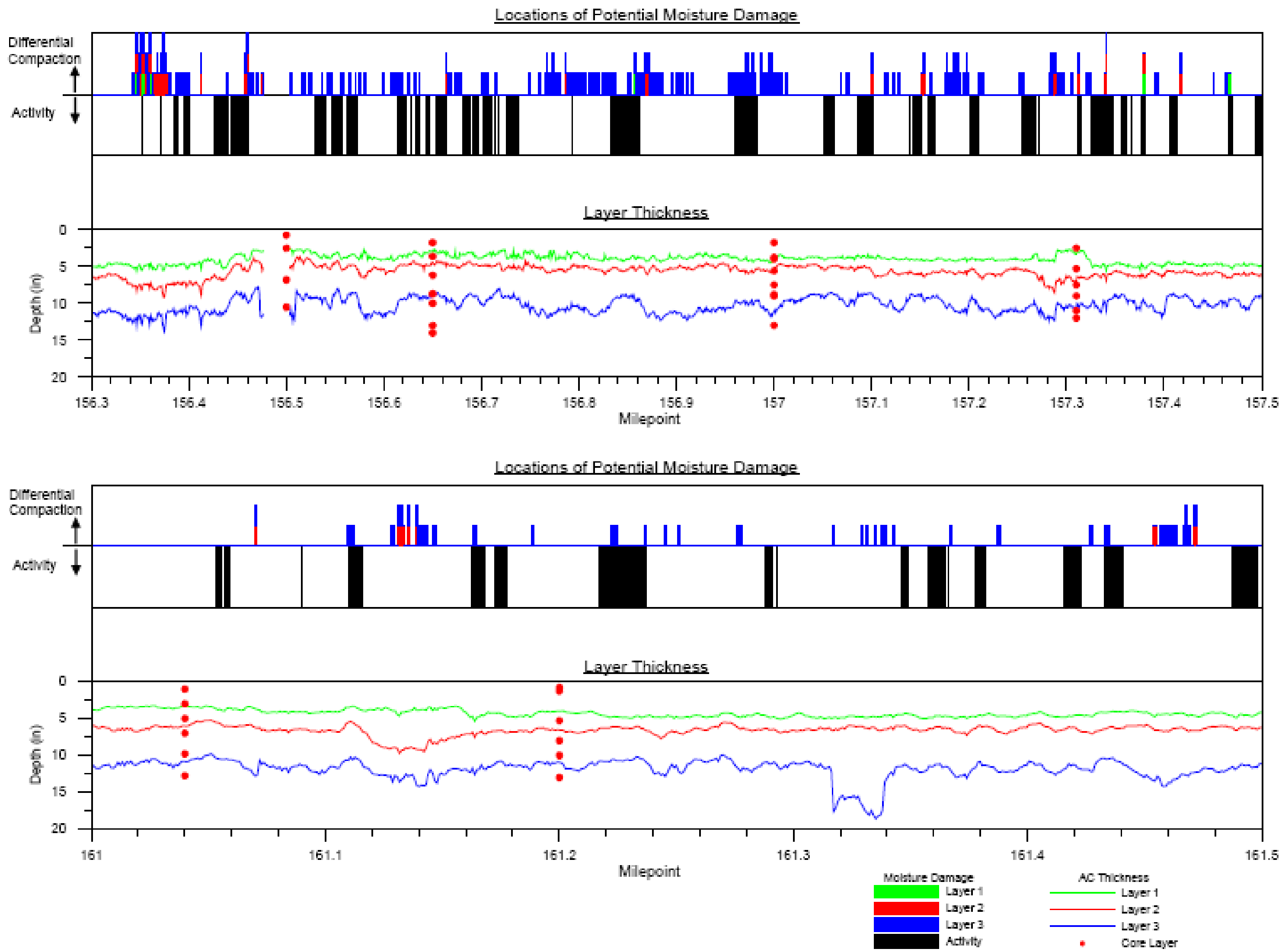


Figure 3.27: Locations of Potential Moisture Damage for the Anlauf – Elkhead Road Project, Northbound Lane

Table 3.4, reproduced from Table 3 of the subcontractors report (Appendix K), provides a summary of results for the Anlauf – Elkhead Road project derived from the GPR data analyses. It shows the percentage of the section length where moisture damage has likely occurred based on the magnitude of the Activity Index and "Differential Compaction" (i.e., differential layer thickness). For example, the results suggest that 19.8% of the length of pavement between mile points 156.30 and 157.50 (i.e., approximately 0.24 miles of the outside lane) in the northbound (NB) direction had moisture-induced damage. The results for the "Differential Compaction" indicator are broken down by layer, where "Layer 1" corresponds to the top layer detected, "Layer 2" corresponds to the layer detected beneath "Layer 1", and "Layer 3" corresponds to the layer detected beneath "Layer 2". The thickness of these layers varies along the length of each section as indicated in Figures 3.26 and 3.27.

Table 3.4 Summary of Results for GPR Data Moisture Damage Analysis for the Anlauf – Elkhead Road Project.

			Predicted Moisture Damage (Percent of Section Length)			
				Differential Compaction		
Direction	Start MP	End MP	Activity Index	Layer 1	Layer 2	Layer 3
NB	156.30	157.50	19.8	1.3	5.1	26.4
NB	161.00	161.50	15.5	0.0	1.7	8.5
SB	161.50	161.00	0.9	0.0	0.6	11.7
SB	158.50	157.00	2.1	1.6	10.3	46.7
SB	155.50	154.50	Missing Data			

3.4.4 Garden Valley – Roberts Creek, Interstate 5 MP 117.74 – 125.0

The field investigations for this project involved coring and trenching operations as well as a GPR survey. The following paragraphs summarize the results of these activities.

Coring

Two sets of twelve cores (six from the outer wheel path of the outside lane and six from between the wheel paths of the outside lane) were obtained from mile points 118.0 and 118.6 approximately 4 years after the rehabilitation activity. Figure 3.28 shows a comparison of adjacent cores characterizing the typical condition of the pavement layers. In most cases, the cores delaminated at a depth of approximately 11 inches (280 mm). The layer immediately below this was rated on the core logs as being in "Fair" or "Poor" condition in most cases, and stripping was identified for this layer on several of the logs. Appendix I includes the pavement core logs for these cores.

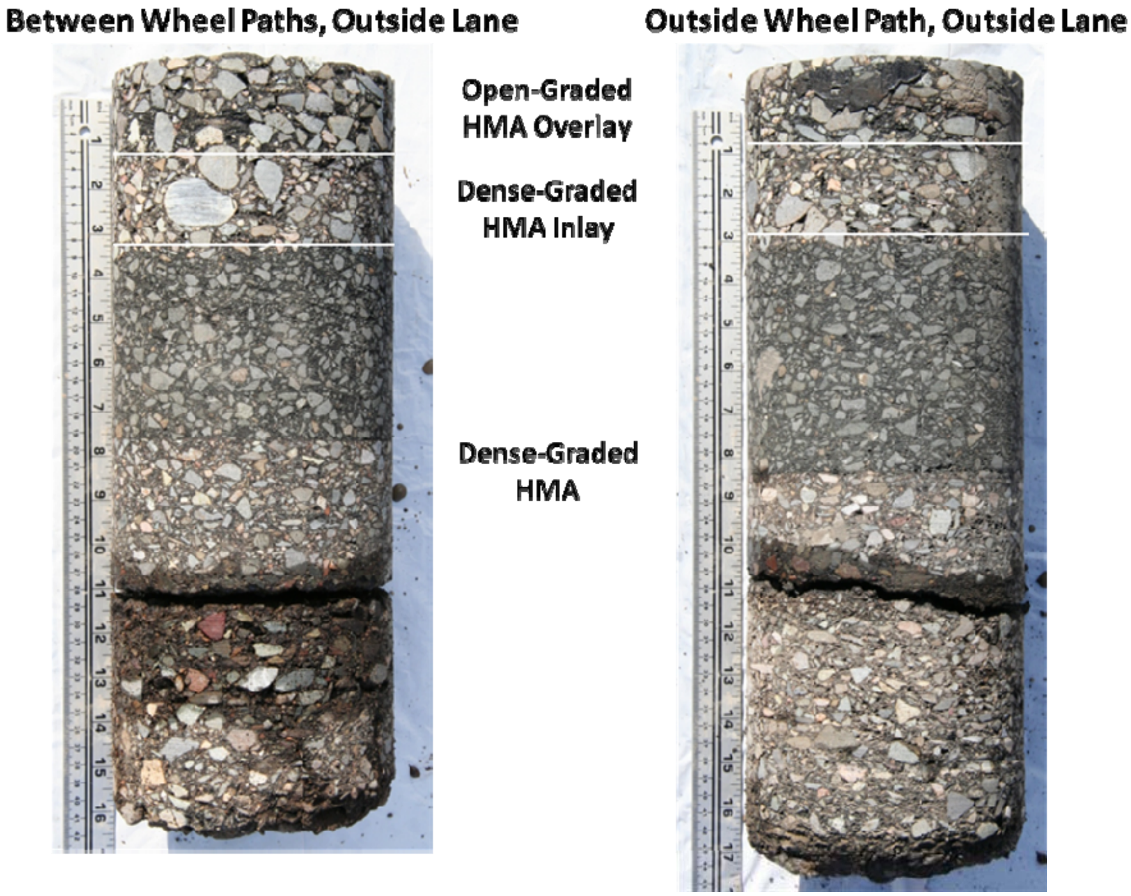


Figure 3.28: Adjacent Cores Obtained from the Garden Valley – Roberts Creek Project.

Trenching

One trench was excavated from the outside, southbound travel lane on a gradual uphill gradient at MP 118.5 in an area with only minimal rutting. Figure 3.29 shows a photo of the uphill trench wall indicating that water was still wetting the trench wall approximately 1½ hours after excavation along portions of most of the bound layers, and the lowest approximately 6 inches (150 mm) appeared almost completely wet. Stripping was also observed in the lowest approximately 4 inches (100 mm) of the bound layers.

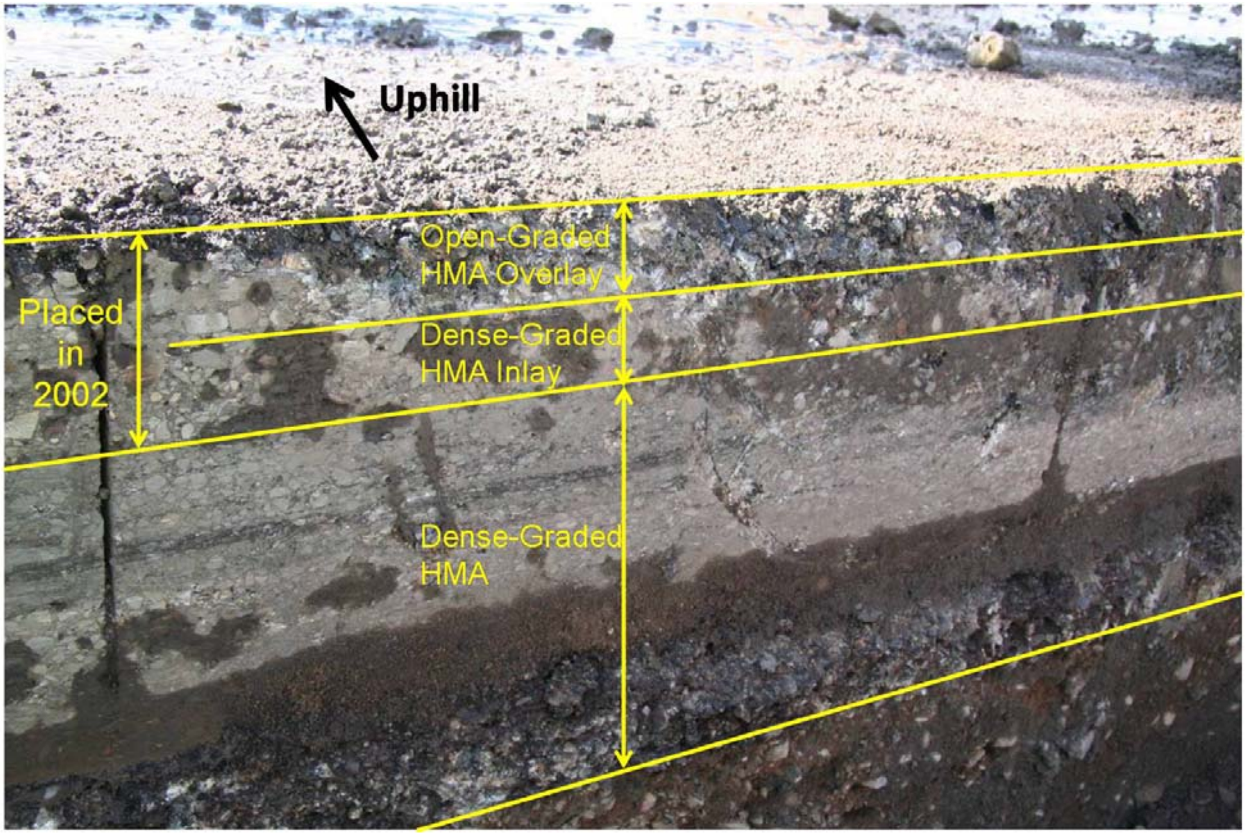


Figure 3.29: Garden Valley – Roberts Creek Trench Wall Approximately 1½ Hours after Excavation.

Ground Penetrating Radar Survey

Data were collected from mile points 117.70 to 124.70 in the northbound direction and from mile points 117.70 to 133.80 in the southbound direction. These locations were selected based on the presence of surface distresses in the form of stripping, patching, and rutting along these stretches of pavement.

Figures 3.30 through 3.33, taken from the subcontractors report (Appendix K), show locations of potential moisture damage and layer thickness based on analyses of the GPR data.

Table 3.5, reproduced from Table 3 of the subcontractors report (Appendix K), provides a summary of results for the Garden Valley – Roberts Creek project derived from the GPR data analyses. It shows the percentage of the section length where moisture damage has likely occurred based on the magnitude of the Activity Index and "Differential Compaction" (i.e., differential layer thickness). For example, the results suggest that 40.2% of the length of pavement surveyed between mile points 117.70 and 124.70 (i.e., approximately 1.2 miles of the outside lane) in the northbound (NB) direction had moisture-induced damage. The results for the "Differential Compaction" indicator are broken down by layer, where "Layer 1" corresponds to the top layer detected, "Layer 2" corresponds to the layer detected beneath "Layer 1", and "Layer 3" corresponds to the layer detected beneath "Layer 2". The thickness of these layers varies along the length of each section as indicated in Figures 3.30 through 3.33.

Table 3.5 Summary of Results for GPR Data Moisture Damage Analysis for the Garden Valley – Roberts Creek Project.

Direction	Start MP	End MP	Predicted Moisture Damage (Percent of Section Length)			
			Activity Index	Differential Compaction		
				Layer 1	Layer 2	Layer 3
NB	117.70	119.00	6.3	14.6	18.4	8.8
NB	120.25	121.00	18.0	0.8	5.8	9.5
NB	123.70	124.70	15.9	2.4	10.4	18.4
SB	119.00	117.70	4.7	3.9	12.3	20.0
SB	121.00	120.25	14.7	1.0	3.6	13.7
SB	122.00	121.50	14.4	3.6	4.0	11.0
SB	123.80	122.70	12.7	3.7	4.7	16.1

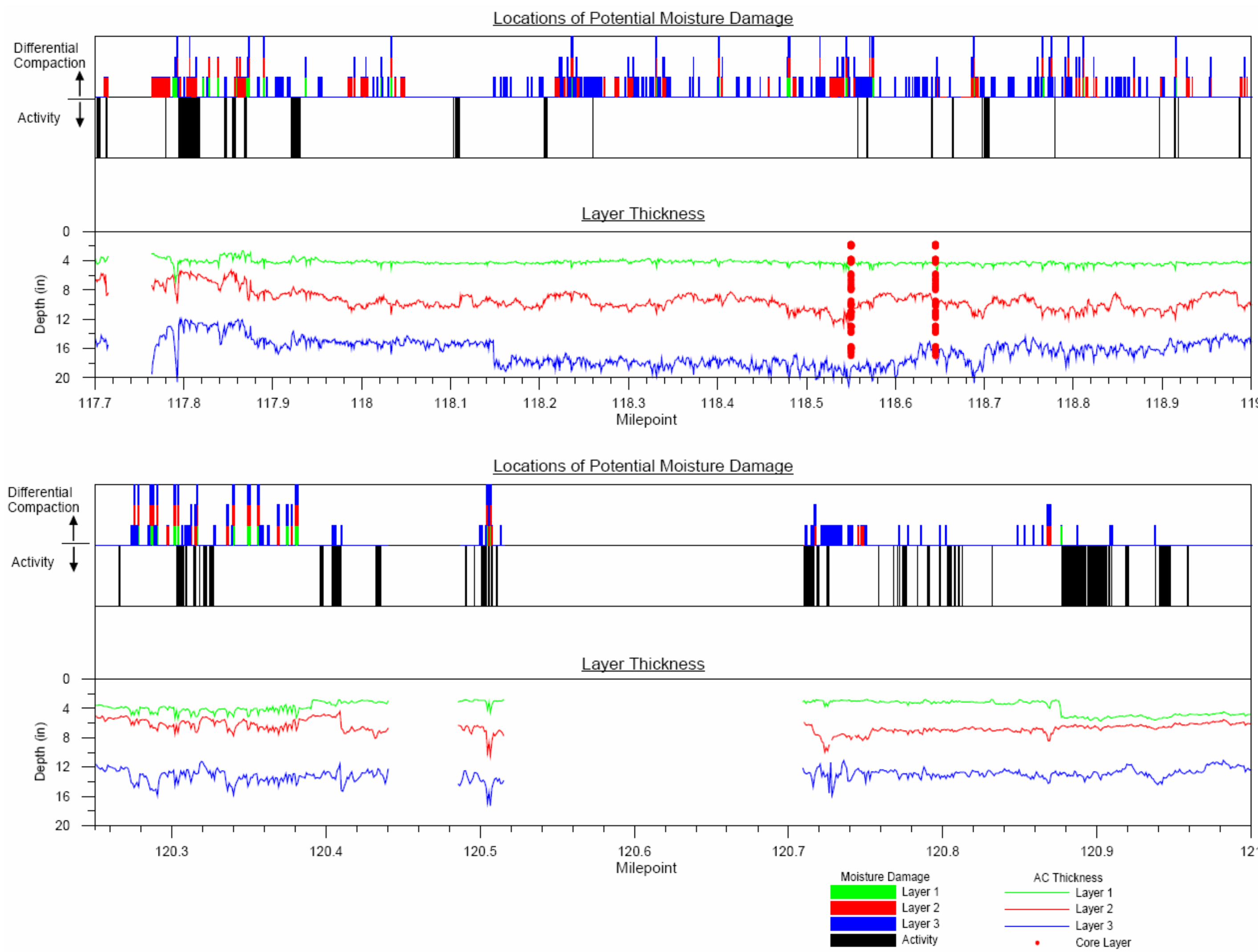


Figure 3.30: Locations of Potential Moisture Damage for the Garden Valley – Roberts Creek Project, MP 117.7 to MP 121.0, Southbound Lanes.

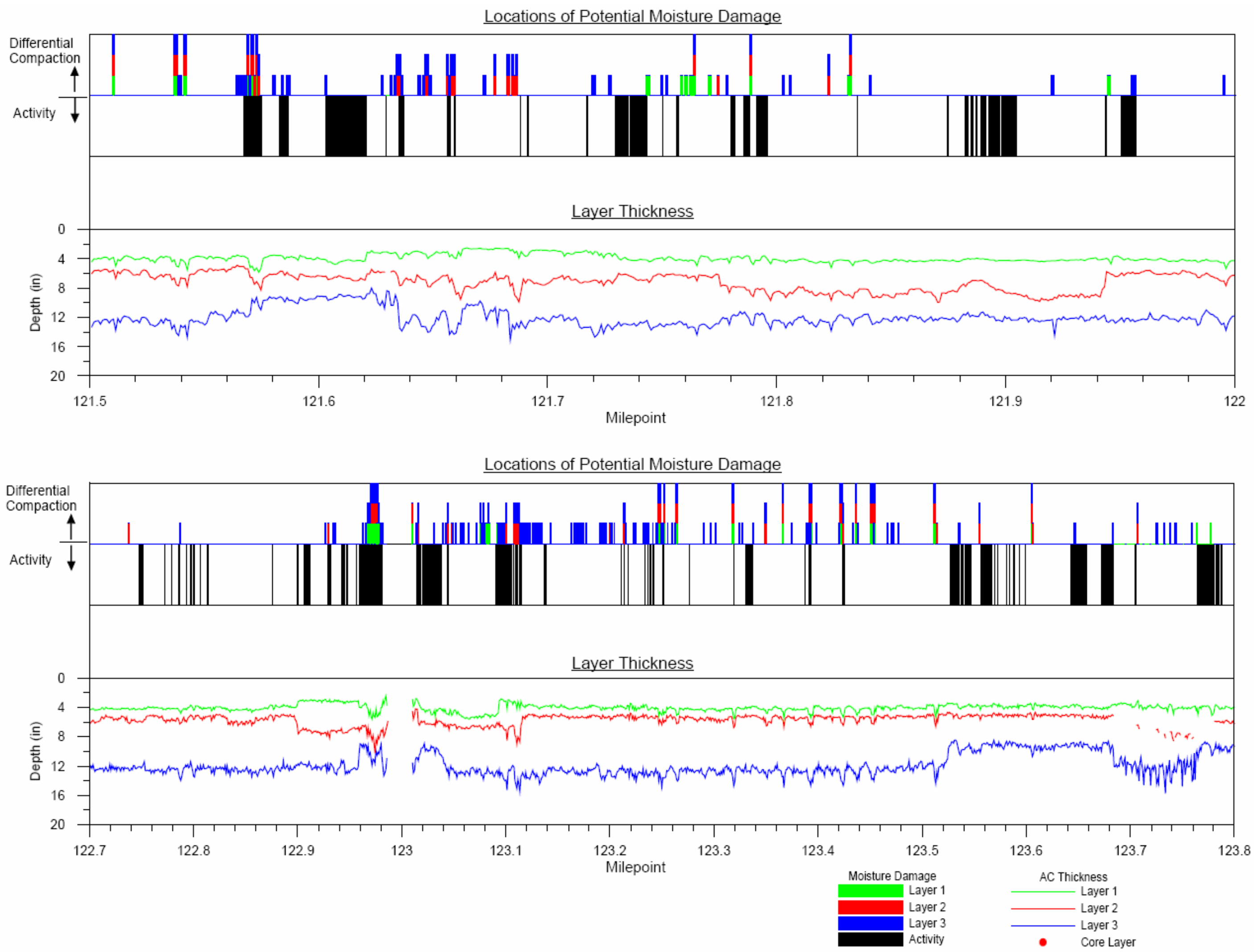


Figure 3.31: Locations of Potential Moisture Damage for the Garden Valley – Roberts Creek Project, MP 121.5 to MP 123.8, Southbound Lanes.

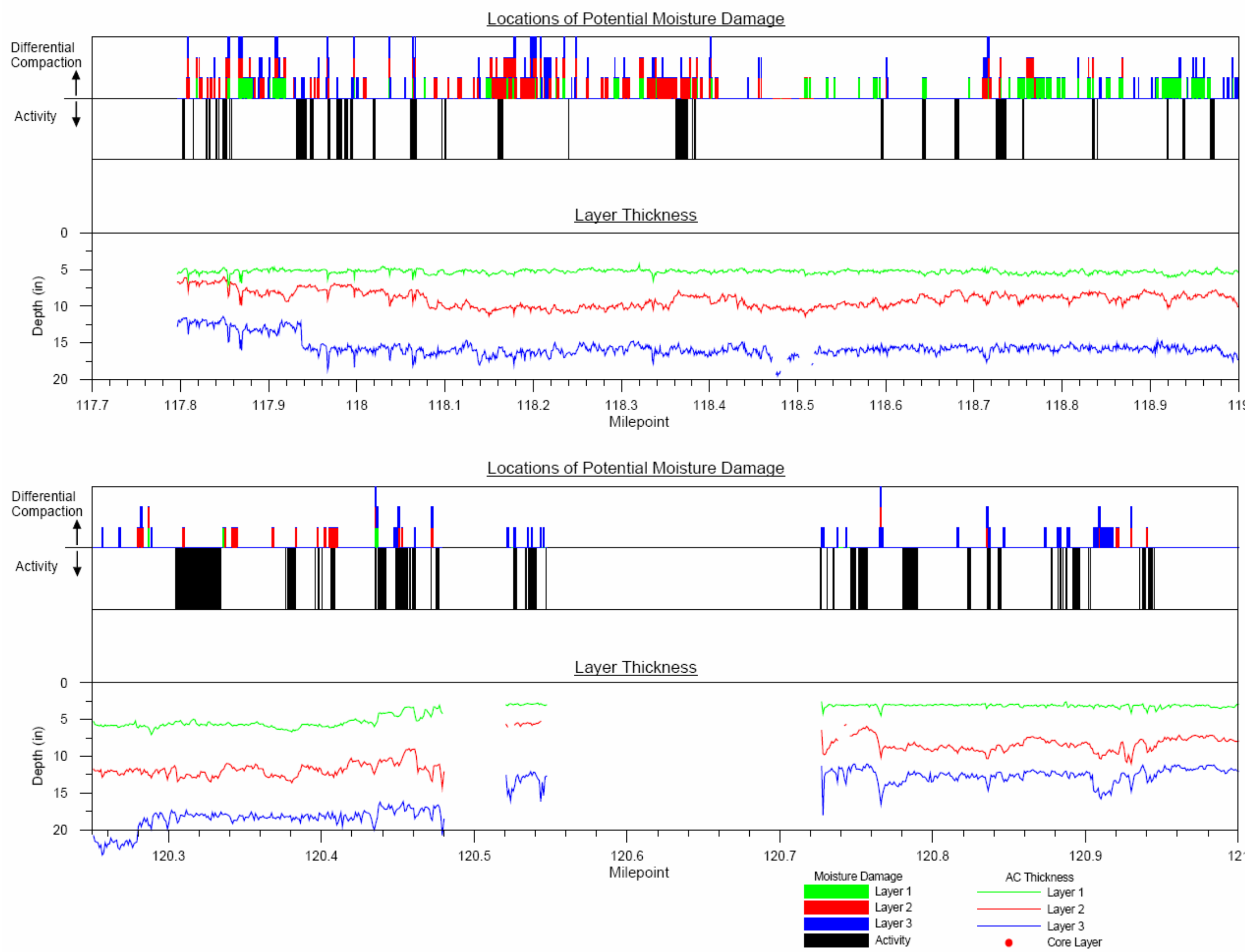


Figure 3.32: Locations of Potential Moisture Damage for the Garden Valley – Roberts Creek Project, MP 117.7 to MP 121.0, Northbound Lanes.

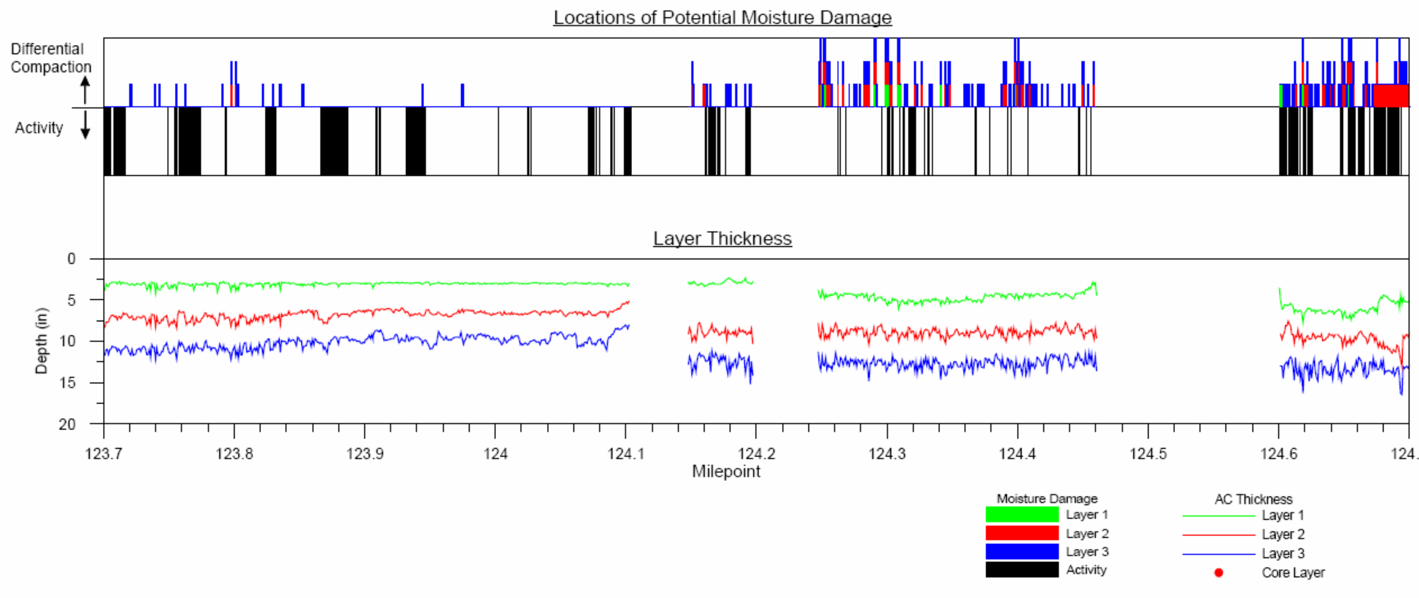


Figure 3.33: Locations of Potential Moisture Damage for the Garden Valley – Roberts Creek Project, MP 123.7 to MP 124.7, Northbound Lanes.

3.4.5 Vets Bridge – Myrtle Creek, Interstate 5 MP 109.0 – 112.5

The field investigations for this project involved coring and trenching operations as well as a GPR survey. The following paragraphs summarize the results of these activities.

Coring

Twelve cores (six from the outer wheel path of the outside lane and six from between the wheel paths of the outside lane) were obtained from MP 109.9 approximately 3 years after the rehabilitation activity. Appendix J includes the pavement core logs for these cores.

Examination of the logs for the cores obtained from the outside wheel path indicated an HMA thickness of 8½ to 10¼ inches with the bottom layer of these cores, at a depth of approximately 5 inches and greater, identified as being in "Fair" condition on four of the six core logs. Nearly identical notations were made on the logs of the cores obtained from between the wheel paths.

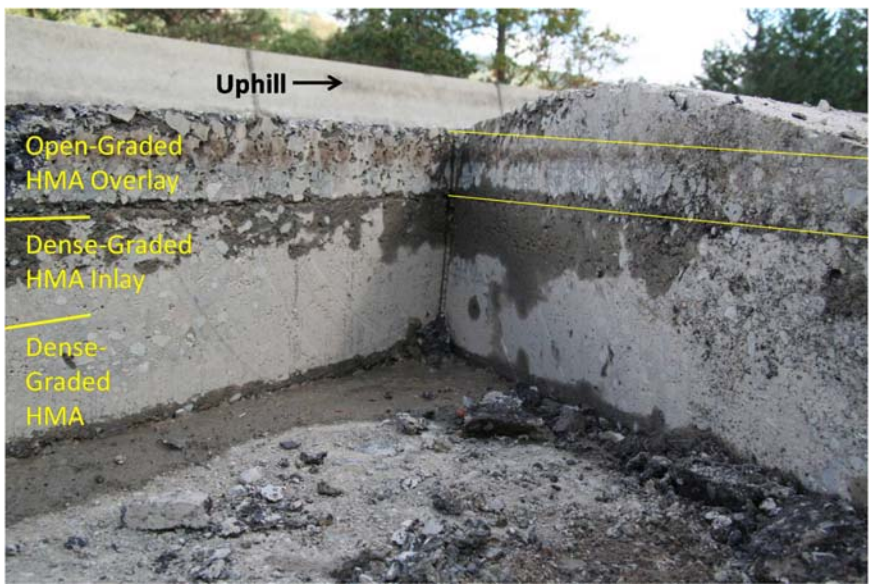
Trenching

One trench was excavated from the southbound, outside travel lane on a gradual uphill gradient at MP 109.9 in an area with bleeding and deep rutting in the wheel paths. Figure 3.34 shows photos of the downhill trench wall shortly after excavation (Figure 3.34a) and the uphill and end walls of the trench approximately 2 hours after excavation (Figure 3.34b). In both cases, it is evident that the top approximately 3 inches (75 mm) of dense-graded HMA material contained a significant amount of moisture throughout the time the trench remained open. This was also true of the bottom approximately 2 inches (50 mm) of the bound layers. Figure 3.34b also shows the approximately 2½ inches (65 mm) outside ridge of the rut in the inside wheel path through which the trench was excavated.

Ground Penetrating Radar Survey

For the Vets Bridge – Myrtle Creek project, data were collected between mile points 109.00 and 112.50 in both directions. Figures 3.35 and 3.36, taken from the subcontractors report (Appendix K), show locations of potential moisture damage and layer thickness, based on analyses of the GPR data, in the southbound and northbound directions, respectively.

a) Downhill View of Trench Shortly after Excavation.



b) Uphill and End View of Trench Walls Approximately 2 hours after Excavation.

Figure 3.34: Vets Bridge – Myrtle Creek Project Trench Walls.

Table 3.6, reproduced from Table 3 of the subcontractors report (Appendix K), provides a summary of results for the Vets Bridge – Myrtle Creek project derived from the GPR data analyses. It shows the percentage of the section length where moisture damage has likely occurred based on the magnitude of the Activity Index and “Differential Compaction” (i.e., differential layer thickness). For example, the results suggest that 15.8% of the length of pavement between mile points 109.00 and 112.50 (i.e., approximately 0.55 miles of the outside lane) in the northbound (NB) direction had moisture-induced damage. The results for the “Differential Compaction” indicator are broken down by layer, where “Layer 1” corresponds to the top layer detected, “Layer 2” corresponds to the layer detected beneath “Layer 1”, and “Layer 3” corresponds to the layer detected beneath “Layer 2”. The thickness of these layers varies along the length of each section as indicated in Figures 3.35 and 3.36.

Table 3.6 Summary of Results for GPR Data Moisture Damage Analysis for the Vets Bridge – Myrtle Creek Project.

Direction	Start MP	End MP	Predicted Moisture Damage (Percent of Section Length)			
			Activity Index	Differential Compaction		
				Layer 1	Layer 2	Layer 3
NB	109.00	112.50	15.8	4.3	5.4	10.2
SB	112.50	109.00	19.0	14.4	14.2	25.0

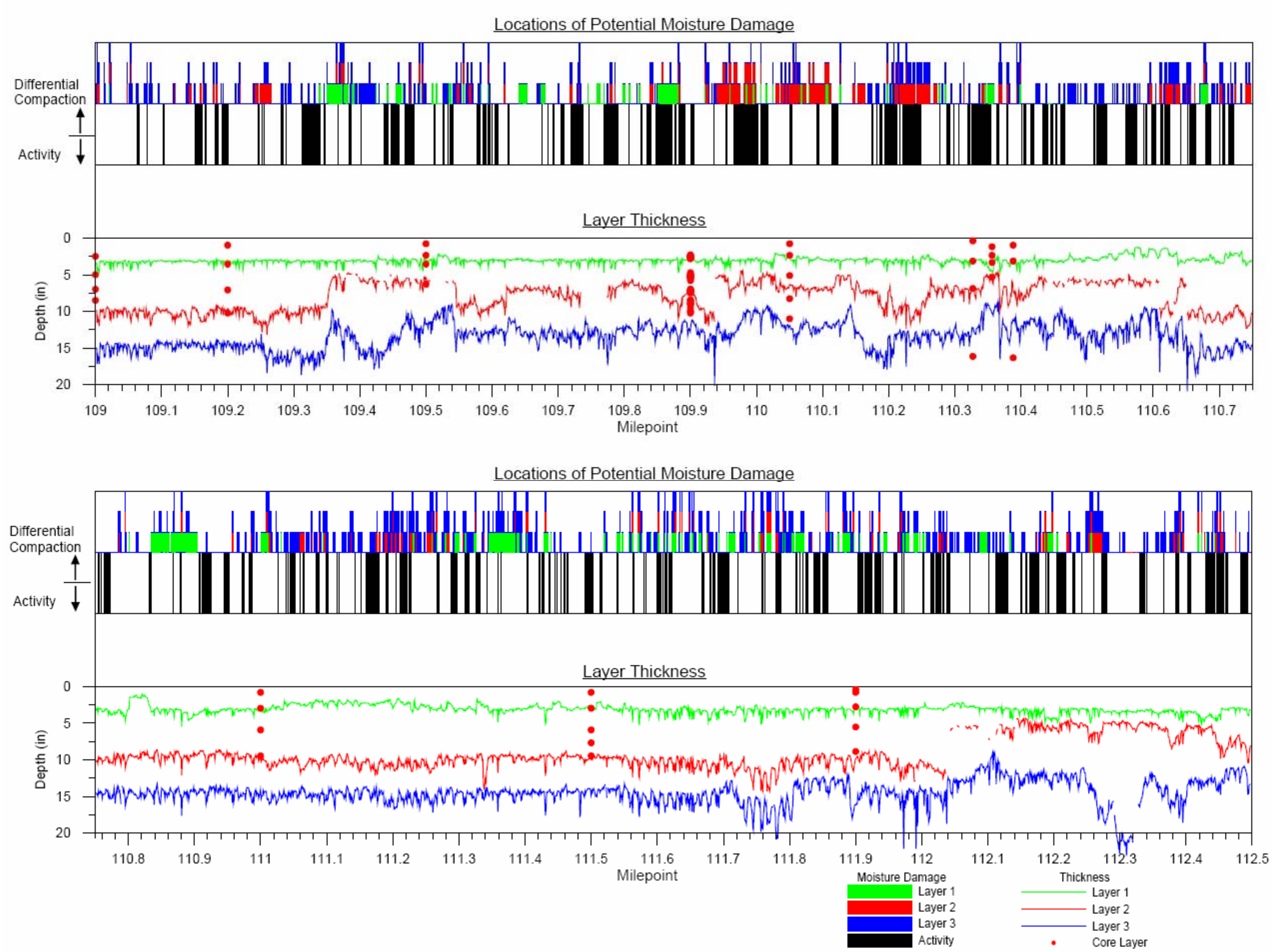


Figure 3.35: Locations of Potential Moisture Damage for the Vets Bridge – Myrtle Creek Project, Southbound Lanes.

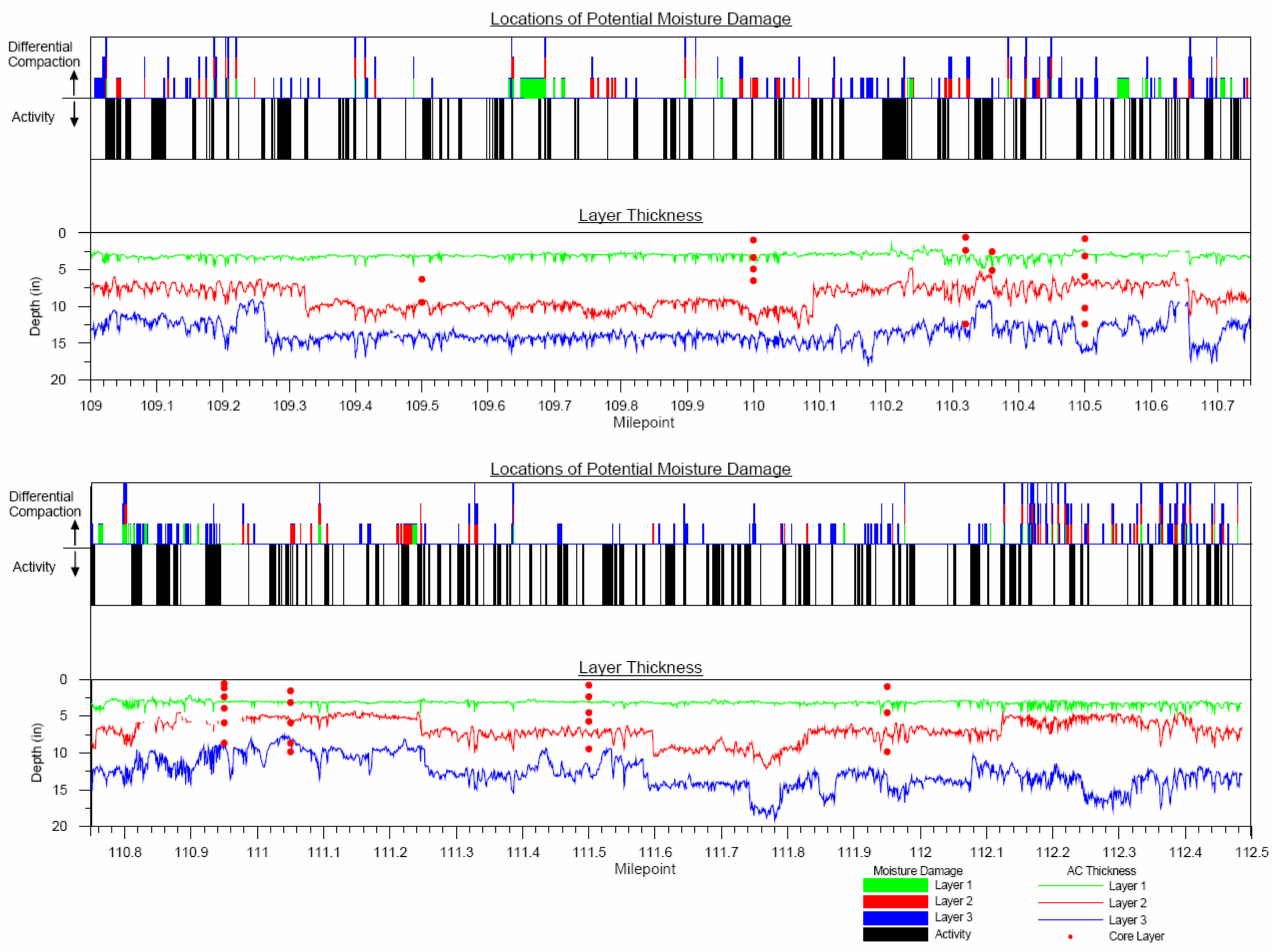


Figure 3.36: Locations of Potential Moisture Damage for the Vets Bridge – Myrtle Creek Project, Northbound Lanes.

4.0 LABORATORY STUDY

This section provides a brief description of the experiment plan followed by details of the test results derived from cores obtained from four of the projects.

4.1 EXPERIMENT PLAN

Cores obtained from four of the five projects were cut into slices representing individual lifts and tested to determine volumetric properties and permeability of the HMA layers. To accomplish this, the following tests were conducted:

1. Bulk specific gravity via parafilm method (modified AASHTO T 275) and CoreLok method (ASTM D 6752),
2. Theoretical maximum specific gravity (AASHTO T 209),
3. Air void content (AASHTO T 269).
4. Permeability (Florida DOT Test Method FM 5-65).

A sufficient number of cores were tested from the four projects to allow adequate replication for statistically sound investigation of the above properties. In addition, it should be emphasized that each core contained several individual layers (lifts), and most of the lifts were tested for the above properties.

4.2 RESULTS

4.2.1 Pleasant Valley – Durkee, Interstate 84 MP 317.5 – 327.3

A number of cores obtained from the Pleasant Valley – Durkee project were cut into slices and tested for bulk specific gravity and permeability. Unfortunately, theoretical maximum (Rice) specific gravity tests were not performed on the slices (due to insufficient time and resources); hence, air void contents of the slices were not determined. The results of the bulk specific gravity and permeability testing are provided in the following paragraphs.

Bulk Specific Gravity – Parafilm Method

Cores were tested for bulk specific gravity following the procedure described in AASHTO T 275 with the important exception that parafilm, not paraffin, was used to coat the specimens. Table 4.1 summarizes the results from nine cores obtained from the eastbound direction and four cores obtained from the westbound direction.

Bulk Specific Gravity – CoreLok Method

Cores were also tested for bulk specific gravity in accordance with ASTM D 6752 using the CoreLok device to vacuum seal the test specimens in the plastic bags used in the test. Table 4.2 summarizes the results from nine cores obtained from the eastbound direction and four cores obtained from the westbound direction.

Comparison of Bulk Specific Gravity Measurements

Figures 4.1 and 4.2 compare the bulk specific gravity test results obtained from the method employing parafilm to those obtained from the method utilizing the CoreLok device for the cores taken in the eastbound and westbound directions, respectively. Note that, in nearly all cases, the procedure utilizing the CoreLok device resulted in greater values for the bulk specific gravity. However, given the inherent variability in results obtained from the two test methods (to be discussed later for the Vets Bridge – Myrtle Creek project in Section 4.2.5), the results suggest little or no difference (statistically) between results derived from the two test methods.

Further, the results suggest little or no statistical difference between the SMA layer (nominal depth <1¾ inches) and the top lift of the inlayed HMA layer (nominal depth 1¾ - 3¾ inches) except, perhaps, in isolated areas (e.g., MP 325.28 eastbound and MP 326.75 eastbound, Figure 4.1). However, there appears to be a trend of decreasing bulk specific gravity (and, hence, density) with depth (but, again, not likely to be statistically significantly different in most cases).

Permeability

Several cores were tested for water permeability in accordance with Florida Department of Transportation test method FM 5-565, which is a falling head water permeability test (with initial head of approximately 2½ ft) used to measure water conductivity through compacted bituminous mixture samples. The test measures the permeability of the mixture in the vertical flow direction. Table 4.3 summarizes the results indicating that most of the slices were either impermeable or had low permeability values. A few exceptions include:

- MP 325.28 eastbound at a nominal depth of 1¾ - 3¾ inches,
- MP 326.75 eastbound at nominal depths of <1¾ inches, 1¾ - 3¾ inches, and 3¾ - 5½ inches,
- MP 321.29 westbound at a nominal depth of 1¾ - 3¾ inches, and
- MP 324.50 westbound at nominal depths of 1¾ - 3¾ inches and 5½ - 7½ inches.

The results suggest that the SMA layer (nominal depth <1¾ inches) was essentially impermeable to water under low pressure, such as that of water on the surface of the pavement. The results also suggest that several of the HMA layers (nominal depth 1¾ - 9¼ inches) possessed the ability conduct water under low pressure. In particular, note that the test results for the core obtained at MP 325.00 in the eastbound direction (within 0.1 mile of the eastern trench) indicated measurable permeability for the top 3¾ inches of the inlayed

HMA layer (i.e., at a nominal depth between 1¾ and 5½ inches). Similarly, the test results for the cores obtained at MP 325.28, MP 325.29, and MP 325.39 in the eastbound direction (just downhill of the eastern trench) indicated measurable permeability for the top 2 inches of the inlayed HMA layer (i.e., at a nominal depth between 1¾ and 3¾ inches).

Table 4.1: Bulk: Specific Gravity Test Results (Parafilm Method) for the Pleasant Valley – Durkee Project.

Mile Point	Direction	Lane	Wheel Path	Core No.	Bulk Specific Gravity (Parafilm Method) at Nominal Depth (inches) of:						
					<1¾	1¾ - 3¾	3¾ - 5½	5½ - 7½	7½ - 9¼	9¼ - 11½	>11½
321.00	EB	R	O	P34	2.466	2.487	2.466	2.348	2.139	2.298	
324.00	EB	R	O	P6	2.442	2.465	2.398	2.472	2.482	2.252	2.322
324.50	EB		O	P27	2.490	2.480	2.424	2.427	2.217		
325.00	EB	R	O	P37-core #32	2.436	2.496	2.464	2.482			2.375
325.28	EB	R	O	P1	2.472	2.382	1.989	2.268	2.439		
325.29	EB		O	P4	2.447	2.428	2.354	2.327			
325.39	EB	R	O	P9	2.445	2.459	2.326	2.222			
325.40	EB		O	P13	2.433	2.480	2.456	2.376			
326.75	EB	R	O	P2 - core #42	2.351	2.527	2.472	2.504	2.454		2.360
321.29	WB		O	P15	2.463	2.445	2.396	2.458	2.414	2.212	
324.50	WB	L	I	P16-core #23	2.393	2.373	2.424	2.290			
324.75	WB		O	P33	2.423	2.459	2.471	2.350	2.344	2.267	
326.21	WB		O	P35-core #5	2.350	2.409	2.414	2.352	2.335	2.329	

Table 4.2: Bulk Specific Gravity Test Results (CoreLok Method) for the Pleasant Valley – Durkee Project.

Mile Point	Direction	Lane	Wheel Path	Core No.	Bulk Specific Gravity (CoreLok Method) at Nominal Depth (inches) of:						
					<1¾	1¾ - 3¾	3¾ - 5½	5½ - 7½	7½ - 9¼	9¼ - 11½	>11½
321.00	EB	R	O	P34	2.470	2.481	2.485	2.384	2.152	2.337	
324.00	EB	R	O	P6	2.475	2.498	2.482	2.483	2.503	2.334	
324.50	EB		O	P27	2.543	2.495	2.453	2.447			
325.00	EB	R	O	P37-core #32	2.477	2.524	2.487	2.526			2.402
325.28	EB	R	O	P1	2.494	2.390		2.286	2.384		
325.29	EB		O	P4	2.454	2.466	2.391	2.348			
325.39	EB	R	O	P9	2.433	2.475	2.346	2.267			
325.40	EB		O	P13	2.464	2.497	2.431	2.423			
326.75	EB	R	O	P2 - core #42	2.410	2.560	2.493	2.536	2.487		2.384
321.29	WB		O	P15	2.478	2.470	2.408	2.472	2.423	2.254	
324.50	WB	L	I	P16-core #23	2.436	2.396	2.441	2.362			
324.75	WB		O	P33	2.462	2.472	2.496	2.370	2.379	2.323	
326.21	WB		O	P35-core #5	2.410	2.434	2.419	2.396	2.396		

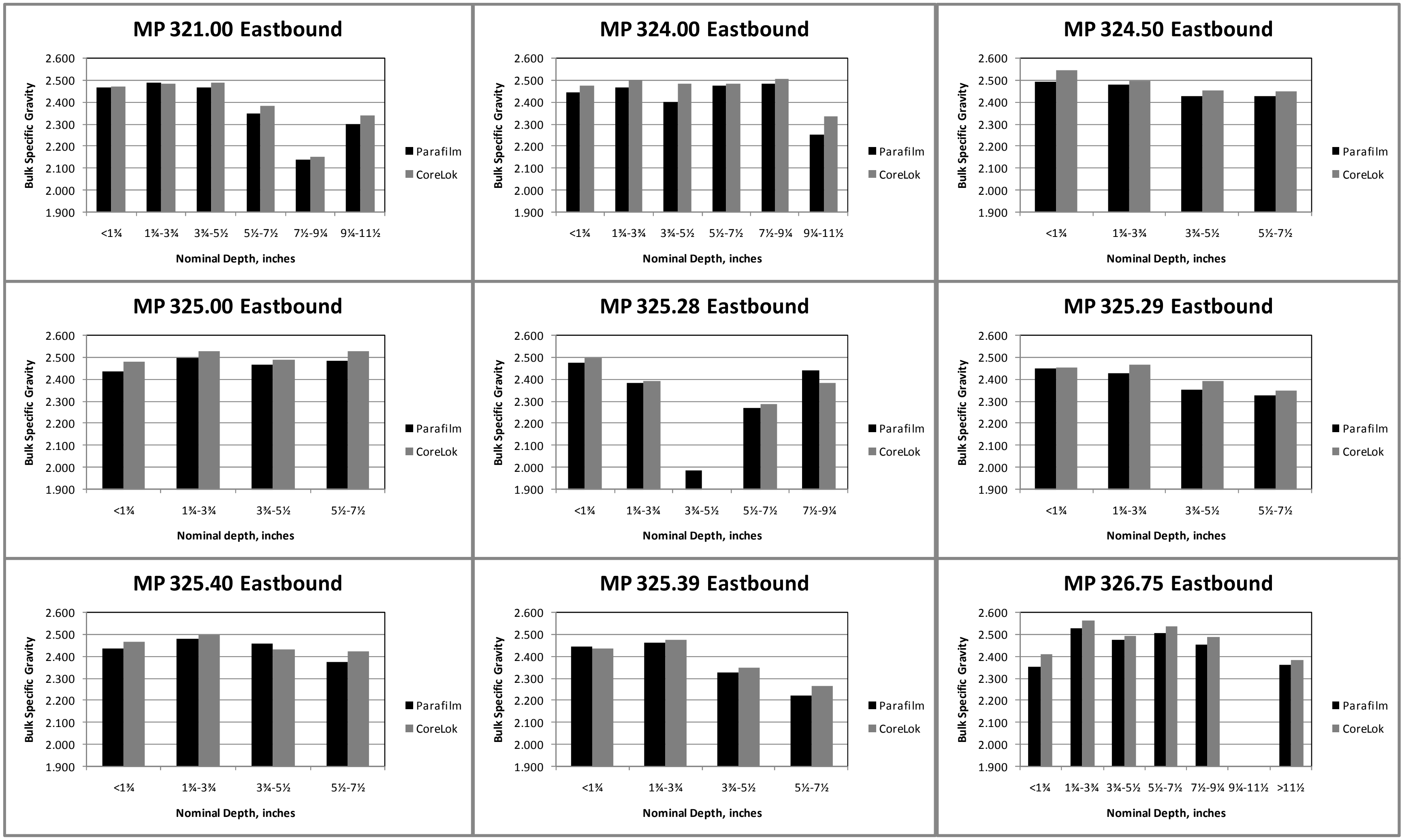


Figure 4.1: Comparison of Bulk Specific Gravity Test Results (Parafilm vs. CoreLok Methods) for the Cores Obtained in the Eastbound Direction of the Pleasant Valley – Durkee Project.

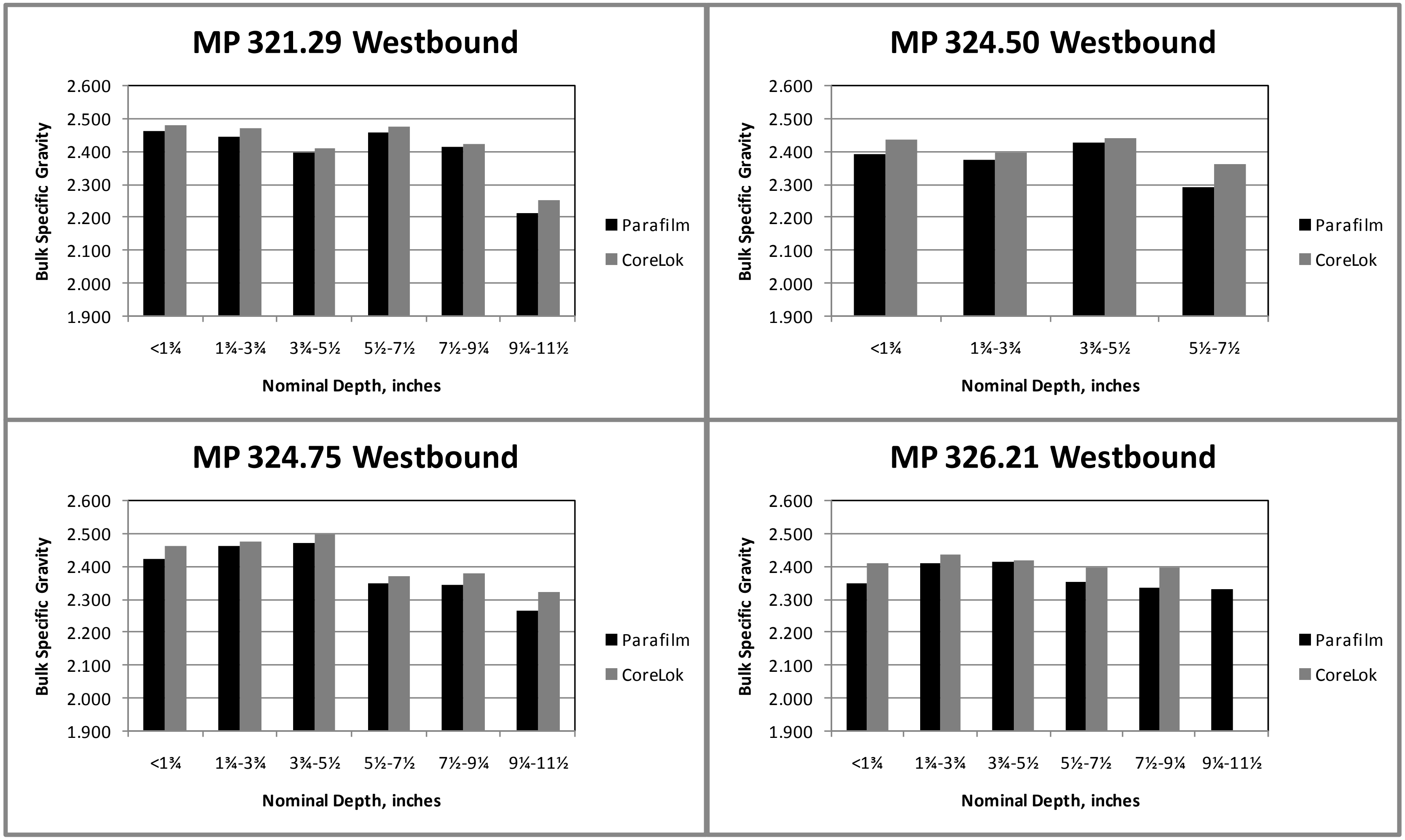


Figure 4.2: Comparison of Bulk Specific Gravity Test Results (Parafilm vs. CoreLok Methods) for the Cores Obtained in the Westbound Direction of the Pleasant Valley – Durkee Project.

Table 4.3: Water Permeability Test Results for Several of the Cores Obtained from the Pleasant Valley – Durkee Project.

Mile Point	Direction	Lane	Wheel Path	Core No.	Average Water Permeability (cm/sec) at Nominal Depth (inches) of:				
					<1¾	1¾ - 3¾	3¾ - 5½	5½ - 7½	7½ - 9¼
321.00	EB	R	O	P34	ND*	ND			
324.00	EB	R	O	P6	ND	ND	5.40E-06	ND	ND
324.50	EB		O	P27	ND	ND			
325.00	EB	R	O	P37-core #32	2.76E-07	2.27E-07	4.89E-06	4.08E-07	ND
325.28	EB	R	O	P1	ND	8.56E-05			
325.29	EB		O	P4	ND	2.65E-06			
325.39	EB	R	O	P9	ND	2.53E-06			
325.40	EB		O	P13	ND	ND			
326.75	EB	R	O	P2 - core #42	6.28E-05	8.03E-07	5.20E-04	2.07E-07	9.21E-05
321.29	WB		O	P15	ND	1.06E-04			
324.50	WB	L	I	P16-core #23	ND	3.67E-05	4.56E-06	2.37E-03	
324.75	WB		O	P33					
326.21	WB		O	P35-core #5					

*ND = not detected (i.e., no flow of water through specimen)

4.2.2 Cottage Grove – Martin Creek, Interstate 5 MP 169.3 – 174.7

Due to the inability to collect cores from this project for this study, laboratory investigations were not conducted on cores from this project.

4.2.3 Anlauf – Elkhead Road, Interstate 5 MP 154.5 – 162.1

Cores obtained from the Anlauf – Elkhead Road project in 1995 (prior to the rehabilitation activity) were cut into slices and tested for bulk specific gravity. Unfortunately, due to insufficient time and resources, the cores obtained following the rehabilitation activity were not tested. The following paragraphs summarize the results of the tests on cores obtained in 1995.

Bulk Specific Gravity – Parafilm Method

The cores were tested for bulk specific gravity following the procedure described in AASHTO T 275 with the important exception that parafilm, not paraffin, was used to coat the specimens. Table 4.4 summarizes the results from six cores obtained from the northbound direction and nine cores obtained from the southbound direction.

Bulk Specific Gravity – CoreLok Method

Cores were also tested for bulk specific gravity in accordance with ASTM D 6752 using the CoreLok device to vacuum seal the test specimens in the plastic bags used in the test. Table 4.5 summarizes the results from six cores obtained from the northbound direction and nine cores obtained from the southbound direction.

Comparison of Bulk Specific Gravity Measurements

Figures 4.3 and 4.4 compare the bulk specific gravity test results obtained from the method employing parafilm to those obtained from the method utilizing the CoreLok device for the cores taken in the northbound and southbound directions, respectively for a limited number of cores. In nearly all cases, the procedure utilizing the CoreLok device resulted in greater values for the bulk specific gravity. However, given the inherent variability in results obtained from the two test methods (to be discussed later for the Garden Valley – Roberts Creek and Vets Bridge – Myrtle Creek projects in Sections 4.2.4 and 4.2.5, respectively), the results suggest little or no statistical difference between results derived from the two test methods.

Table 4.4: Bulk Specific Gravity Test Results (Parafilm Method) for the Anlauf – Elkhead Road Project.

Mile Point	Direction	Lane	Wheel Path	Core No.	Bulk Specific Gravity (Parafilm Method) at Nominal Depth (inches) of:							
					<1¾	1¾ - 3¾	3¾ - 5½	5½ - 7½	7½ - 9½	9½ - 11½	11½ - 13½	>13½
156.51	NB	R	O	A93	2.063	2.365	2.235		2.325			
157.25	NB	L	I	A55		2.257	2.244	2.203	2.291	2.277		
159.00	NB	L	I	A68	2.219	2.278	2.306	2.154			2.313	2.271
159.75	NB	R	O	A53	2.273	2.177	2.306	2.314	2.201	2.172	2.157	
160.30	NB	R	O	A13	2.155	2.278	2.228	2.187	2.190	2.293	2.125	
162.05	NB	R	O	A20	2.200	2.277	2.222	2.169				
155.00	SB	L	I	A23	2.202	2.328	2.351	2.236	2.294	2.292	2.207	2.223
155.75	SB	R	O	A44	2.120	2.214	2.263	2.239				
156.05	SB	L	I	A36	2.084	2.164	2.289	2.128				
156.05	SB	R	O	A52	2.179	2.216	2.151	2.167	2.281		2.162	
156.47	SB	R	O	A26	2.227	2.232	2.227	2.280	2.123			
157.05	SB	L	O	A24	2.031	2.236	2.330	2.323	2.122			
158.25	SB	R	O	A21	2.313	2.315	2.247	2.352	2.347	2.154		
161.00	SB	L	I	A93	2.151	2.334	2.266	2.205	2.210	2.218	2.109	1.987
162.08	SB	L	I	A91	2.105	2.204	2.124	2.263	2.263			

Table 4.5: Bulk Specific Gravity Test Results (CoreLok Method) for the Anlauf – Elkhead Road Project.

Mile Point	Direction	Lane	Wheel Path	Core No.	Bulk Specific Gravity (CoreLok Method) at Nominal Depth (inches) of:							
					<1¾	1¾ - 3¾	3¾ - 5½	5½ - 7½	7½ - 9½	9½ - 11½	11½ - 13½	>13½
156.51	NB	R	O	A93								
157.25	NB	L	I	A55								
159.00	NB	L	I	A68	2.305	2.331	2.352	2.215			2.366	2.314
159.75	NB	R	O	A53	2.334	2.228	2.328	2.342	2.252	2.215	2.190	
160.30	NB	R	O	A13								
162.05	NB	R	O	A20								
155.00	SB	L	I	A23								
155.75	SB	R	O	A44								
156.05	SB	L	I	A36								
156.05	SB	R	O	A52	2.197	2.244	2.181	2.227	2.295	2.293	1.667	
156.47	SB	R	O	A26								
157.05	SB	L	O	A24								
158.25	SB	R	O	A21	2.304	2.369	2.300	2.374	2.367	2.255		
161.00	SB	L	I	A93	2.193	2.350	2.345	2.236	2.103	2.255	2.176	2.040
162.08	SB	L	I	A91								

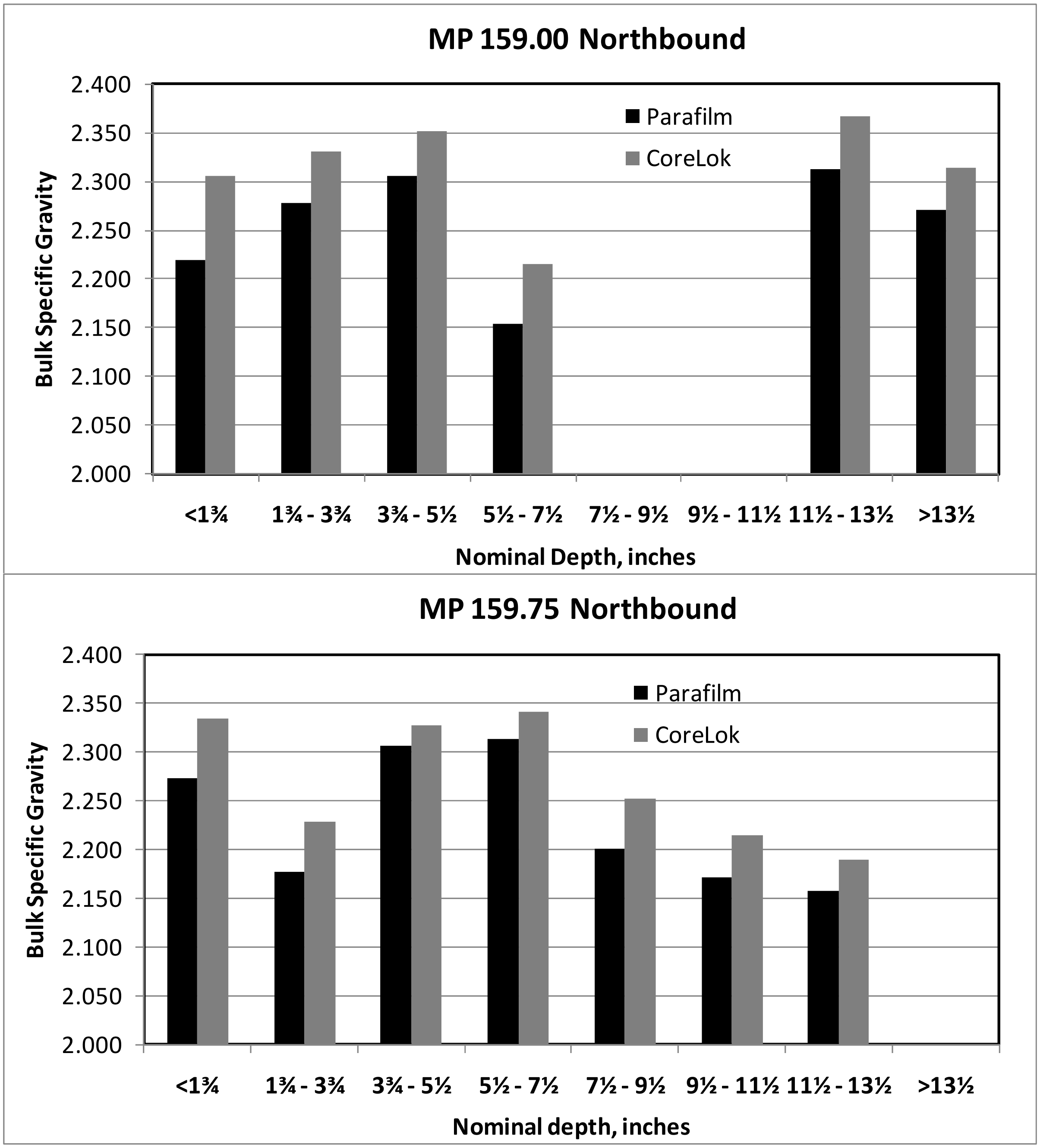


Figure 4.3: Comparison of Bulk Specific Gravity Test Results for the Cores Obtained in the Northbound Direction of the Anlauf – Elkhead Road Project.

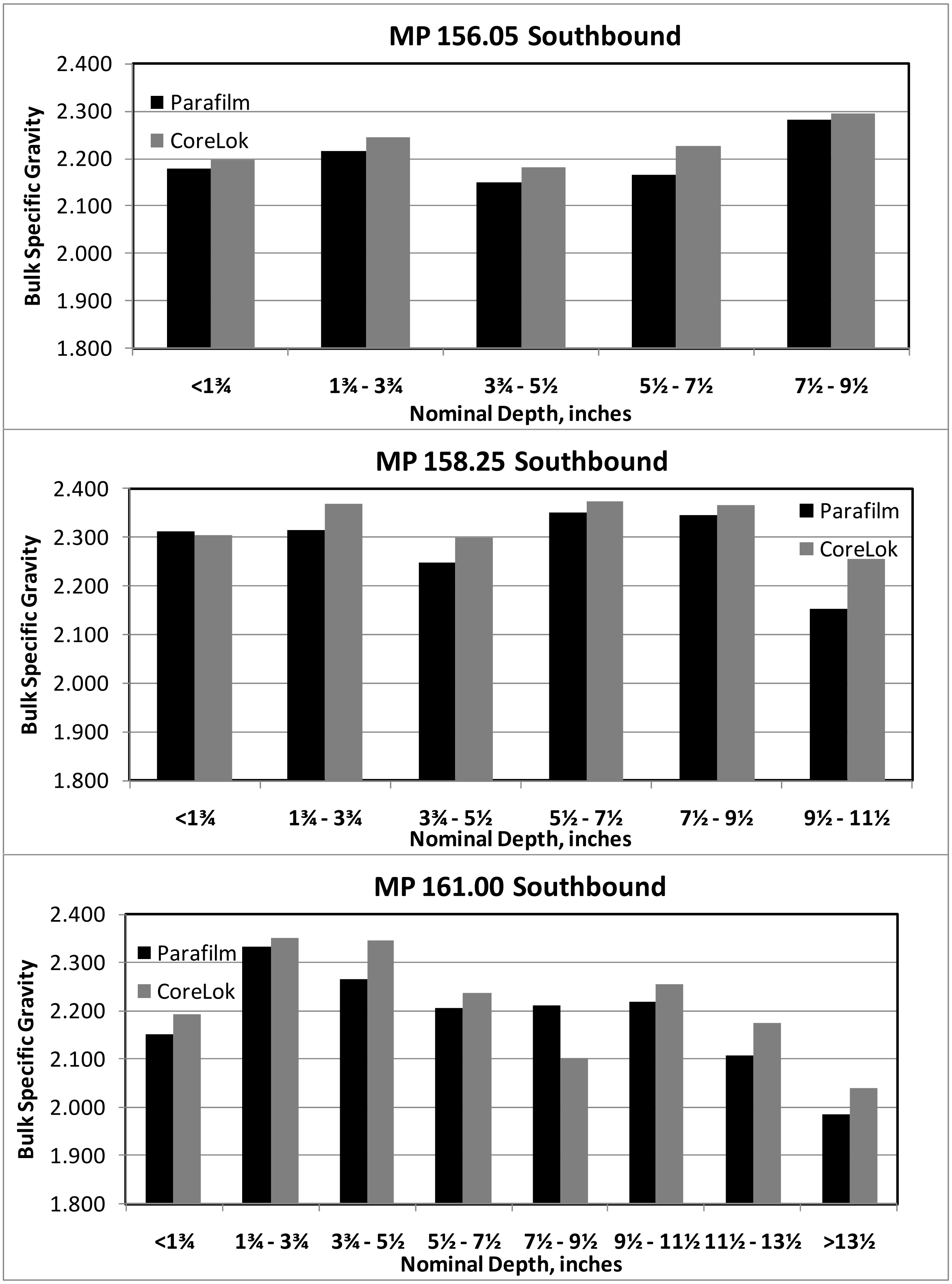


Figure 4.4: Comparison of Bulk Specific Gravity Test Results for the Cores Obtained in the Southbound Direction of the Anlauf – Elkhead Road Project.

4.2.4 Garden Valley – Roberts Creek, Interstate 5 MP 117.74 – 125.0

Several of the cores obtained from the Garden Valley – Roberts Creek project were tested for bulk specific gravity, theoretical maximum specific gravity, and permeability. The following paragraphs summarize the results of these tests.

Bulk Specific Gravity – Parafilm Method

Cores were tested for bulk specific gravity following the procedure described in AASHTO T 275 with the important exception that parafilm, not paraffin, was used to coat the specimens. Table 4.6 summarizes the results of these tests.

Table 4.6: Bulk Specific Gravity Test Results (Parafilm Method) for the Garden Valley – Roberts Creek Project.

Mile Point	Direction	Lane	Wheel Path	Core No.	Bulk Specific Gravity (Parafilm Method) at Nominal Depth (inches) of:				
					<1¾	1¾ - 3½	3½ - 6	6 - 7½	>7½
118.0	SB	R	B	7	2.278	2.281	2.415	2.403	2.332
118.0	SB	R	B	8	2.265	2.283	2.405	2.394	2.233
118.0	SB	R	B	9	2.319	2.313	2.427	2.358	2.279
118.0	SB	R	B	10		2.272	2.397	2.380	2.285
118.0	SB	R	B	12	2.286	2.256	2.403	2.384	2.302
118.0	SB	R	O	1	2.277	2.309	2.383	2.409	2.390
118.0	SB	R	O	2	2.245	2.274	2.337	2.259	2.408
118.0	SB	R	O	3	2.277	2.268	2.368	2.347	2.388
118.0	SB	R	O	4	2.257	2.301	2.410	2.409	2.152
118.0	SB	R	O	3	2.247	2.312	2.310	2.280	2.372
118.6	SB	R	B	2	2.288	2.313	2.411	2.221	
118.6	SB	R	B	6	2.283	2.310	2.421	2.383	2.218
118.6	SB	R	B	8	2.320	2.302	2.427	2.293	2.148
118.6	SB	R	B	10	2.358	2.305	2.419	2.362	2.229
118.6	SB	R	B	12	2.308	2.293	2.411	2.279	2.254
118.6	SB	R	O	1	2.256	2.354	2.412	2.395	2.330
118.6	SB	R	O	3	2.267	2.322	2.414	2.371	2.185
118.6	SB	R	O	9	2.225	2.320	2.406	2.343	2.137

Bulk Specific Gravity – CoreLok Method

Cores were also tested for bulk specific gravity in accordance with ASTM D 6752 using the CoreLok device to vacuum seal the test specimens in the plastic bags used in the test. Table 4.7 summarizes the results of these tests.

Table 4.7: Bulk Specific Gravity Test Results (CoreLok Method) for the Garden Valley – Roberts Creek Project.

Mile Point	Direction	Lane	Wheel Path	Core No.	Bulk Specific Gravity (CoreLok Method) at Nominal Depth (inches) of:				
					<1¾	1¾ - 3½	3½ - 6	6 - 7½	>7½
118.0	SB	R	B	7	2.296	2.295	2.516	2.407	
118.0	SB	R	B	8	2.318	2.309	2.426	2.421	
118.0	SB	R	B	9	2.367	2.339	2.446	2.414	
118.0	SB	R	B	10		2.299	2.431	2.402	2.311
118.0	SB	R	B	12	2.347	2.294	2.402		
118.0	SB	R	O	1	2.318	2.335	2.406	2.423	
118.0	SB	R	O	2	2.307	2.311		2.360	2.427
118.0	SB	R	O	3	2.306	2.298	2.414	2.415	2.367
118.0	SB	R	O	4	2.295	2.331	2.434	2.428	2.191
118.0	SB	R	O	3	2.303	2.321	2.387	2.397	2.394
118.6	SB	R	B	2	2.389	2.377	2.442		
118.6	SB	R	B	6	2.346	2.332	2.444	2.396	2.274
118.6	SB	R	B	8	2.359	2.324	2.440	2.316	
118.6	SB	R	B	10	2.403	2.329	2.447	2.378	
118.6	SB	R	B	12	2.355	2.362	2.460	2.331	
118.6	SB	R	O	1	2.323	2.369	2.437	2.410	2.348
118.6	SB	R	O	3	2.334	2.339	2.429	2.397	
118.6	SB	R	O	9	2.308	2.341	2.390	2.359	

Comparison of Bulk Specific Gravity Measurements

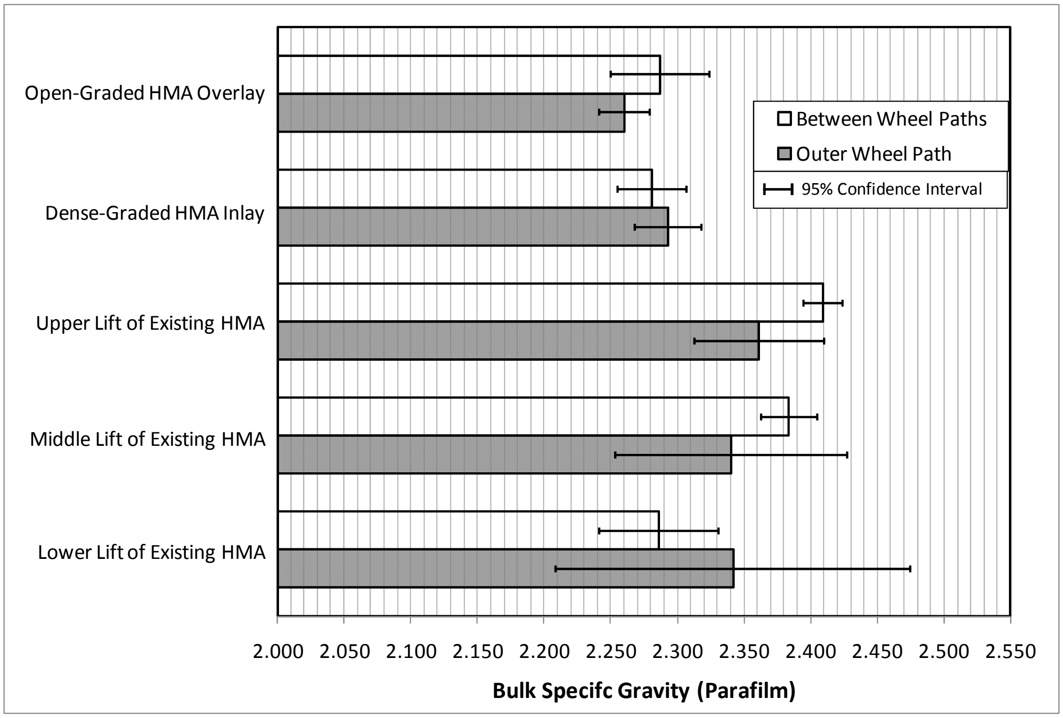
Figures 4.5 through 4.8 compare the results of bulk specific gravity tests conducted on the slices (lifts) of the cores obtained from MP 118.0 and MP 118.6. In all cases, the large bars indicate the magnitude of the test results and the shorter, thinner bars indicate the 95% confidence interval of the test results. Overlap of the 95% confidence intervals of any two test results indicates that the two results are not statistically significantly different at a 95% confidence (5% significance) level.

Figure 4.5 compares the results of bulk specific gravity tests conducted on the cores obtained between wheel paths to those of cores obtained in the outside wheel path from MP 118.0 for both methods of measurement. In all cases for a given lift, there exists sufficient evidence at a 95% confidence level to indicate that there is not a significant difference between results from cores obtained between wheel paths and those obtained from the outside wheel path.

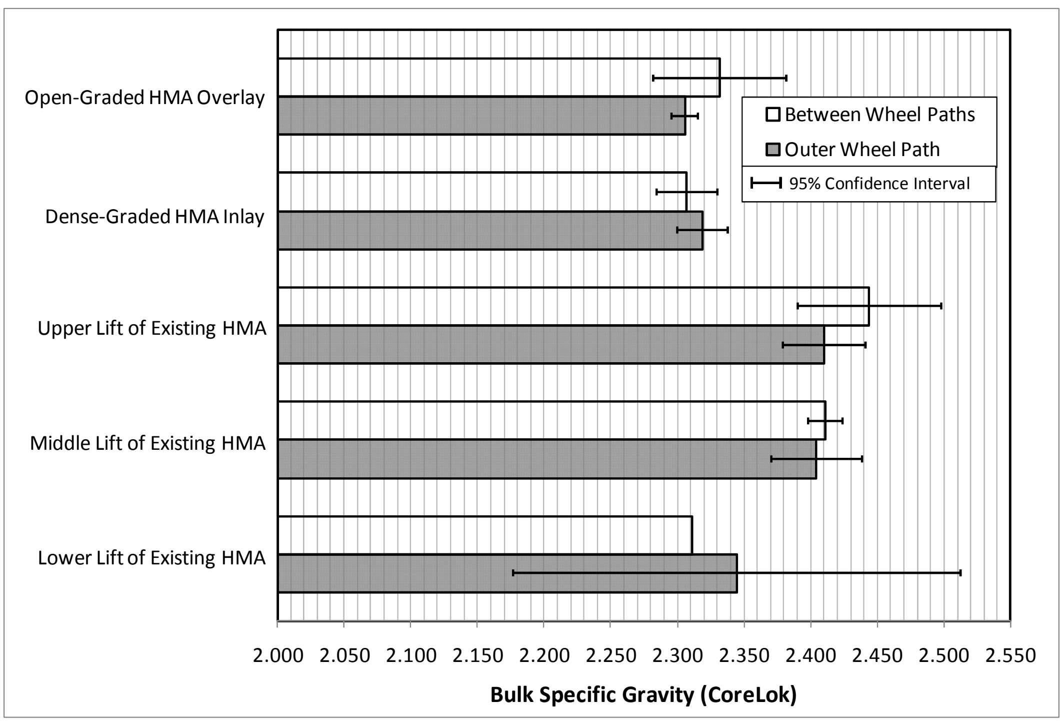
Contrary to expectations, Figure 4.5 also indicates that there is not a significant difference between the densities of the open-graded HMA overlay and the underlying dense-graded HMA inlay. However, there is some evidence to indicate a significant difference in densities between the dense-graded HMA inlay and the top two lifts of the underlying (existing) dense-graded HMA pavement.

Figure 4.6 compares the bulk specific gravity test results based on method of measurement for the cores obtained from between wheel paths and those obtained from the outer wheel path from MP 118.0. In all but one case for a given lift, there exists sufficient evidence at a 95% confidence level to indicate that there is not a significant difference between results based on method of measurement. The results shown in Figure 4.6 also reinforce the inferences made previously regarding differences in densities between the dense-graded HMA inlay and overlying open-graded HMA overlay as well as differences in densities between the dense-graded HMA inlay and underlying (existing) dense-graded HMA pavement.

Figures 4.7 and 4.8 show comparisons of densities of the cores obtained from MP 118.6. Inferences nearly identical to those drawn from the results from cores obtained from MP 118.0 can be drawn from these results.



a) AASHTO T 275 Using Parafilm



b) ASTM D 6752 Using the CoreLok Device

Figure 4.5: Comparison of Bulk Specific Gravity Test Results for Cores Obtained Between the Wheel Paths and from Within the Outer Wheel Path at MP 118.0.

Theoretical Maximum Specific Gravity

Several of the cores were tested for theoretical maximum specific gravity in accordance with AASHTO T 209. Table 4.8 summarizes the results of these tests. Calculation of the average, standard deviation, and coefficient of variation assumes that each slice represented the same mix design.

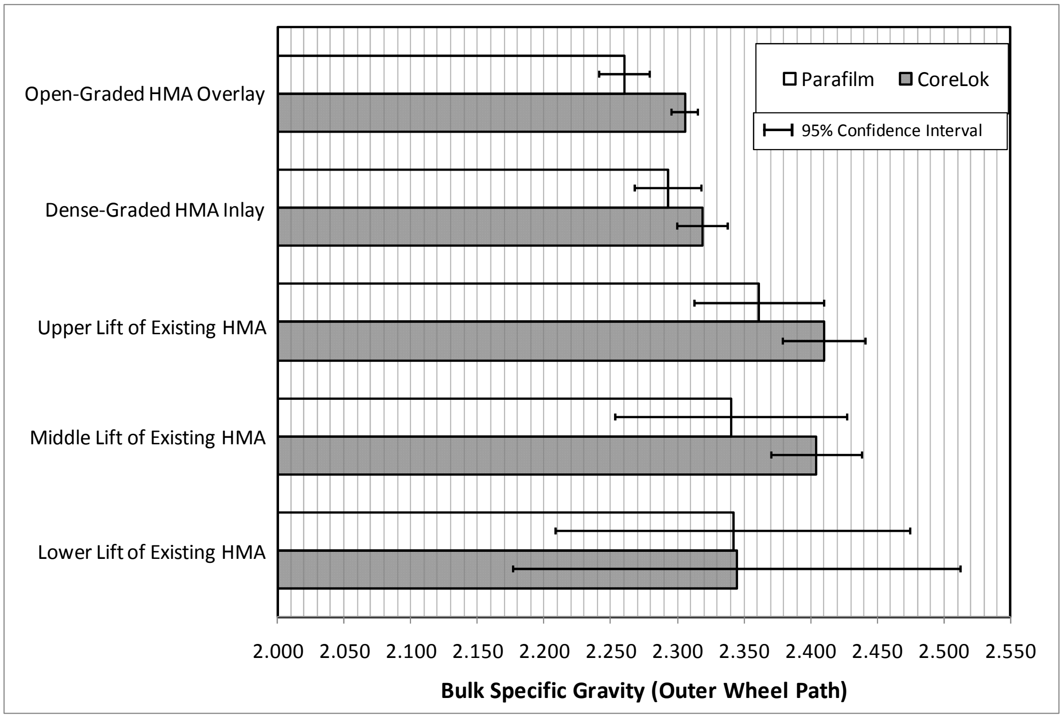
Air Void Content

Air void contents of the core slices were determined from the bulk specific gravity and theoretical maximum specific gravity test results in accordance with AASHTO T 269. Figure 4.9 shows the air void contents for the cores obtained from both coring locations. As indicated, the dense-graded HMA inlay had similar or slightly lower air void contents than the overlying open-graded HMA overlay and higher air void contents than the top lift of the underlying (existing) dense-graded HMA pavement. The results also indicate that the air void contents of the three lifts of existing dense-graded HMA pavement increased with depth.

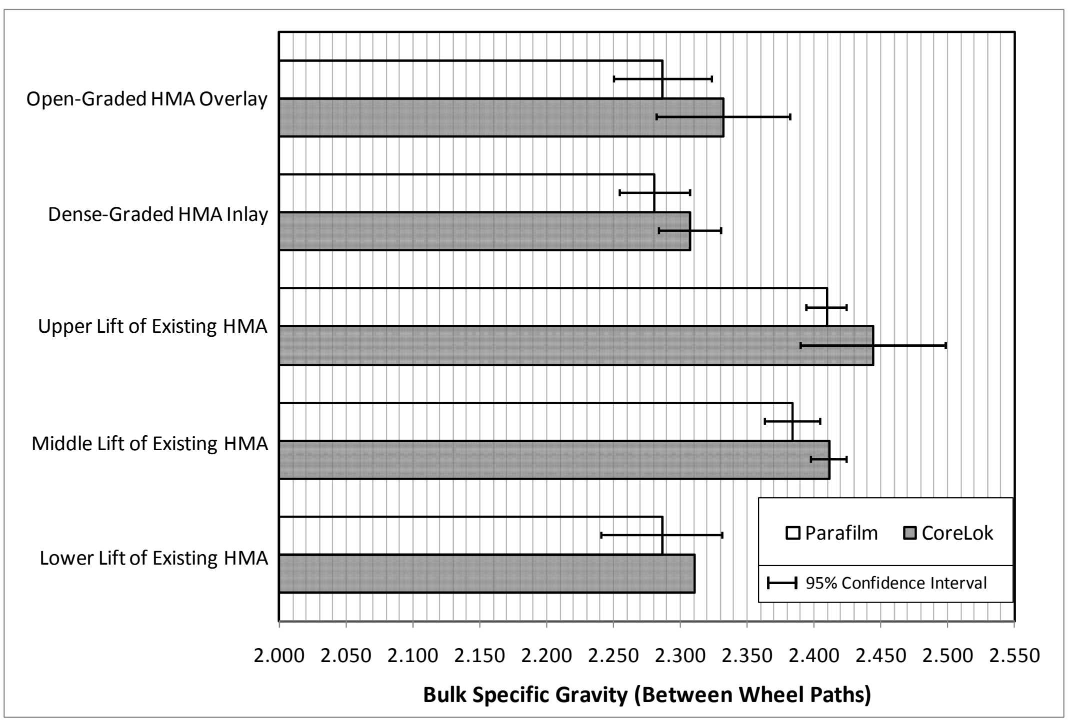
Permeability

Several of the core slices were tested for permeability in accordance with Florida DOT test method FM 5-565 as shown in Table 4.9. The results shown in Figure 4.10 indicate that the permeability of the core slices from the dense-graded HMA inlay were of similar magnitude to the core slices from the overlying open-graded HMA overlay and higher than the top lift of the underlying (existing) dense-graded HMA pavement.

It is also evident from the results that the permeability of the existing dense-graded pavement increased with depth. At both core locations the permeability of the lowest portion of the existing dense-graded HMA pavement was of similar magnitude to, or slightly higher than, the open-graded HMA overlay layer.

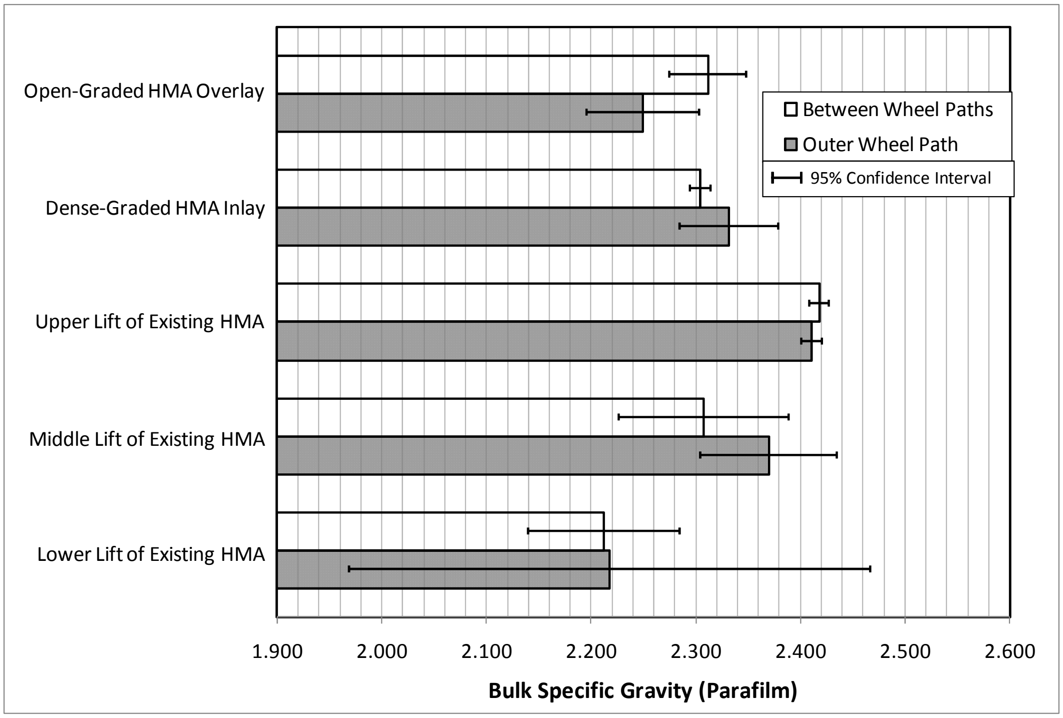


a) Outer Wheel Path

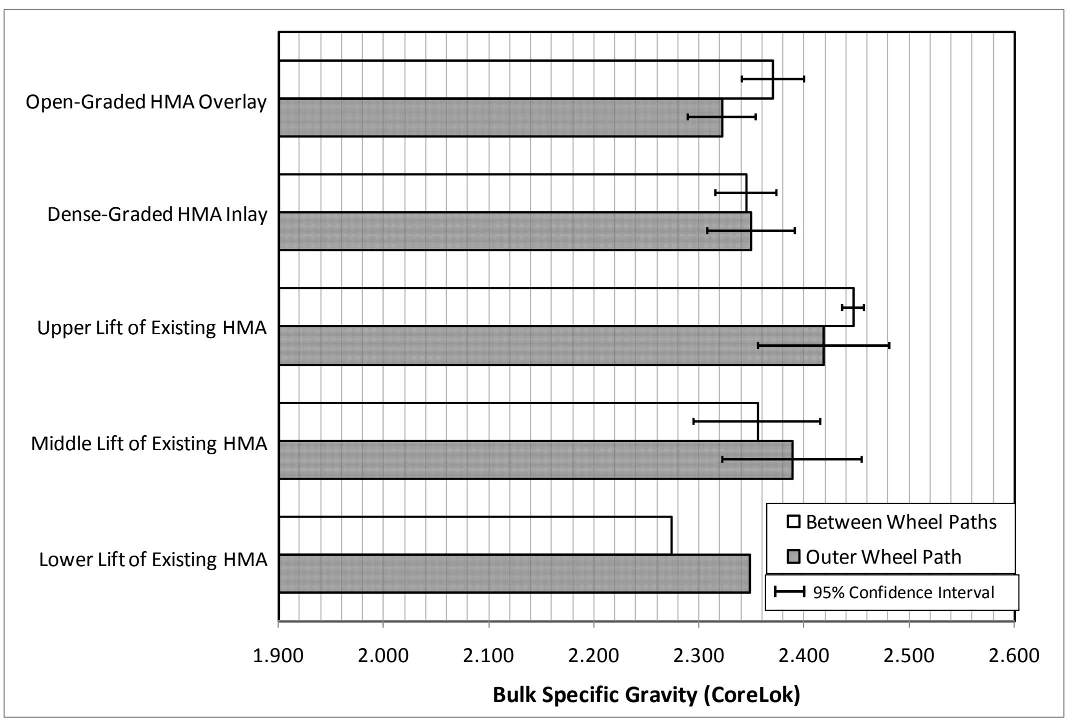


b) Between Wheel Paths

Figure 4.6: Comparison of Bulk Specific Gravity Test Methods for Cores Obtained from MP 118.0.

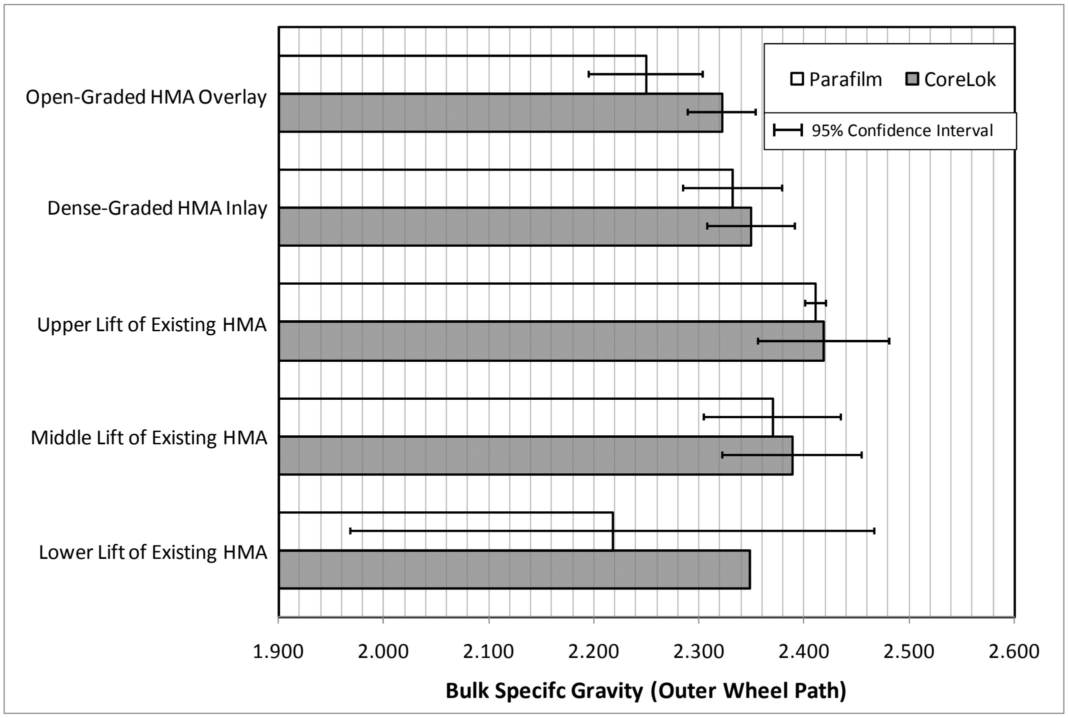


a) AASHTO T 275 Using Parafilm

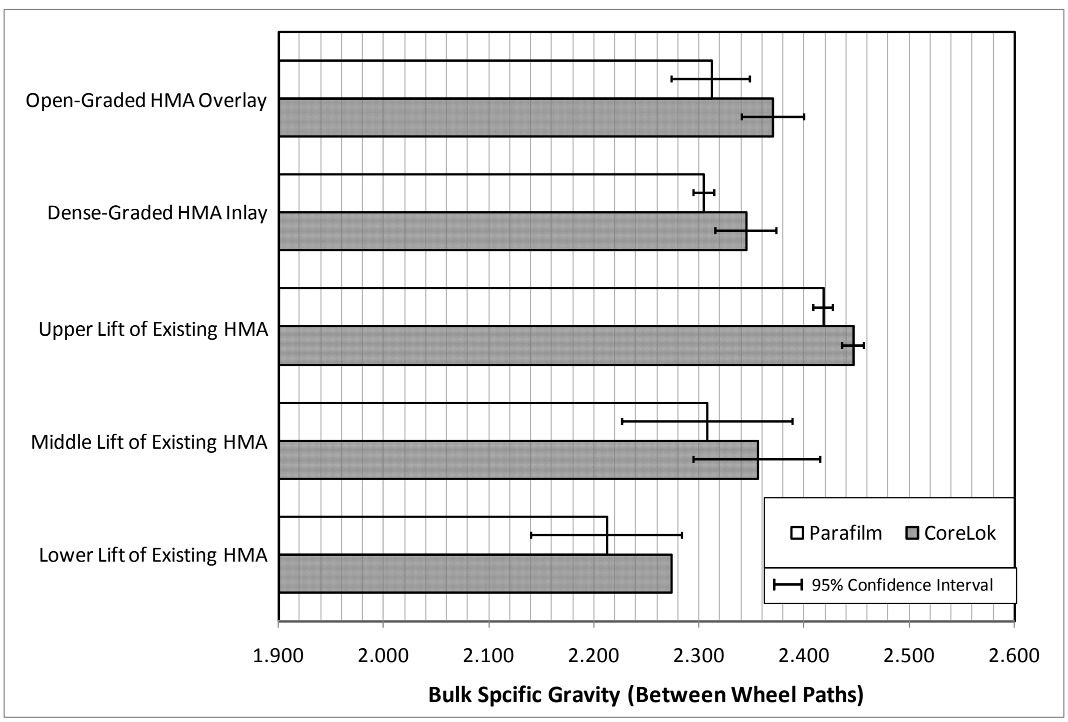


b) ASTM D 6752 Using the CoreLok Device

Figure 4.7: Comparison of Bulk Specific Gravity Test Results for Cores Obtained Between the Wheel Paths and from Within the Outer Wheel Path at MP 118.6.



a) Outer Wheel Path

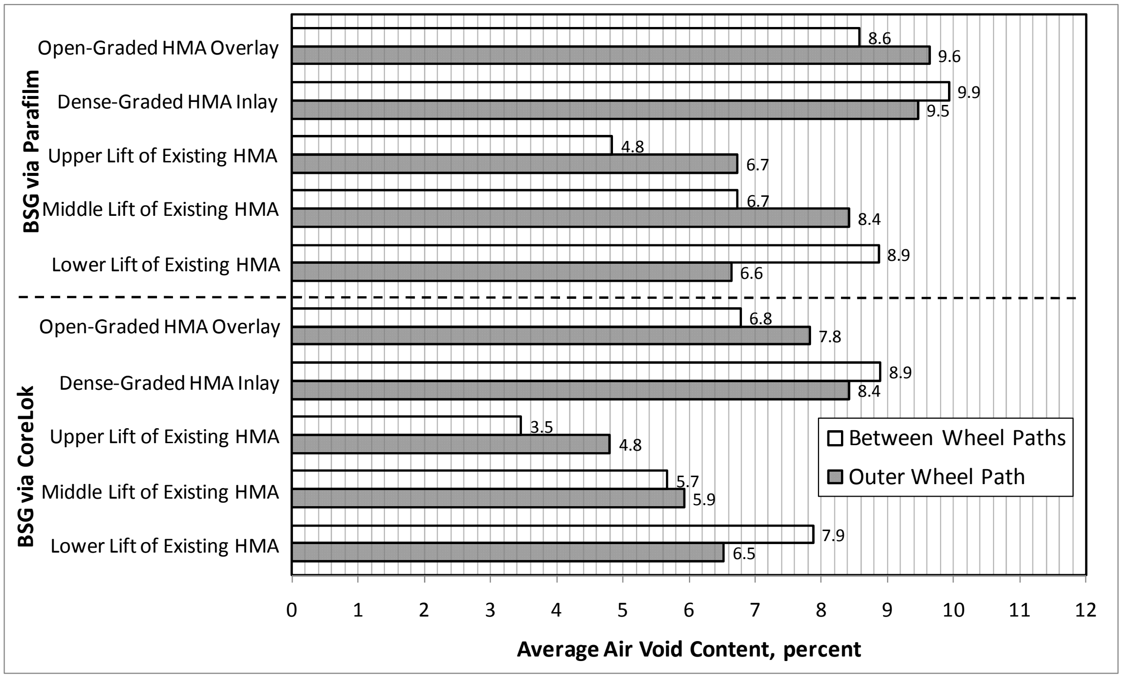


b) Between Wheel Paths

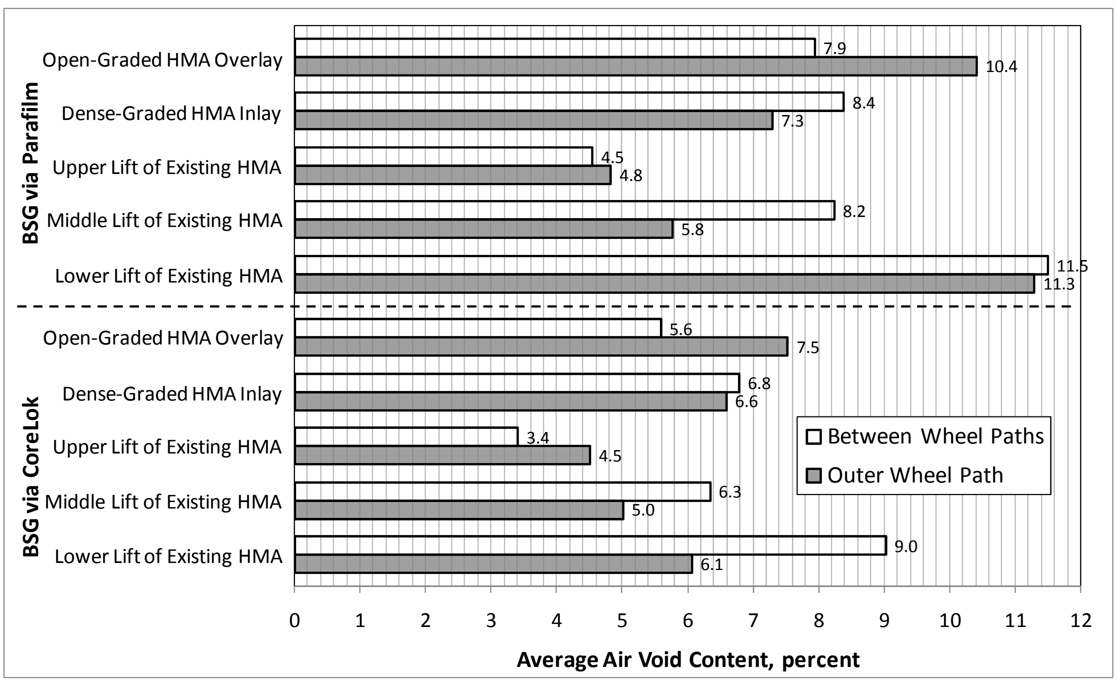
Figure 4.8: Comparison of Bulk Specific Gravity Test Methods for Cores Obtained from MP 118.6.

Table 4.8: Theoretical Maximum Specific Gravity Test Results for the Garden Valley – Roberts Creek Project.

Mile Point	Direction	Lane	Wheel Path	Core No.	Theoretical Maximum Specific Gravity of:				
					<1¾	1¾ - 3½	3½ - 6	6 - 7½	>7½
118.0	SB	R	B	9	2.490	2.504	2.535	2.555	2.472
118.0	SB	R	B	12	2.513	2.558	2.521		
118.0	SB	R	O	4		2.535	2.540	2.557	2.545
118.6	SB	R	B	8	2.519	2.525	2.544		
118.6	SB	R	B	10	2.528	2.507	2.527		2.493
118.6	SB	R	B	12	2.486	2.512	2.530	2.548	2.476
118.6	SB	R	O	9	2.509	2.517	2.530	2.481	2.529
Average					2.508	2.523	2.532	2.535	2.503
Standard Deviation					0.016	0.019	0.008	0.036	0.033
Coefficient of Variation, %					0.66	0.75	0.31	1.4	1.3



a) MP 118.0



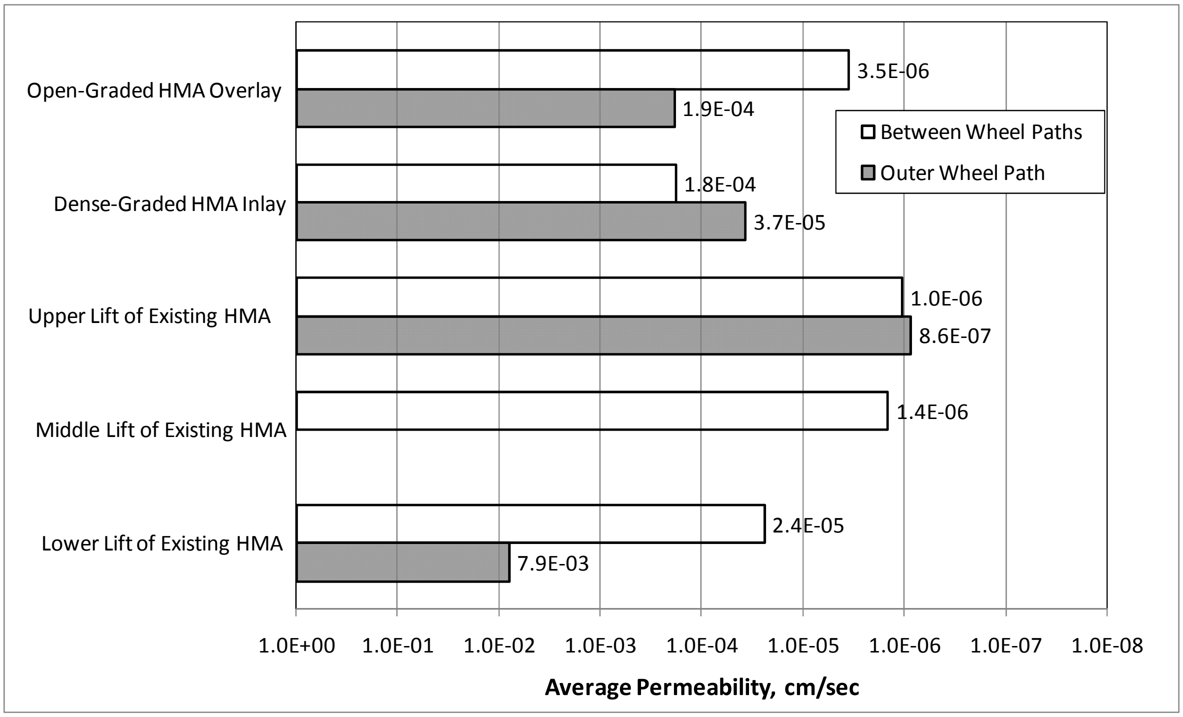
b) MP 118.6

Figure 4.9: Air Void Content of Cores from the Garden Valley – Roberts Creek Project.

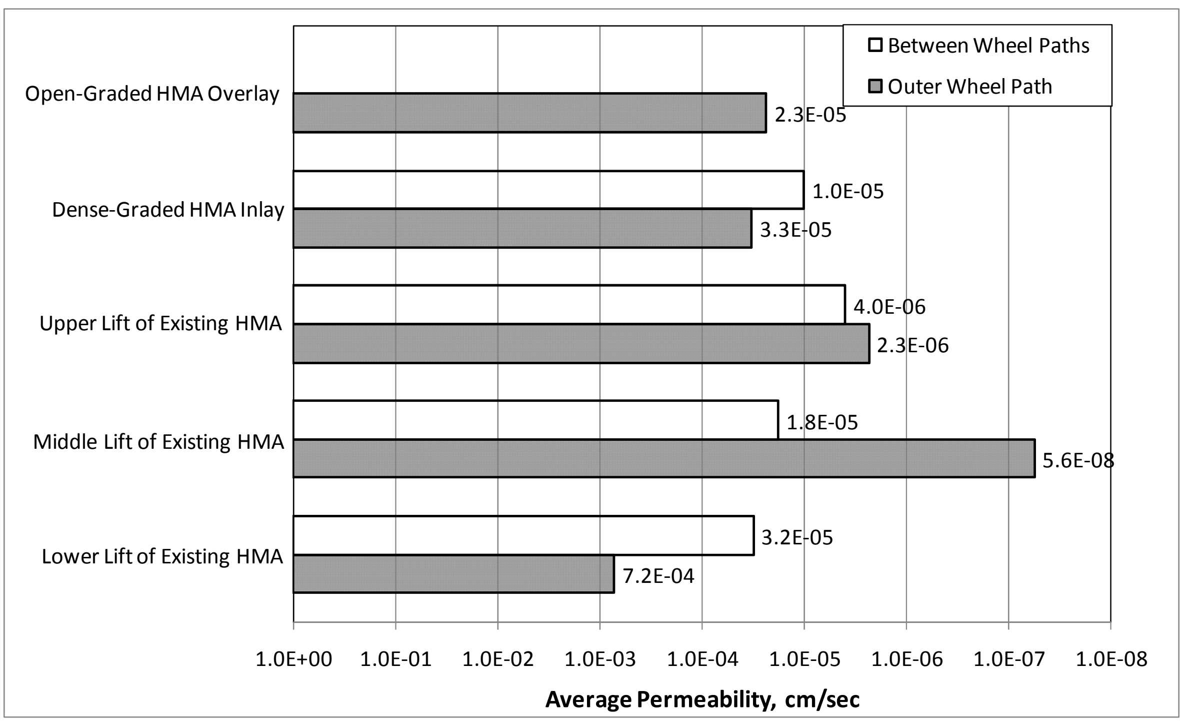
Table 4.9: Average Water Permeability of Core Slices for the Garden Valley – Roberts Creek Project.

Mile Point	Direction	Lane	Wheel Path	Core No.	Average Water Permeability (cm/sec) at Nominal Depth (inches) of:				
					<1¾	1¾ - 3½	3½ - 6	6 - 7½	>7½
118.0	SB	R	B	8	ND*	ND	ND	ND	ND
118.0	SB	R	B	9			ND	4.33E-06	3.10E-05
118.0	SB	R	B	12	7.03E-06	3.60E-04	3.15E-06	ND	4.11E-05
118.0	SB	R	O	4	1.86E-04	3.68E-05	8.59E-07	ND	7.92E-03
118.6	SB	R	B	8			2.88E-06	1.13E-05	
118.6	SB	R	B	12		1.01E-05	5.04E-06	2.46E-05	3.16E-05
118.6	SB	R	O	9	2.35E-05	3.26E-05	2.35E-06	5.63E-08	7.20E-04

*ND = not detected (i.e., no water flow through the specimen)



a) MP 118.0



b) MP 118.6

Figure 4.10: Permeability of Cores from the Garden Valley – Roberts Creek Project.

4.2.5 Vets Bridge – Myrtle Creek, Interstate 5 MP 109.0 – 112.5

Several of the cores obtained from the Vets Bridge – Myrtle Creek project were tested for bulk specific gravity, theoretical maximum specific gravity, and permeability. The following paragraphs summarize the results of these tests.

Bulk Specific Gravity – Parafilm Method

Cores were tested for bulk specific gravity following the procedure described in AASHTO T 275 with the important exception that parafilm, not paraffin, was used to coat the specimens. Table 4.10 summarizes the results of these tests.

Table 4.10: Bulk Specific Gravity Test Results (Parafilm Method) for the Vets Bridge – Myrtle Creek Project.

Mile Point	Direction	Lane	Wheel Path	Core No.	Bulk Specific Gravity (Parafilm Method) at Nominal Depth (inches) of:		
					<2.7	2.7 - 5.3	> 5.3
109.90	SB	R	B	8	2.146	2.303	2.389
109.90	SB	R	B	9	2.251	2.333	2.399
109.90	SB	R	B	10	2.260	2.328	2.392
109.90	SB	R	B	11	2.250	2.336	2.411
109.90	SB	R	B	12	2.287	2.343	2.394
109.90	SB	R	O	1	2.178	2.347	2.420
109.90	SB	R	O	2	2.170	2.336	
109.90	SB	R	O	3	2.210	2.348	2.378
109.90	SB	R	O	5	2.209	2.340	2.358
109.90	SB	R	O	6	2.282	2.332	2.369

Bulk Specific Gravity – CoreLok Method

Cores were also tested for bulk specific gravity in accordance with ASTM D 6752 using the CoreLok device to vacuum seal the test specimens in the plastic bags used in the test. Table 4.11 summarizes the results of these tests.

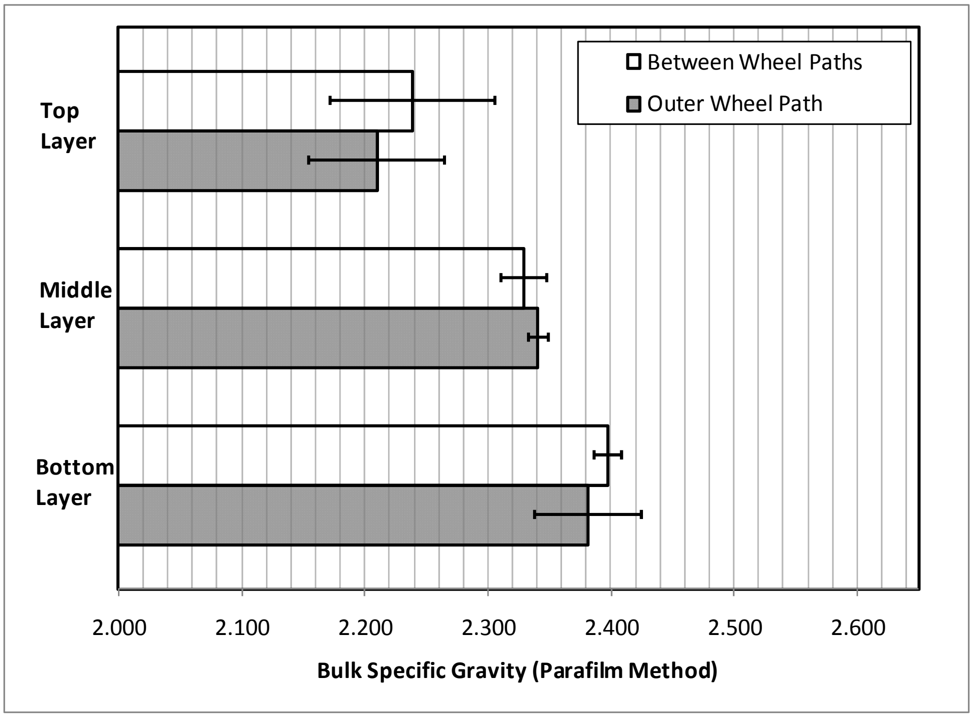
Table 4.11: Bulk Specific Gravity Test Results (CoreLok Method) for the Vets Bridge – Myrtle Creek Project.

Mile Point	Direction	Lane	Wheel Path	Core No.	Bulk Specific Gravity (CoreLok Method) at Nominal Depth (inches) of:		
					<2.7	2.7 - 5.3	> 5.3
109.90	SB	R	B	8	2.214	2.323	2.425
109.90	SB	R	B	9	2.288	2.350	2.427
109.90	SB	R	B	10	2.293	2.354	2.397
109.90	SB	R	B	11	2.277	2.356	2.421
109.90	SB	R	B	12	2.320	2.377	2.466
109.90	SB	R	O	1	2.239	2.410	2.417
109.90	SB	R	O	2	2.031	2.366	
109.90	SB	R	O	3	2.238	2.371	
109.90	SB	R	O	5	2.257	2.409	2.516
109.90	SB	R	O	6	2.300	2.396	2.391

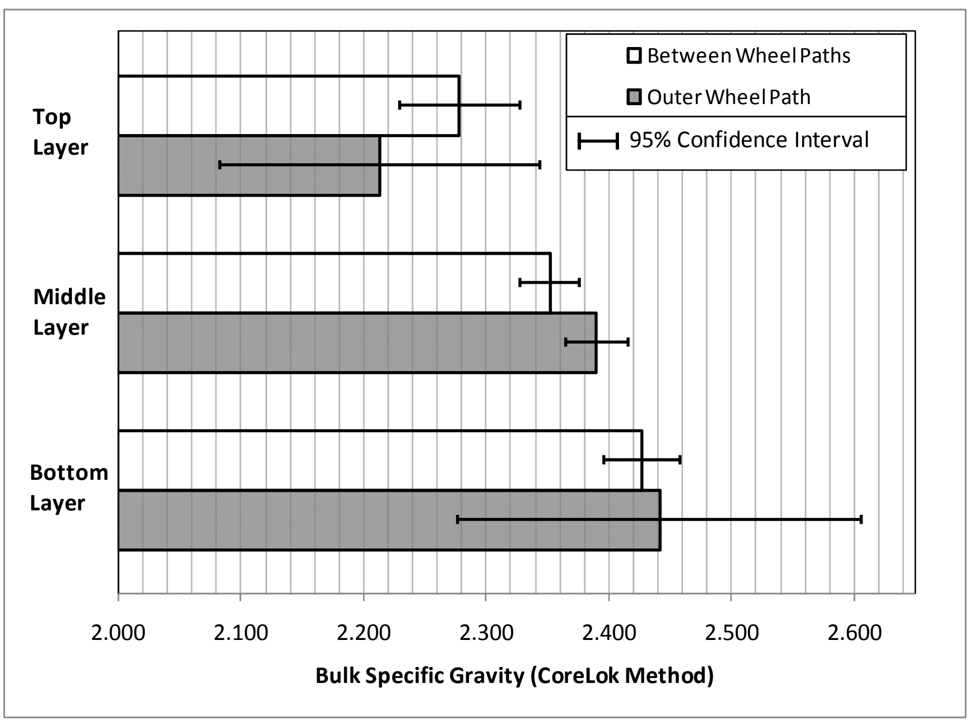
Comparison of Bulk Specific Gravity Measurements

Figures 4.11 and 4.12 compare the results of bulk specific gravity tests conducted on the slices (lifts) of the cores obtained from MP 109.9. In all cases, the large bars indicate the magnitude of the test results and the shorter, thinner bars indicate the 95% confidence interval of the test results. Overlap of the 95% confidence intervals of any two test results indicates that the two results are not statistically significantly different at a 95% confidence (5% significance) level.

Figure 4.11 compares the results of bulk specific gravity tests conducted on the cores obtained between wheel paths to those of cores obtained in the outside wheel path from MP 109.9 for both methods of measurement. In all cases for a given lift, there exists sufficient evidence at a 95% confidence level to indicate that there is not a significant difference between results from cores obtained between wheel paths and those obtained from the outside wheel path.



a) AASHTO T 275 (Parafilm)



b) ASTM D 6752 (CoreLok)

Figure 4.11: Comparison of Bulk Specific Gravity Test Results for Cores Obtained Between the Wheel Paths and from Within the Outer Wheel Path at MP 109.9.

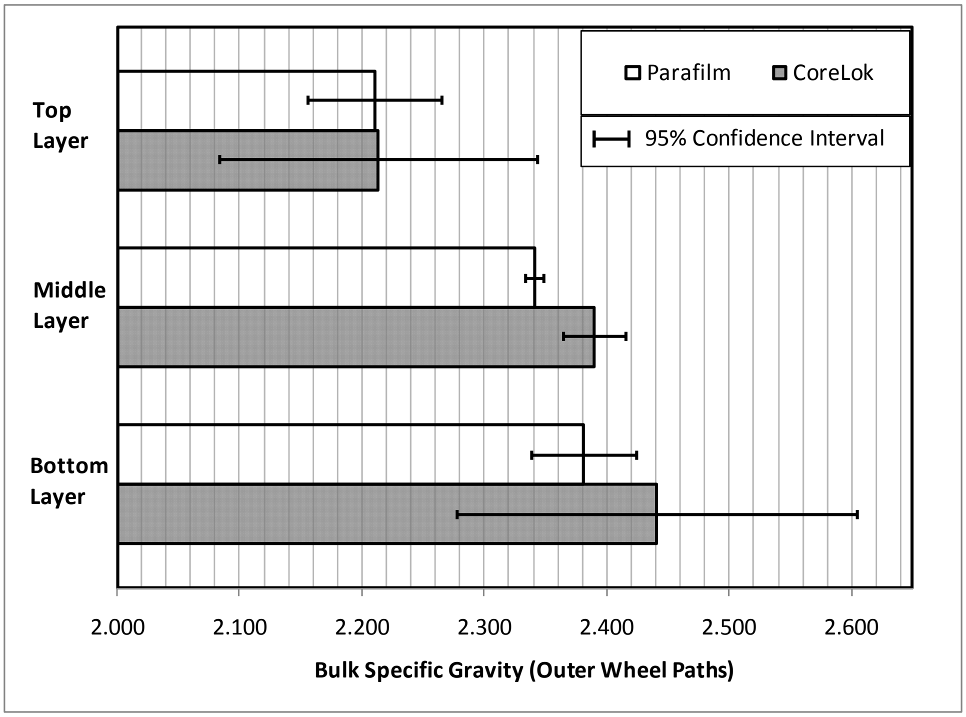
Figure 4.12 compares the bulk specific gravity test results based on method of measurement for the cores obtained from between wheel paths and those obtained from the outer wheel path from MP 109.0. In all but one case for a given lift, there exists sufficient evidence, at a 95% confidence level, to indicate that there is not a significant difference between results based on method of measurement.

Theoretical Maximum Specific Gravity

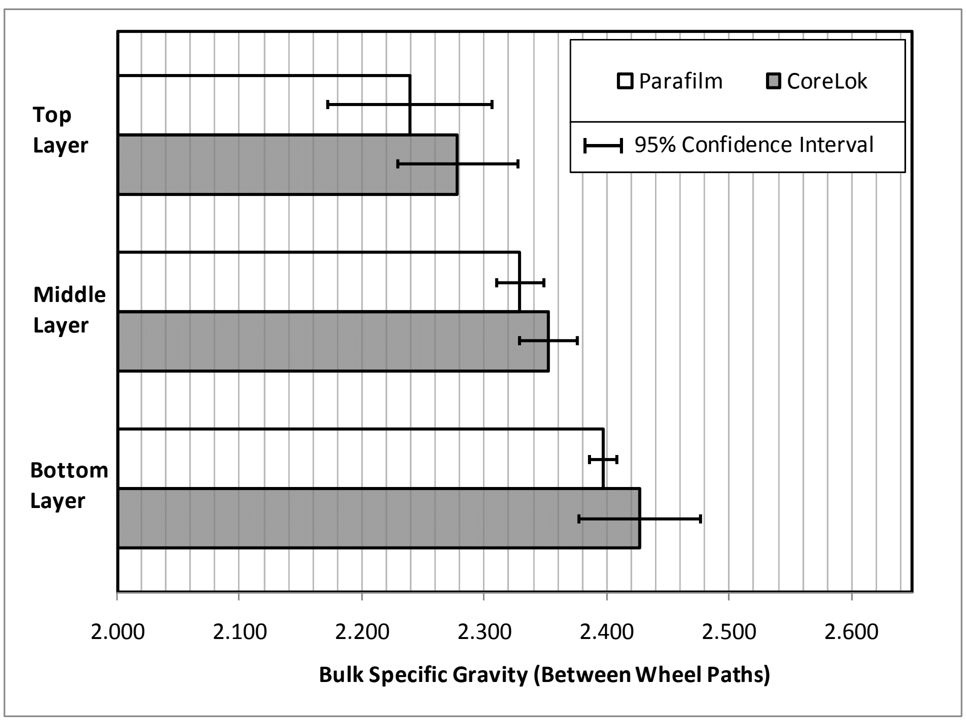
Several of the cores were tested for theoretical maximum specific gravity in accordance with AASHTO T 209. Table 4.12 summarizes the results of these tests. Calculation of the average, standard deviation, and coefficient of variation assumes that each slice represented the same mix design.

Air Void Content

Air void contents of the core slices were determined from the bulk specific gravity and theoretical maximum specific gravity test results in accordance with AASHTO T 269. Figure 4.13 summarizes the results derived from both methods of bulk specific gravity (BSG) measurement. It indicates that the open-graded HMA overlay layer had the highest air void content (although lower than expected), and air void content decreased with depth in the existing HMA layer. Comparing results derived from cores obtained in the wheel path to those of cores obtained between wheel paths, it can be seen that there is greater than 2% difference between air void contents for the open-graded HMA overlay layer and top lift of the existing dense-graded HMA layer, but essentially no difference for the bottom lift of the existing dense-graded HMA layer.



a) Outer Wheel Path



b) Between Wheel Paths

Figure 4.12: Comparison of Bulk Specific Gravity Test Methods for Cores Obtained from MP 109.9.

Table 4.12: Theoretical Maximum Specific Gravity Test Results for the Vets Bridge – Myrtle Creek Project.

Mile Point	Direction	Lane	Wheel Path	Core No.	Nominal Layer Depth, inches		
					< 2.7	2.7 - 5.3	> 5.3
109.9	South-bound	Outside	Between	8	2.489	2.525	2.508
				9	2.493	2.523	2.507
				10	2.487	2.519	2.513
				11	2.524	2.521	2.518
				12	2.488	2.600	2.510
			Outer	1	2.479	2.525	2.514
				2	2.517	2.509	
				3	2.494	2.514	
				5	2.506	2.535	2.526
				6	2.524	2.526	2.513
Average					2.500	2.530	2.514
Standard Deviation					0.016	0.026	0.006
Coefficient of Variation, %					0.66	1.0	0.24

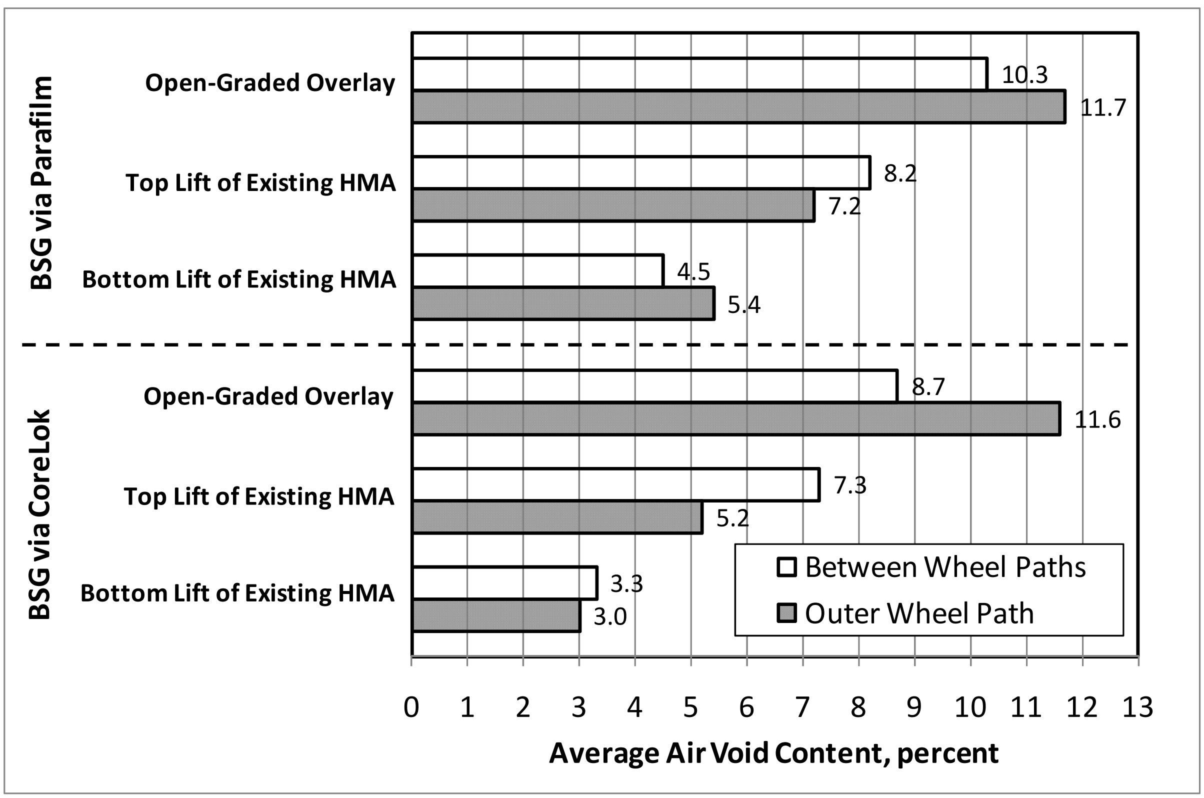


Figure 4.13: Air Void Content of Cores from the Vets Bridge – Myrtle Creek Project.

Permeability

Several of the core slices were tested for permeability in accordance with Florida DOT test method FM 5-565 as shown in Table 4.13. The results shown in Figure 4.14 indicate that the open-graded HMA overlay layer had a much higher permeability than either lift of the underlying dense-graded HMA layer, and that the bottom lift of the dense-graded HMA layer had a much lower permeability than the top lift of the dense-graded HMA layer. Comparing the results shown in Figure 4.13 with those shown in Figure 4.14, it can be seen that there appears to be a reasonable correlation between air void content and permeability of the pavement layers, with higher air void content corresponding to higher permeability.

Table 4.13: Average Water Permeability of Core Slices for the Garden Valley – Roberts Creek Project.

Mile Point	Direction	Lane	Wheel Path	Core No.	Average Water Permeability (cm/sec) at Nominal Depth (inches) of:		
					< 2.7	2.7 - 5.3	> 5.3
109.90	SB	R	B	8	4.16E-03	1.11E-04	1.22E-05
109.90	SB	R	B	9	1.36E-06	1.13E-04	
109.90	SB	R	B	10		1.27E-04	4.24E-06
109.90	SB	R	B	11	3.13E-07	6.07E-05	4.00E-06
109.90	SB	R	B	12		3.41E-04	9.68E-06
109.90	SB	R	O	1		3.59E-05	8.17E-07
109.90	SB	R	O	2	3.30E-03	3.70E-06	
109.90	SB	R	O	3	5.59E-04	5.24E-06	
109.90	SB	R	O	5		3.47E-06	1.10E-06
109.90	SB	R	O	6	4.42E-06	4.61E-07	2.73E-06

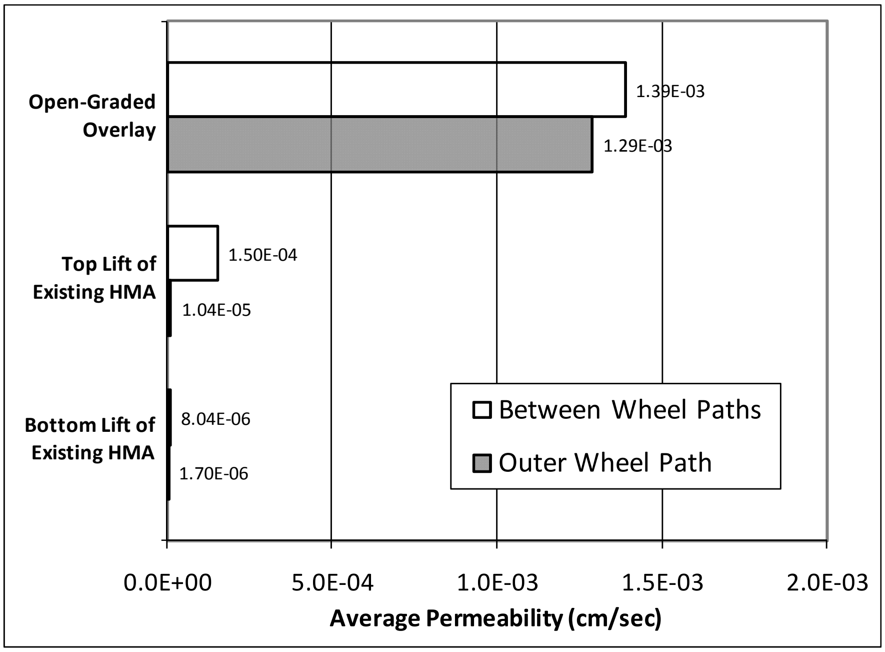


Figure 4.14: Permeability of Cores from the Vets Bridge – Myrtle Creek Project.

5.0 DISCUSSION OF RESULTS

The following discussion of results is categorized by the principal activities undertaken during the field and laboratory investigations for this study.

5.1 CORING ACTIVITIES

In general, visual examination of cores obtained from the projects provided valuable information regarding the pavement layers such as thickness of layers and lifts, delaminated layers, presence of voids, layers with excessive asphalt binder, disintegrating layers, and evidence of stripping. Unfortunately, notations made by field crew personnel of layer conditions on the core logs did not always accurately reflect actual layer conditions. For example, the broken slabs from the trenches excavated along the Pleasant Valley – Durkee project clearly indicated the presence of stripping, whereas the core logs of cores taken in the vicinity of the trenches indicated the layers to be in “Good” condition without any notations identifying layers that had stripped (possibly due to the presence of stripping being masked by the cut circumference of the cores). This suggests the need for a more thorough evaluation of the cores to definitively detect stripping, such as the simple technique proposed by Terrel and Al-Swailmi (*1994*) whereby a core is broken and compared to a set of standard photos indicating various percentages of stripping within the cores.

5.2 TRENCHING ACTIVITIES

The trenches excavated along three of the five projects provided essentially the same information derived from the cores, but also provided additional valuable information that was not readily (or, for certain observations, could not be) obtained from cores alone. For this study, the additional information included verification of observations derived from cores taken in the vicinity of the trenches (e.g., compare Figure 3.28 with Figure 3.29), subsurface profile and condition (e.g., Figure 3.7), variations in structure profile along the length of the trench (e.g., rutting) and the layer(s) affected (e.g., Figure 3.34), an informal assessment of moisture within the bound and subsurface layers (e.g., Figures 3.7 and 3.9), and perhaps most importantly, locations where water was flowing through the structure (e.g., Figures 3.6, 3.7, 3.9, 3.10, 3.13, 3.14, 3.18, 3.19, 3.29, and 3.34).

5.2.1 Pleasant Valley - Durkee

Two trenches were excavated through the shoulder and outside travel lane at MP 323.5 and MP 324.9, both on a downhill gradient, in the eastbound direction. During removal of the pavement slab from the trench at MP 323.5 it was observed that the SMA wearing course layer delaminated from the underlying dense-graded HMA inlay layer (Figure 3.4). It was also observed that lifts within the existing dense-graded HMA material (below the HMA inlay) had also delaminated (Figure 3.5). Nearly identical observations were made during the excavation of the trench at MP 324.9 (Figures 3.11, 3.12, and 3.13). These observations clearly indicated failure of the bond between layers and lifts. Close examination of the trench walls indicated the prevalence of moisture at the interfaces that had delaminated (Figures 3.9, 3.10, 3.12, and 3.13).

Following excavation, the broken faces of the pavement slabs were closely examined. These observations revealed that some stripping had occurred in all of the bound layers (Figures 3.8, 3.16, and 3.17).

Observations of the trench walls approximately 2 hours after excavation revealed that water was flowing through the pavement, predominately at layer and lift interfaces and, to a much lesser degree, through the layers near the interfaces at both locations (Figures 3.9, 3.10, 3.18, and 3.19). It was also noted that the PMBB layer appeared saturated throughout the time the trenches remained open (approximately 2½ hours).

These observations provide strong evidence to suggest that moisture-induced damage in the pavement structure was a significant contributing factor for the observed failures between layers and lifts of the bound layers during excavation of the trenches. Assuming that the conditions observed at the trenches were representative of the conditions elsewhere along the project, it can be postulated that moisture-induced damage was also a significant contributing factor for the observed rutting problems throughout the project.

5.2.2 Garden Valley – Roberts Creek

One trench was excavated through the outside travel lane at MP 118.5 on an uphill gradient in the southbound direction. Observations made while the trench was open clearly revealed water flow through the newly-placed dense-graded HMA inlay as well as through the lower portion of the existing dense-graded HMA layer (Figure 3.29). Stripping was also observed in the lowest approximately 4 inches of the bound layers.

These observations clearly indicate that the dense-graded HMA inlay and lowest dense-graded layer were permeable and that moisture-induced damage had occurred in lower portion of the bound layers. The presence of moisture and stripping in the bound layers observed at the trench strongly suggest that moisture-induced damage was also a significant contributing factor for the observed rutting problems throughout the project assuming, of course, that the conditions observed at the trench were representative of conditions throughout the project.

5.2.3 Vets Bridge – Myrtle Creek

One trench was excavated through the outside travel lane at MP 109.9 on an uphill gradient in the southbound direction. Observations made while the trench was open clearly revealed water flow through the top 2 to 3 inches of the existing dense-graded HMA material (Figure 3.34).

These observations clearly indicate that the top portion of the existing dense-graded HMA was permeable. Assuming that the conditions observed at the trench were representative of the conditions elsewhere along the project, it can be postulated that moisture within the bound layers was a significant contributing factor to the observed rutting problems throughout the project.

5.2.4 Summary Remarks

At all four trench locations it was clearly evident that water was flowing through the bound layers of the pavement structures, or along lift interfaces of the bound layers. It is likely that the source of this moisture was from surface moisture that infiltrated the bound layers through cracks, construction joints, or, for the Garden Valley – Roberts Creek and Vets Bridge – Myrtle Creek projects, the permeable wearing courses. The trenches along the Pleasant Valley – Durkee project were excavated in a cut area and the trench along the Garden Valley – Roberts Creek project was excavated at a transition from cut to fill with the cut section immediately uphill of the trench; hence, the moisture in the bound layers of these pavements may have come from surface moisture that infiltrated the pavements through the edges of the pavements or from the side ditches along the cut sections. In addition, the observance of water seeping out of the construction joints at isolated locations along the Pleasant Valley – Durkee project (see Appendix D) suggested the possibility of underground aquifers or localized springs as being yet another source of the moisture within the pavement along this project. These remarks highlight the importance of identifying potential sources of moisture along a project during the pre-construction site investigation phase of a project. Section 6.1 provides guidance on how this can be accomplished.

Given that HMA materials have a greater coefficient of permeability in the horizontal direction than in the vertical direction (*Kutay et al., 2007; Al Omari, 2004*), once water enters an HMA layer, it will more readily flow along the pavement (e.g., down a gentle slope) than vertically through the layer. Water seepage out of the cut faces of the uphill side of the trenches (transverse to the direction of travel) indicated horizontal flow of water down the sloped pavement (i.e., in the longitudinal directional). Similarly, water seepage out of the cut faces of the ends of the trenches (parallel to the directional of travel) indicated horizontal flow of water down the cross-slope of the pavement (i.e., in the transverse direction). In addition, water was not observed to seep out of the pavement surfaces downhill of the trenches. These observations highlight the importance of maintaining adequate cross-slope during rehabilitation activities, maintaining storm drains, edge drains, and ditches to ensure adequate drainage, and providing wearing course surface drains at regular intervals along gradients. Section 6.2 discusses these techniques in further detail.

In additional to the observations cited above, stripping was observed in the HMA layers exposed during the trenching activities in three of the four cases. Together, these findings point to moisture-induced damage as being a principal cause of the rutting problems observed along the projects. Assuming conditions throughout the projects were similar to those at the trench locations, it can be surmised that lack of adequate drainage of the pavement structures was a root cause for the presence of moisture and consequential stripping and rutting problems observed along the projects.

5.3 GROUND PENETRATING RADAR SURVEYS

Analyses of the data derived from the GPR surveys provided a continuous representation of the thickness of distinct pavement layers (i.e., those of differing age and/or mix design). Comparison of these results with layer/lift thicknesses measured from cores indicated reasonable correlations in many cases (Figures 3.21, 3.22, 3.26, 3.27, 3.30, 3.35, and 3.36). However, all layers/lifts were not identified by the GPR data analyses. The final report provided by the subcontractor responsible for the GPR surveys and data analyses (Appendix K) noted that core locations indicated on the core logs were often reported to the tenth of a mile and that they did not precisely correspond to the mile point indicated by the ODOT videologs. With this being the case, potential for discrepancy existed between GPR data and core data.

The GPR data analyses also provided a continuous representation of an indicator that the subcontractor termed "Differential Compaction," with values above a given threshold potentially indicative of moisture-induced damage. Where these data were compared to notations from a very limited number of core logs or observations at trench locations, the subcontractor reported reasonable correlation for the Anlauf – Elkhead Road and Garden Valley – Roberts Creek projects, but inconsistent results for the Vets Bridge – Myrtle Creek project.

Analyses of the GPR data also provided a continuous representation of an indicator that the subcontractor termed the "Activity Index," with values above a given threshold potentially indicative of moisture-induced damage. Where these data were compared to notations from a very limited number of core logs or observations at trench locations, the subcontractor reported a reasonable correlation for the Anlauf – Elkhead Road and Vets Bridge – Myrtle Creek projects, but a lack of correlation for the Garden Valley – Roberts Creek project.

It should be noted that, given the significant potential for discrepancy between core locations and GPR data analysis locations as well as the previously noted issues concerning potentially inaccurate notations on core logs, it is not surprising that stronger correlations were not realized between the core data and GPR data. Additional selective coring at locations identified by the GPR analyses as having a high and low Activity Index should provide sufficient information to conclusively evaluate the merits of GPR surveys for detecting locations of moisture-induce damage.

5.4 LABORATORY TESTS ON CORES

In summary, several cores from each project, except for the Cottage Grove – Martin Creek project, were tested for volumetric properties (bulk specific gravity, theoretical specific gravity, and air voids) as well as for water conductivity (permeability). The following paragraphs provide a discussion of the results of these tests.

5.4.1 Bulk Specific Gravity

Slices (lifts) from numerous cores were tested for bulk specific gravity utilizing both vacuum-sealed plastic bags (CoreLok) and parafilm to seal the specimens prior to determining the submerged mass of the specimens. Comparison of the results revealed a slight difference between the two methods, with the parafilm sealing method consistently giving lower values (Figures 4.1, 4.2, 4.3, 4.4, 4.6, 4.8, and 4.12). However, based on statistical comparisons at a 95% confidence level, there was sufficient evidence to indicate that the results were not significantly different in 23 of the 26 cases (88%) of the cases evaluated (Figures 4.6, 4.8, and 4.12).

It should be noted that ODOT uses AASHTO T 166 for determining the bulk specific gravity of compacted bituminous mixtures, where the specimens are not sealed in a plastic bag or with wax (parafilm) prior to determining the submerged mass of the specimen. Although AASHTO T 166 was not utilized in this study, other researchers have investigated the various methods for determining the bulk specific gravity of compacted bituminous mixtures.

In a study that investigated the CoreLok, parafilm, water displacement, and dimensional analysis methods on specimens prepared in the laboratory using a gyratory compactor as well as on cubes cut from the compacted specimens, Buchanan (*2000*) found that the vacuum sealing (CoreLok) method provided statistically similar results (at a 95% confidence level) to the water displacement (AASHTO T 166 or ASTM D 2726) method for fine- and coarse-graded Superpave mixtures in approximately 79% of the observations, largely owing to low water absorption (less than about 0.5%) of the mixtures during the water displacement tests. However, the CoreLok method yielded higher air void contents (lower bulk specific gravities) than the AASHTO T 166 method for stone matrix asphalt (SMA) and open-graded friction course (OGFC) mixtures (due to prevention of water intrusion into the specimens). Buchanan further found that the parafilm method provided similar results to the CoreLok and AASHTO T 166 methods for fine- and coarse-graded Superpave mixtures, but tended to overestimate air void content (i.e., underestimate the bulk specific gravity) for SMA and OGFC mixtures, due to bridging of the surface voids when using the parafilm. Buchanan also found that the use of the dimensional analysis method resulted in the lowest bulk specific gravities for the majority of specimens evaluated, and that the difference between this method and the others evaluated increased as the surface texture of specimens became rougher. Buchanan concluded that the vacuum sealing (CoreLok) method appeared to provide the most accurate measure of the bulk specific gravity of the specimens evaluated regardless of mixture type (fine- or course-graded Superpave mixtures, or SMA or OGFC mixtures), aggregate type (granite or limestone), compaction effort (15, 50, or 125 gyrations), or uncut versus cut specimens.

In a round-robin study involving bulk specific gravity measurements conducted at 18 laboratories on fine- and coarse-graded Superpave mixtures and SMA mixtures that were compacted in the laboratory using a gyratory compactor with 15, 50, and 100 gyrations and tested using the CoreLok device and by AASHTO T 166, Cooley et al. (*2003*) found statistically significant differences (at a 95% confidence level) between the two methods in 8 of the 9 treatments evaluated. Based in the results from the laboratory-prepared specimens, they concluded that the vacuum sealing method should be utilized for mixtures with aggregate gradations falling below the restricted zone when water absorption values are above 0.4%. It should be noted that this water absorption criterion proposed by Cooley et al. is similar in magnitude to the 0.5% value below which Buchanan (*2000*) found good agreement in results derived from the vacuum sealing and water displacement test methods.

Cooley et al. (2003) also found significant differences in air void contents for 3/8 inch NMAS and 1/2 inch NMAS fine- and coarse-graded Superpave mixtures, but not for 3/4 inch NMAS fine-graded Superpave mixtures, when the CoreLok method was compared with the water displacement (AASHTO T 166) method for determining the bulk specific gravity of 355 cores obtained from 42 projects across 16 states representing different climatic conditions, materials, mixture types, and construction techniques. They concluded from these results that the water displacement (AASHTO T 166) method should only be used for mixtures with aggregate gradations passing more than 5% above the maximum density line (i.e., fine-graded mixtures); the vacuum sealing method should be used for all other mixture gradations.

Clearly, the results derived from the study reported by Cooley et al. (*2003*) contradict those reported by Buchanan (*2000*). However, both are in agreement that the vacuum sealing (CoreLok) method provided a better assessment of the bulk specific gravity of the mixtures tested, particularly when the water absorption of the mixtures was on the order of about 1/2% or greater. Although the findings reported herein cannot be directly compared with those reported by Buchanan and Cooley et al., the conclusions drawn by Buchanan and Cooley et al. lend credence to the improvement in bulk specific gravity determination by virtue of sealing the specimens (with a plastic bag or wax) prior to determining the submerged mass, particularly if the specimens have a propensity for absorbing water.

5.4.2 Theoretical Maximum Specific Gravity

Several core slices from the Garden Valley – Roberts Creek and Vets Bridge – Myrtle Creek projects were tested for theoretical maximum specific gravity for the purposes of determining air void content. For both projects, there existed very low variation amongst the individual values as indicated by the very low coefficients of variation (Tables 4.8 and 4.12).

5.4.3 Air Void Content

The air void content was determined for the core slices from the Garden Valley – Roberts Creek and Vets Bridge – Myrtle Creek projects. For the former project, the results indicated that the dense-graded HMA inlay had an unexpectedly high air void content very similar to the overlying open-graded HMA overlay (Figure 4.9). Assuming that the test results are reasonably accurate, this indicates that the dense-graded layer was probably not adequately compacted in the stretch of pavement from which the cores were obtained and/or that moisture damage had occurred. The presence of water in the dense-graded layer hours after excavation of the trench along this project (Figure 3.29) indicated that the layer was conducting water and, thus, provides corroborating evidence to indicate that the layer was not adequately compacted or had incurred moisture damage, or both. It should be noted that the air void content of the open-graded HMA overlay was much lower than the design air void content of 13½ to 16%, possibly due to the mixture being clogged with debris or over-compacted during construction. The presence of water in the open-graded layer hours after excavation of the trench (Figure 3.29) indicated that the layer was retaining water, which likely contributed to the presence of water in the underlying dense-graded layer.

For the Vets Bridge – Myrtle Creek project, the test results indicated that the air void content of the dense-graded layer (top lift of existing HMA in Figure 4.13) was higher than expected for a pavement that had received years of trafficking, likely indicating that moisture damage had occurred assuming the layer had been properly compacted. The presence of moisture is this layer, as observed hours after the trench along this project had been excavated (Figure 3.34), provides corroborating evidence that moisture damage had likely occurred in this layer.

Another interesting observation of the results is that the air void content of the open-graded wearing course layer was greater in the outer wheel path than between the wheel paths (Figure 4.13). Given this, and that bleeding in the wheel paths had occurred in the stretch of pavement from which the cores were obtained, it is possible that moisture damage had occurred in the wheel paths of the open-graded HMA overlay.

5.4.4 Permeability

As found by other researchers (*McGhee, et al. 2008*), considerable variably existed amongst the permeability test results (Tables 4.3, 4.9, and 4.13). Nevertheless, the means (averages) of the test results indicated reasonable trends that generally corroborated the observations made during the trenching activities of the three projects that were trenched. That is, layers with relatively high permeability values were observed to conduct water, whereas layers with relatively low permeability values did not (compare Figure 3.29 with Figure 4.10 and compare Figure 3.34 with Figure 4.14).

6.0 GUIDELINES

One of the two principal objectives of this research study was to evaluate design, construction, and materials requirements that will minimize the risk of premature failures due to moisture-related damage in future rehabilitation projects. This was accomplished through evaluation of the findings from the literature review in combination with the findings from the forensic investigations. These efforts were undertaken with the ultimate goal of developing the following guidelines:

1. Guidelines for pre-construction site investigations to identify the potential for moisture-related problems,
2. Guidelines for pavement structural design when the potential for moisture-related problems exist,
3. Guidelines for construction techniques when the potential for moisture-related problems exist, and
4. Guidelines for materials selection and testing when the potential for moisture-related problems exist.

The guidelines developed under this project are provided in the following four sections. Companion checklists are provided in Appendices L through O.

6.1 PRE-CONSTRUCTION SITE INVESTIGATIONS

Chapter 4 of the ODOT Pavement Design Guide (*2007*) provides general guidance for data collection activities to assist in the development of pavement designs. The guidelines presented below are intended to supplement the guidance provided in the ODOT Pavement Design Guide (*2007*) and, in particular, to help pavement specialists and pavement engineers identify the potential for damage due to moisture in hot mix asphalt pavements and composite pavements (i.e., HMA over PCC) that are being considered for a mill-and-overlay or a mill-and-inlay-plus-overlay rehabilitation treatment. The guidelines assume the existing pavement structure exhibits sufficient distress so as to warrant rehabilitation. This being the case, the guidelines draw heavily upon techniques used in forensic investigations of pavement failures, with emphasis on determining existing characteristics of the pavement structure that might lead to moisture-related problems following rehabilitation.

The basic tasks for conducting pre-construction site investigations include: 1) records review, 2) site observations and condition surveys, 3) field investigations, 4) laboratory investigations, 5) data analysis, and 6) report of findings. The following provides guidelines for conducting each of these tasks.

6.1.1 Records Review

Conduct a review of historical records to obtain project-specific information so that the project can be clearly defined with regard to pavement characteristics and external influences (traffic and weather). Table 6.1 list the items considered to be essential for these purposes.

Table 6.1: Essential Information to be obtained During the Records Review.

Information Category	Specific Information
Pavement Construction	• Drawings, • Specifications, • Schedules, • As-built drawings, • Quality control and quality assurance records, • Results from field and laboratory investigations, and • Drainage system features.
Pavement Structure	• Thickness of each layer, and • Materials comprising each layer.
Pavement Materials	• Aggregate types and properties, • Asphalt binder type and properties, • Mix design details, and • Base and subbase (if applicable) materials information.
Soil and Geology	• Classification, • Geologic origin, and • Terrain.
Traffic	• Average daily traffic, • Percent trucks, and • Average daily truck traffic and/or equivalent single axle loads.
Historical Pavement Performance	• Pavement condition information from the ODOT Pavement Management System database and/or, • Details regarding type, severity, extent, and location of any distresses.
Weather	• Rainfall data, • Weather during construction and maintenance activities, and • Temperature data

6.1.2 Site Observations and Condition Surveys

Visit the site to conduct a visual examination of the project so as to document pavement distresses along the project and to develop potential locations and methods for field investigations, potential methods for laboratory investigations, and hypotheses for the cause(s) of distressed pavement areas. The following lists the recommended activities to accomplish these goals:

1. Hold a preliminary meeting with appropriate personnel to discuss the records review findings and to become familiar with the project location and surrounding area. This can be performed in conjunction with the following activity.

2. Conduct interviews with personnel familiar with the project (e.g., construction and maintenance personnel) to obtain their opinions and observations regarding

construction activities, pavement performance issues, and any maintenance treatments applied to the project under investigation. Findings from the records review can assist in the development of questions to be asked during the interview process. This activity can be performed in conjunction with the previous activity.

3. Conduct a condition survey of the pavement along the project to gather information regarding the type, severity, extent, and location of distresses. In addition to distress information, an informal assessment of drainage conditions, smoothness, traffic control options, and safety issues should be made. A minimum of 10% of the project area should be surveyed. Table 6.2, reproduced from the MEDPG documentation (*ARA 2004*), provides a comprehensive checklist of factors that should be considered when conducting the pavement condition survey. The ODOT Distress Survey Manual outlines the procedure for conducting the survey. During the survey, photograph the pavement in accordance with the ODOT Pavement Design Guide (*2007*) requirements as specified in Section 4.3.5. The ODOT Digital Video Log is also an excellent resource that can used to assist in obtaining a general overview of the condition of the pavement along the project.

4. Document the findings from the above activities. Table 6.3 lists the items that are recommended for inclusion in the document.

Table 6.2: Checklist of Factors for Pavement Condition Assessment and Problem Definition (*ARA 2004*).

Facet	Factors	Description
Structural adequacy	Existing distress	1. Little or no load/fatigue-related distress 2. Moderate load/fatigue-related distress (possible deficiency in load-carrying capacity) 3. Major load/fatigue-related distress (obvious deficiency in current load-carrying capacity) 4. Load-carrying capacity deficiency: (yes or no)
	Nondestructive testing (deflection testing)	1. High deflections 2. Are backcalculated layer moduli reasonable? 3. Are joint load transfer efficiencies reasonable?
	Nondestructive testing (GPR testing)	1. Determine layer thickness
	Nondestructive testing (profile testing)	1. Determine joint/crack faulting
	Destructive testing	1. Are cores strengths and condition reasonable? 2. Are the layer thicknesses adequate?
	Previous maintenance performed	Minor, Normal, Major
	Has lack of maintenance contributed to structural deterioration?	Yes, No, Describe________________
Functional adequacy	Smoothness Cause of smoothness deficiency	Measurement _____________ Very Good, Good, Fair, Poor, Very Poor Foundation movement Localized distress or deterioration Other
	Noise	Measurement _____________ Satisfactory, Questionable, Unsatisfactory
	Friction resistance	Measurement _____________ Satisfactory, Questionable, Unsatisfactory
Subsurface drainage	Climate (moisture and temperature region)	Moisture throughout the year • Seasonal moisture • Very little moisture • Deep frost penetration • Freeze-thaw cycles • No frost problems
	Presence of moisture-accelerated distress	Yes, Possible, No
	Subsurface drainage facilities	Satisfactory, Marginal, Unsatisfactory
	Surface drainage facilities	Satisfactory, Marginal, Unsatisfactory
	Has lack of maintenance contributed to deterioration of drainage facilities?	Yes, No, Describe________________
Materials durability	Presence of durability-related distress (surface layer)	1. Little or no durability-related distress 2. Moderate durability-related distress 3. Major durability-related distress
	Base erosion or stripping	1. Little or no base erosion or stripping 2. Moderate base erosion or stripping 3. Major base erosion or stripping
	Nondestructive testing (GPR testing)	1. Determine areas with material deterioration/moisture damage (stripping)

Table 6.2: Checklist of Factors for Pavement Condition Assessment and Problem Definition (continued) (*ARA 2004*).

Facet	Features	Description
Shoulder adequacy	Surface condition	1. Little or no load-associated/joint distress 2. Moderate load-associated/joint distress 3. Major load-associated/joint distress 4. Structural load-carrying capacity deficiency: (yes or no)
	Localized deteriorated areas	Yes, No Location:
Condition/ performance variability	Does the project section include significant deterioration of the following: • Bridge approaches • Intersections • Lane to lane • Cuts or fills	 Yes, No Yes, No Yes, No Yes, No
	Is there a systematic variation in pavement condition along project (localized variation)?	Yes, No
	Systematic lane to lane variation in pavement condition	Yes, No
Miscellaneous	PCC joint damage: • Is there adequate load transfer (transverse joints)? • Is there adequate load transfer (centerline joint)? • Is there excessive centerline joint width? • Is there adequate load transfer (lane-shoulder)? • Is there joint seal damage? • Is there excessive joint spalling (transverse)? • Is there excessive joint spalling (longitudinal)? • Has there been any blowups?	 Yes, No Yes, No Yes, No Yes, No Yes, No Yes, No Yes, No Yes, No
	Past maintenance • Patching • Joint resealing	 Yes, No Yes, No
	Traffic capacity and geometrics • Current capacity • Future capacity • Widening required now	 Adequate, Inadequate Adequate, Inadequate Yes, No
Constraints?	Are detours available for rehabilitation construction?	Yes, No
	Should construction be accomplished under traffic?	Yes, No
	Can construction be done during off-peak hours?	Yes, No, Describe_______________
	Bridge clearance problems	Describe_______________
	Lateral obstruction problems	Describe_______________
	Utilities problems	Describe_______________
	Other constraint problems	Describe_______________

Table 6.3: Recommended Information to Include in the Documentation for Site Visits and Condition Surveys.

Section	Information to Include in the Section
Introduction	• Project identification (e.g., name, highway number, beginning and ending mile points, etc.), • Surveyors and date(s) of survey, and • Brief description of methodology (e.g., windshield survey of approximately 15% of the project area with detailed condition assessment, photographs, and video).
Data Collected	• Narration of information gathered, • Sketches, drawings, and photographs, and • Map(s) or line chart(s) showing distresses and other features such as structures, drainage features, potential areas for further evaluation, areas in which to avoid further investigation due to safety concerns, etc.
Discussion	• Principal findings from the records review and site visit/condition survey (with emphasis on the assessment of pavement structural, functional, and drainage adequacy, materials durability, maintenance applications, adequacy of shoulders, and variability within the project).
Conclusions and Recommendations	• Hypotheses for the cause(s) of distressed pavement areas, • Additional data needs, • Potential locations and methods for field investigations, • Potential methods for laboratory investigations.
Appendices	• Meeting notes, field notes, raw data, etc.

6.1.3 Field Investigations

Develop a plan for and conduct field investigations with the aim to evaluate the following major aspects of the existing pavement (*ARA 2004*):

- Structural adequacy
- Functional adequacy
- Subsurface drainage adequacy
- Material durability
- Shoulder condition
- Variation in pavement condition
- Miscellaneous constraints

The following lists the recommended activities to accomplish these objectives:

- Develop a plan to conduct field testing and sampling that is guided by the information gathered during the records review and the recommendations derived from the site visit and condition survey documentation as well as the recommendations identified in the ODOT Pavement Design Guide (*2007*). The plan should include consideration for the following activities:

- Division of the project into segments with similar structural design, site conditions (e.g., traffic levels), and pavement conditions.
- Deflection measurements using a falling weight deflectometer (FWD) at intervals not exceeding 250 feet.
- Pavement cores from the travel lane(s) at intervals not exceeding ½ mile (2640 feet). Information about the cores should be recorded on core log sheets as indicated in the ODOT Pavement Design Guide (*2007*). If the records review and/or site visit/condition survey indicates that moisture damage may have occurred along portions of the project, it is recommended to obtain additional pavement cores to verify/refute the presence of moisture damage. The following provides guidance for obtaining additional cores for these purposes (*NHI 1999*):
  - Based on information from the records review and/or site visit/condition survey, identify two 500-foot long sections; one that represents a distressed area, and the other that represents an area with minimal or no distress.
  - Using a dry cutting process (e.g., using compressed air or CO_2 instead of water), obtain at least six 4-inch diameter cores from randomly-selected locations from both areas. On four-lane highways, obtain the cores from the inside wheel path of the outside travel lane. On two-lane highways, obtain the cores from the outside wheel path. Record information about the core on core log sheets and photograph the cores as per instructions in the ODOT Pavement Design Guide (*2007*). Place the cores in air-tight containers as soon as possible following the documentation activities.
- Exploration holes as outlined in the ODOT Pavement Design Guide (*2007*).
- Evaluation of drainage along the project. Some of the questions that should be answered during such an evaluation are listed below (*ARA 2004*) whereas Figure 6.1 provides an example form that can be used for drainage surveys (*NHI 1999*):
  - How deep are the ditches?
  - Is the flow-line of the ditch beneath the top of the subgrade?
  - Are the ditches clear of standing water?
  - Are the ditches and pavement edges free of vegetation that would clog the drainage path?
  - After rainfall, does water stand in the joints and cracks? Is there evidence of pumping or water bleeding? Does water stand at the outer edge of the shoulder, or is there evidence that the water may pond on the shoulder?
  - Are inlets clear and set at proper elevations, with adequate cross-slope to get water to the pavement edge?
  - Are joint or crack sealants in good condition, and do they prevent water from entering the pavement?
  - If a subsurface drainage system is present:
    - Are the outlets clearly marked and easily found?
    - Are the outlets clear of debris and set at the proper elevation above the bottom of the ditch?

        - Are the drainage system components properly installed?
        - Are the drainage system components functional?
    - Consideration for conducting a ground penetrating radar (GPR) survey. Information obtained from such a survey can provide valuable information with regard to thickness of pavement layers and potentially can provide an indication of areas with moisture damage.
    - Consideration for excavating at least one trench along the project, particularly if moisture damage is suspected to have occurred. Although not necessarily recommended for routine field investigation, excavating a trench in a distressed area can provide valuable information that cannot always be obtained from cores and exploration holes alone. In particular, the additional information can include verification of observations derived from cores taken in the vicinity of the trenches, subsurface profile and condition, variations in structure profile along the length of the trench (e.g., rutting) and the layer(s) affected, an informal assessment of moisture within the bound and subsurface layers, and locations where water is flowing through the structure.
- Coordinate with district maintenance personnel, motor carrier personnel, pavement services field crew personnel, and any required subcontractors (e.g., asphalt/concrete cutters, GPR surveyors, etc.) dates and times to conduct the field investigations.
- Conduct the field investigations.

Field Survey: ***Drainage Information*** Project ID: ________________
Date of survey (mm/dd/yy): Surveyor's initials: __________

Slope Measurements:

	Station	Slope	
		Outer	Inner
Longitudinal Slope (m/m) 3 measurements equally spaced along the project	+	/	/
	+	/	/
	+	/	/
Cross-slope (m/m) 3 measurements equally spaced along project	+	/	/
	+	/	/
	+	/	/
Shoulder Slope (m/m) 3 measurements equally spaced along project	+	/	/
	+	/	/
	+	/	/

Cut/Fill and Ditch Line Depth:

Circle if Cut/Fill Depth Uniform	Cut/Fill Depth	Station		Depth of Ditch Line
1	Fill > 13.3 m	+	+	Meters
2	Fill 4.85 – 13.3 m	+	+	Meters
3	Fill 1.82 – 4.85 m	+	+	Meters
4	At Grade (1.5-m fill to 1.5-m cut)	+	+	Meters
5	Cut 1.82 – 4.85 m	+	+	Meters
6	Cut 4.85 – 13.3 m	+	+	Meters
7	Cut > 13.3 m	+	+	Meters

Lane/Shoulder Joint Integrity:

	Outer Shoulder	Inner Shoulder
Sealant Damage	N L M H	/
Blow Holes	N L M H	/
Sealant Type	None HP SI Preform Other	

Subsurface Drainage (visual): Type of drainage system present: ________________________
(1 = none; 2 = longitudinal drains; 3 = transverse drains; 4 = other)

Condition of Drainage Outlets: ____

Indicators of Poor Drainage:

Cattails or willows growing in ditch: Y / N
Drainage outlets clogged: Y / N
Drainage outlets below ditch line: Y / N
Non-continuous cross-section, crown to drainage ditch: Y / N
Pumping: N L M H
Other: ___________________________

Figure 6.1: Example Form for Conducting Drainage Surveys (*NHI 1999*).

6.1.4 Laboratory Investigations

The cores obtained during the field investigation phase should be grouped in accordance with the division of segments along the project. A sufficient number of cores from each grouping should be selected for laboratory testing. The actual number of cores selected is the responsibility of the pavement engineer or pavement specialist, but should take in account a balance between cost and an adequate number of tests to ensure statistical significance. In the latter regard, it is recommended that an absolute minimum of three cores per segment be selected for each laboratory test conducted, but a greater number would be preferred. With regard to cost, it should be realized that each core will likely consist of more than one layer of unique material (e.g., an open-graded mix over a dense-graded mix, or multiple layers of dense-graded mix placed during multiple projects spanning years or decades, etc.); hence, one core will likely require at least one laboratory test for each unique material. To determine if moisture damage has occurred, if not obvious from visual observation of the as-received cores, the following laboratory procedures and tests are recommended:

1. Cut each core into slices that represent unique material. Information from the records review can assist in determining expected depths of unique materials. Differences in appearance can also assist in this regard.

2. Test each core slice for bulk specific gravity in accordance with AASHTO T 166 Method A or AASHTO T 331 (with determination of the dry mass performed last in either procedure).

3. Split each core slice along its diameter and examine it visually to determine if moisture damage (stripping) has occurred. Ignore crushed or fractured aggregate particles. Heat the specimen just enough to push it apart by hand and observe the extent of stripping. A rating of the stripping on the exposed surface or aggregate particles should be made and documented. Save the material for theoretical maximum (Rice) specific gravity testing.

4. Test the material from each core slice for theoretical maximum (Rice) specific gravity in accordance with AASHTO T 209. If needed, combine like material from individual core slices to obtain a sufficient quantity of material to perform the test. Note that more than three core slices may be needed for thin layers; hence, more than three cores may be needed per segment of the project to satisfy the minimum quantity requirement for the test.

5. Determine the air void content for each layer (core slice) in accordance with AASHTO T 269.

6. Optionally, test each unique material (layer) for moisture susceptibility in accordance with AASHTO T 283 (field mixed, field compacted method).

7. Optionally, test each unique material (layer) for permeability (water conductivity) in accordance with Florida DOT test method TM 5-565.

8. If additional cores were obtained from areas with suspected moisture damage and from areas with minimal or no distress (to verify or refute the presence of moisture damage), each set of cores should be tested as described above, including AASHTO T 283. Using the mean and standard deviation from each set to test results, the student's t-test statistic can be used to compare means for a given confidence level.

6.1.5 Data Analysis

Conduct analyses of the information derived from the field and laboratory investigations with the aim of assessing the pavement structural, functional, and drainage adequacy, materials durability, and variability within the project. Chapter 5 of Part 2 of the MEDPG (*ARA 2004*) documentation provides general guidance for accomplishing these objectives.

6.1.6 Report

Prepare a report to document the efforts and findings from the above investigations. Chapter 12 of the ODOT Pavement Design Guide (*2007*) provides guidance for such documentation.

6.2 PAVEMENT STRUCTURAL DESIGN TECHNIQUES

The guidelines presented below are intended to assist a pavement designer in developing hot mix asphalt (HMA) design alternatives for rehabilitating HMA pavements with emphasis on reducing the risk of failure due to moisture damage in the rehabilitated pavement. The guidelines do not include all-encompassing recommendations for the structural design of HMA rehabilitation alternatives. Hence, these guidelines are intended to be used as a supplement to guidelines for the design of the pavement for other requirements (e.g., load-carrying capacity, functional design, etc.) such as that provided in the ODOT Pavement Design Guide (*2007*). The guidelines are also intended to be used in conjunction with the guidelines for pre-construction site investigations (Section 6.1) and the guidelines for materials selection and testing (Section 6.4), both of this report.

The structural design process for rehabilitation of HMA pavements is a multi-step process involving evaluation of the existing pavement, rehabilitation strategy selection (including materials incorporated), structural design, and life cycle cost analysis and optionally consideration of non-monetary factors. Section 6.1 of this report addresses evaluation of the existing pavement. The following guidelines address 1) rehabilitation strategy selection and 2) structural design. Recommendations concerning life cycle cost analyses are covered in the ODOT Pavement Design Guide (*2007*), whereas non-monetary considerations are covered in the MEDPG documentation (*ARA 2004*).

6.2.1 Rehabilitation Strategy Selection

One of the principal tasks of rehabilitation strategy selection is to determine feasible strategies that address the cause of pavement deterioration and effectively prevent or minimize its reoccurrence. The following lists activities that can assist in developing feasible strategies to repair or prevent pavement deterioration due to moisture damage:

1. Based on the documented findings from the pre-construction site investigation activities (see Section 6.1), determine if the existing pavement is a candidate for an inlay and/or inlay/overlay rehabilitation treatment. The following lists scenarios in which the existing pavement would not be considered a viable candidate for an HMA overlay (and/or inlay) rehabilitation treatment (*ARA 2004*). That is, if one or more of the following scenarios is/are true, then the pavement should be reconstructed rather than rehabilitated with an inlay and/or overlay.

    - The extent of high-severity fatigue cracking is so great as to dictate complete removal and replacement of the existing pavement.
    - Excessive surface rutting indicates insufficient stability in the existing materials to prevent recurrence of severe rutting in the rehabilitated pavement.
    - Existing stabilized base materials show signs of a sufficient amount of significant deterioration so as to require an inordinate amount of repair to provide uniform support for the new HMA material (i.e., overlay).
    - Existing granular base is sufficiently contaminated with fine-grained soil (clay and/or silt) to warrant removal and replacement.
    - Stripping throughout the existing HMA layers dictates removal and replacement of most or all of the HMA layers.

2. Assuming that the existing pavement can be considered as a viable candidate for rehabilitation through use of an HMA inlay and/or HMA overlay, formulate a list of practical rehabilitation strategy alternatives. Typical strategies include:

    - Open-graded HMA overlay.
    - Dense-graded HMA overlay.
    - Gap-graded HMA overlay (i.e., stone matrix asphalt—SMA).
    - Removal of a portion of the existing HMA layers and replacement with new material (open-graded, dense-graded, or gap-graded mix).
    - Inlay (i.e., remove and replace) plus overlay. In such a case, the inlay material would typically consist of a dense-graded material, but the overlay could include any of the overlay options listed above.

    Note that constraints such as climatic conditions (e.g., freeze-thaw cycling at higher elevations, coastal environment, etc.) and maintenance activities such as snow plowing, ease of construction (e.g., contractor inexperience with SMA), risk of premature failure, traffic levels, cost, etc., may preclude the use of open-graded and/or gap-graded mixtures. In addition, resistance to moisture damage depends more on the compatibility between the asphalt cement and the aggregate than the nominal maximum size of aggregate and, for dense-graded mixtures, the permeability of the compacted mixture. Hence, if properly designed, produced, and compacted, either a 3/4 inch or a 1/2 inch nominal maximum size of aggregate would be expected to be equally resistant to moisture damage.

6.2.2 Structural Design

With the aid of the ODOT Pavement Design Guide (*2007*), develop pavement structures for the list of viable rehabilitation strategies to satisfy the criteria for load-carrying capabilities. Inherent in this activity is specification of material attributes such as binder grade and volumetric properties such that the HMA mixture will not crack at low temperatures or deform excessively at high temperatures. The materials and their attributes should also be specified such that the mixture is resistant to moisture damage (see Section 6.4).

Additional considerations regarding the design of the structure to minimize the risk of moisture damage in the rehabilitated pavement include provision for drainage of moisture that enters the pavement (i.e., drainage design). Sources of moisture in the pavement structure and subgrade include:

- Seepage of moisture from the water table into the pavement through capillary action and vapor movement.
- Flow of water from the pavement edges or side ditches into the pavement structure and subgrade.
- Infiltration of rain and snow/ice melt into the pavement through cracks or other surface deficiencies (e.g., segregated wearing course mixture), construction joints, and permeable wearing course mixtures (e.g., open-graded mixtures).

Moisture that enters and remains in the pavement structure for extended periods can cause moisture damage (e.g., stripping) or contribute to or accelerate damage initiated by other factors such as wheel loading or freeze-thaw cycling. Hence, minimization of moisture infiltration, use of moisture-insensitive materials, or incorporation of means or methods for rapid removal of water from the pavement structure can significantly reduce the risk of moisture-related problems in the rehabilitated pavement. Each of these approaches is discussed in further detail as follows:

Minimization of Moisture Infiltration. From a practical standpoint, water infiltration into the pavement cannot be prevented, but it can be minimized. Design considerations to accomplish this include pavement geometry, pavement edge drainage, and wearing course surface drains.

- Pavement geometry – adequate cross-slopes and longitudinal slopes are essential for quickly removing water from the surface of the pavement, thus minimizing the time available for the water to infiltrate the pavement. The ODOT Highway Design Manual provides details regarding pavement cross-slopes and longitudinal gradients.
- Pavement edge drainage – adequate drainage of water away from the pavement edge will help prevent it from entering the structure. This can be accomplished with adequately designed ditches or edge drains. The ODOT Highway Design Manual provides details regarding these features. Although not a design feature, routine inspection and maintenance of these features will help to ensure proper functioning of the features over time.
- Wearing course surface drains – The ODOT Highway Design Manual recommends wearing course surface drains for pavements with an open-graded wearing course

mixture that slopes toward and abuts an impermeable surface (i.e., adjacent dense-graded or portland cement concrete pavement or bridge deck), or as recommended by the pavement designer. Oregon Standard Drawing RD314 provides details of wearing course surface drains. In addition to the recommendations provided in the ODOT Highway Design Manual, installation of this drainage feature should be considered for the following conditions along projects with an open-graded wearing course:

- Long stretches of gentle or no gradient, particularly along stretches that incorporate a curb.
- Long stretches in cut sections, particularly if the top of cut is substantially higher than the pavement surface.
- Mill-and-inlay or mill-and-inlay-plus-overlay projects, particularly if the milled surface does not extend to the edge of pavement.

Use of Moisture-Insensitive Materials. Although traditionally considered a mix design issue, the pavement designer should ensure that the materials incorporated in the HMA layers have been tested for moisture sensitivity and surpass minimum acceptable requirements (see Section 6.4).

Rapid Removal of Water from the Pavement Structure. In addition to surface drainage (discussed above), removal of free water from the pavement structure involves groundwater drainage and subsurface drainage, where free water refers to moisture not held by capillary forces in soil and fine aggregate. Groundwater drainage is an issue usually handled by geotechnical engineers during embankment design. Hence, the Geo-Environmental Section should be consulted in developing solutions for groundwater drainage design issues.

Subsurface drainage systems are intended to remove water that infiltrates the pavement structure primarily through discontinuities (e.g., cracks) in the pavement surface. With adequate design and proper functioning, such systems minimize the time that the pavement structure is completely or partially saturated and, hence, can significantly reduce the propensity for moisture-related damage. The methods in common usage to accomplish adequate drainage are discussed as follows:

- Edge drain systems – Edge drains are typically perforated pipes or prefabricated geocomposite edge drains (PGEDs) installed adjacent to the lane-shoulder joint that collect water discharged from the pavement structure and convey it to outlets that, in turn, convey it to side ditches. If edge drains exist, these should be inspected and/or tested for proper functioning. Any deficiencies should be corrected during the pavement rehabilitation activities. If edge drains do not exist, consideration should be given to retrofitting the pavement structure with such a system.

- Side ditches – Side ditches convey water discharged from the pavement surface and the subsurface drainage system (if installed) away from the pavement. Side ditches should be inspected to ensure adequate minimum longitudinal grade and sufficient freeboard. Any deficiencies should be corrected during the pavement rehabilitation activities.

- Storm drains – Storm drains are used in areas that cannot accommodate side ditches (e.g., urban areas) and, like side ditches, convey water discharged from the pavement surface and the subsurface drainage system (if installed) away from the pavement. Storm drains should be inspected and/or tested to ensure adequate functioning. Any deficiencies should be corrected during the pavement rehabilitation activities.

- Daylighted base/subbase – Daylighted bases and subbases are exposed at the shoulder-ditch or embankment-ditch interface to allow discharge of water from these layers into the side ditch. The exposed surface should be inspected to ensure that it is not clogged with debris or covered with vegetation. Periodically scraping the exposed surface and applying herbicide can help ensure adequate flow of water from the base and subbase into the side ditches.

It should be emphasized that drainage systems, independent of type, need to be inspected at least annually and maintained as warranted. Lack of maintenance will likely result in reduced effectiveness of the system and could conceivably render the system completely ineffective. In the case of reduced effectiveness, water will remain in the pavement for longer durations and thus likely contribute to moisture-related distress. Worse yet, a completely ineffective (i.e., clogged) drainage system will prevent drainage of free water from the pavement structure potentially leading to long periods of saturated conditions which will eventually result in substantial moisture-related distress and significant pavement failure. Hence, ease of maintenance of the system should be taken into account when considering the above options. Daylighted bases/subbases and ditches are easier to maintain than are subsurface systems (including storm drains). If maintenance of the system is unlikely (e.g., due to budgetary constraints, lack of maintenance personnel, etc.), it may be better to not include a subsurface system unless, of course, other factors such as topography, stretches of pavement with standing water, urban environment, etc. dictate otherwise (in which case, it is advised to dedicate funds, equipment, and personnel to the maintenance of the system).

6.3 CONSTRUCTION TECHNIQUES

The guidelines presented below are intended to assist ODOT personnel responsible for oversight of construction activities during the rehabilitation of HMA pavements with emphasis on techniques that reduce the risk of failure due to moisture damage in the rehabilitated pavement. More specifically, the guidelines cover 1) surface preparation of the existing pavement, 2) production of hot-mix asphalt, 3) paving operations, 4) pavement drainage, and 5) quality control/quality assurance.

6.3.1 Surface Preparation

Surface preparation (or pre-overlay) treatment involves the processes necessary to make the existing pavement ready to receive an overlay (or inlay). The processes that can help minimize the risk of moisture-related problems in the rehabilitated pavement include:

1. Ensuring adequate cross-slope,
2. Proper cold milling operations,

3. Ensuring the surface is clean immediately before placing the overlay, and
4. Proper application of the tack coat.

These processes are discussed in further detail below.

Cross-slope of the prepared surface. The surface to receive the overlay should be nearly planar and have a cross-slope to promote drainage away from the travel lanes. Restoration to a nearly planar surface can be accomplished by surface leveling (rut filling) or cold milling. Adequate cross-slope can be achieved by a leveling course or cold milling. Generally, a cross-slope of 2% is recommended.

Proper cold milling operations. The following practices for cold milling operations are recommended:

- Repair or identify locations of failed pavement areas prior to cold milling as these may be difficult to locate after the surface has been milled.
- Ensure the milling operation produces a uniform texture. The surface texture is controlled by the forward speed of the machine, the rotational velocity of the cutting drum, the configuration of the carbide bits, the degree of wear of the carbide bits, and the grade control of the drum. ASTM E965 can be used to assess the quality of the milled surface.
- The longitudinal profile of cold milling should be held to the same tolerance as that of new base course construction.
- The transverse profile (cross-slope) should be specified as discussed above.
- Cold milling limits as well as transitions or stop lines at bridges and ramps should be clearly identified on plans and adhered to closely on the project.

Ensure the surface is clean. The milling operation results in a roughened surface that can be quite dirty and dusty. Hence, it should be adequately cleaned by sweeping or washing to ensure proper bond between the milled surface and overlay. More than one pass of the broom may be required to remove a sufficient amount of dirt and dust when sweeping. The pavement surface should be allowed to dry prior to tack coat application when washing.

Proper tack coat application. The tack coat should be applied at the specified dilution and at an appropriate rate such that the milled surface is completely and uniformly coated. Clogged nozzles and spray bar height can significantly affect the uniformity of the applied tack coat.

6.3.2 Pavement Drainage

Many, if not most, problems with drainage systems originate in the construction phase. Satisfactory long-term performance of drainage systems therefore depends on good construction control. Best practices include:

- Ensure the paved lanes have adequate cross-slope.

- Maintain tight control of edge drain grades. Water collects in sags of undulating pipelines. Outlets above edge drains are not effective. Ensure that outlets are placed in sag curves.
- Backfill edge drain trenches with material at least as permeable as the material (base or subbase) it is intended to drain. Ensure the backfill material meets specifications. Compact the backfill material adequately, but without damaging the pipe.
- Ensure that heavy construction equipment does not crush edge drain and outlet pipes.
- Ensure that guardrail and delineator posts are not driven through edge drain and outlet pipes.
- Inspect and test edge drain systems near the end of construction, before final acceptance.

6.3.3 HMA Production

Adherence to proper production techniques so as to achieve uniform, high quality hot-mix asphalt can help minimize moisture damage. These are discussed in more detail as follows:

Aggregate stockpiling. Proper stockpiling of aggregates can help ensure production of uniform, high quality hot-mix asphalt. Issues of particular concern to minimize moisture-related damage in the HMA include cleanliness, segregation, and moisture content. Asphalt cement coats and adheres to clean aggregates better than dusty aggregates; hence, use of clean aggregates for hot-mix asphalt provide better resistance to moisture damage than do dusty/dirty aggregates. Frequent Sand Equivalent testing can be conducted to determine the cleanliness of the aggregates. Segregated aggregates (particularly for coarse-graded blends) are more prone to moisture-related damage than are blends without segregation. Frequent gradation analyses can be conducted to determine if the aggregates are significantly segregated. Moisture content of stockpiled aggregate can affect the resistance to moisture damage of the hot-mix asphalt produced using the aggregates; water trapped in aggregates (particularly high absorptive aggregates) can eventually displace the asphalt cement coating the aggregate resulting in moisture-related damage. Building and maintaining stockpiles on a slight slope can facilitate drainage of free water from the stockpile and help to avoid wide variation in the moisture content of the stockpiled aggregates.

Cold feed system. Appropriate feed rates from cold bins ensure that the proper aggregate blend is being provided to the mixing drum, thus minimizing the propensity for segregation and, in turn, the propensity for moisture-related in the hot-mix asphalt. Frequent gradation analyses (during the mix design verification process) can be used to verify that the proper aggregate blend has been used to produce the hot-mix asphalt.

Drying and mixing process. Water can become entrapped in aggregates when coated with asphalt cement during the mixing process if the aggregates are not adequately dried prior to mixing, which can result in moisture-related damage in the hot-mix asphalt. The drying time prior to mixing should be adjusted according to the moisture content of the aggregate blend with higher moisture contents requiring longer drying times. Hence, the moisture content of the aggregates should be monitored during hot-mix asphalt production. The mixing operation should also be managed properly to ensure adequate coating of the aggregates with asphalt cement. Observations of uncoated or partially coated aggregates or a particularly dry-looking mix are indications of improperly managed mixing operations.

Loading the mixture into haul units. Aggregate segregation can occur due to improper loading of the mix into the haul units which, in turn, can lead to moisture damage in the finished mat. Discharging the mix from a silo or batcher into the haul unit in one large mass can cause the larger-sized particles to roll off the pile and collect near its base. Hence, rather than discharging the mix into one large pile, it is recommended to create two or more smaller piles to minimize the amount of segregation that occurs during load out. In addition, the haul units should be cleaned and lubricated prior receiving the hot-mix asphalt. Non-petroleum based lubricants are recommended.

Transport of the mixture. The two primary issues that influence moisture-susceptibility of the hot-mix asphalt during transport to the paving site include draindown and thermal segregation. Draindown can occur in mixes with thick binder films (e.g., open-graded or gap-graded mixtures) resulting in inadequately coated aggregate particles that are prone to moisture damage. Observation of asphalt cement dripping from the haul units or fat spots in the finished mat are indications that draindown may be occurring. Thermal segregation results when the exterior portion of the hot-mix asphalt cools and forms an insulating crust surrounding the hotter, interior mass. If the cooler mass of hot-mix asphalt is not adequately remixed prior to or during paving, it will not be compacted to the same degree as the hotter mass leading to lower density, higher permeability, and higher propensity to moisture damage. Minimizing haul distances, utilizing haul units with insulated beds, covering the mix with a tarpaulin (when using haul units that do not have an insulated bed), or remixing the hot-mix asphalt utilizing a material transfer vehicle can minimize problems that occur due to thermal segregation.

Loading the paver. Several construction techniques associated with loading the paver can influence moisture-susceptibility of the hot-mix asphalt. Best practices include:

- Loading the paver as quickly as possible and avoid queuing of haul units so as to minimize excessive cooling of the mixture.
- Ensuring the correct mixture has been delivered, particularly if more than one mix design is utilized on the project.
- Preventing the paver hopper from being completely emptied between loads so as to avoid segregation.
- Utilizing a material transfer vehicle whenever possible or practical, particularly when the haul distances are very long. These units are effective in avoiding both aggregate and thermal segregation.

6.3.4 Paving Operations and Compaction

Proper paving and particularly compaction operations are important construction techniques that can help prevent moisture-related distress in the finished pavement. These are discussed in further detail as follows.

Paving operations. Mixture (aggregate) segregation is the key issue with regard to the paving operation that can be controlled so as to minimize moisture damage. Ideally, the paver should operate without stopping to avoid the almost inevitable open texture that occurs between trucks loads but, as mentioned previously, the hopper should at least never be allowed to be completely emptied between loads. Additionally, excessive folding of the hopper wings should be avoided as this can cause segregation. Finally, haul units that load directly into the paver hopper should avoid bumping hard into the paver.

Compaction. Assuming a properly designed mixture, achieving adequate density (but not too high of density) through compaction of the mixture, is the most important technique in the construction of a pavement that is resistant to moisture damage. Numerous studies have indicated that dense-graded mixtures should be compacted to at least 92% of theoretical maximum density so as to minimize interconnectivity of air voids and, hence, reduce permeability (rich binder bases should be compacted to at least 94% of theoretical maximum density). Establishing and maintaining a proper rolling pattern should, in most cases, result in adequate compaction. Too much compaction, particularly at inadequate mat temperatures, should also be avoided so as to prevent crushing and breakage of aggregate particles.

6.3.5 Quality Control and Quality Assurance

Quality Control (QC) and Quality Assurance (QA) are some of the most important aspects of the construction process. Some of the best practices utilized to ensure the quality of hot-mix asphalt include:

QC/QA plans. QC and QA plans developed for hot-mix asphalt paving projects need to be implemented. A QC plan should state the quality policies, practices, organization, and activities that will be conducted to produce a quality product. QA testing should be conducted as specified in the plan to verify the QC test results.

Checklists. Checklists should be developed to identify all important steps or actions that need to be undertaken at each step in the construction process. Effective checklists have the following key characteristics:

- Standardized form;
- Clear and simple form to facilitate ease of recording;
- Appropriate spacing for recording;
- Clear and concise directions for use of the form; and
- Ample space for recording the project number and location, weather conditions, signature, date, and remarks.

Daily diaries. Implementation of QC /QA plans necessitates keeping daily diaries (in addition to checklists) to record daily project conditions and activities and, in particular, items such as changes that occur during operations, different or unusual events on the projects, visitors to the project, names and titles of any persons involved in discussions (including topics discussed and outcomes) regarding the project, and reasons for operational delays (e.g., equipment breakdown or inclement weather).

Feedback system. All personnel, independent of position, should be trained on the worthiness of feedback regarding any operational problem or defect associated with the project. This will help ensure that timely and proper corrective actions are taken before it is too late.

6.4 MATERIALS SELECTION AND TESTING

The guidelines presented below are intended to help ODOT materials engineers select materials for hot-mix asphalt (HMA) mixtures that will reduce the risk of moisture-related problems in rehabilitated HMA pavements that incorporate the mixtures. The guidelines are intended to supplement the guidance provided in the ODOT Contractor Mix Design Guidelines for Asphalt Concrete.

The principal ingredients of hot-mix asphalt are mineral aggregates and asphalt cement, and resistance to moisture-induced damage to the mixture largely depends on the strength of the adhesive bond established between the asphalt cement and mineral aggregate. The strength of the bond, in turn, largely depends on chemical compatibility between the asphalt cement and mineral aggregate (which can be enhanced using additives such as lime or anti-stripping agents). The following discusses materials selection and testing for 1) aggregates, 2) asphalt cements, 3) HMA mixtures, and 4) additives so as to reduce the risk of moisture-related problems in hot-mix asphalt concrete mixtures.

6.4.1 Aggregates

Mineral aggregates are comprised of a wide variety of inorganic polar compounds and hence differ significantly with respect to the properties important to moisture susceptibility of HMA mixtures. The polarity of the compounds gives the aggregate surface a positive or negative charge (or both), the nature of which significantly affects adhesion characteristics with asphalt cement and consequently resistance to moisture damage. Aggregates with a high concentration of siliceous compounds tend to have negative surface charges in the presence of water which creates a slightly acidic solution. Conversely, aggregates with a high concentration of alkaline or alkaline earth compounds tend to have positive surface charges in the presence of water which creates a slightly basic solution. Many aggregates contain both types of compounds resulting in some sites on the aggregate surface being positively charged and other sites being negatively charged. Aggregates with an acidic nature tend to be hydrophilic (water loving) whereas aggregates with a basic nature tend to be hydrophobic (water hating). Figure 6.2 illustrates the general classification of aggregates according to the surface charge imparted by silicon dioxide content and alkaline or alkaline earth oxide content (*Hoiberg 1965*).

Aggregates that tend to be hydrophilic (acidic) in nature include granite, siliceous gravel, and quartz, whereas those that tend to be hydrophobic (basic) in nature include limestone, dolomite, and other calcareous materials. Trap rock, basalt, porphyries, and siliceous limestone are examples of aggregates with mixed surface charge (*Roberts et al. 1996*). Table 6.4 summarizes the desirable properties of aggregates for HMA mixtures (*Cordon 1979*). Hydrophilic aggregates and those with mixed surface charge can be treated with additives to enhance the adhesion characteristics of asphalt cement to these types of aggregate. Additives are covered in more detail below.

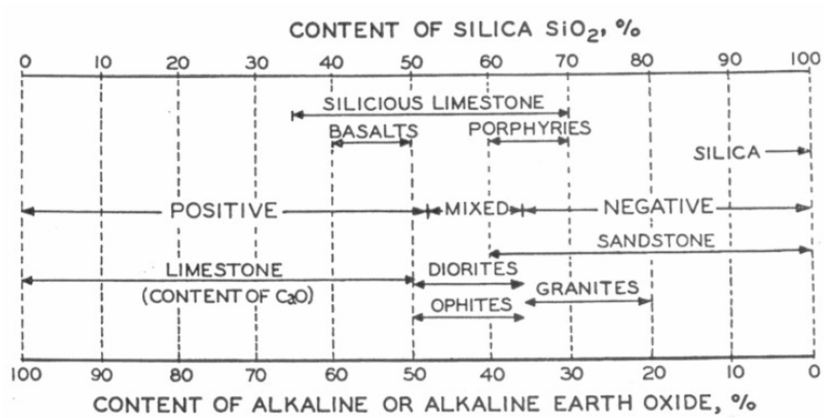


Figure 6.2: General Classification of Aggregates According to Surface Charge (*Hoiberg 1965*)

Table 6.4: Desirable Properties of Aggregates for HMA Mixtures (*Cordon 1979*).

Rock type	Hardness, toughness	Resistance to stripping†	Surface texture	Crushed shape
Igneous				
Granite	Fair	Fair	Fair	Fair
Syenite	Good	Fair	Fair	Fair
Diorite	Good	Fair	Fair	Good
Basalt (trap rock)	Good	Good	Good	Good
Diabase (trap rock)	Good	Good	Good	Good
Gabbro (trap rock)	Good	Good	Good	Good
Sedimentary				
Limestone, dolomite	Poor	Good	Good	Fair
Sandstone	Fair	Good	Good	Good
Chert	Good	Fair	Poor	Good
Shale	Poor	Poor	Fair	Fair
Metamorphic				
Gneiss	Fair	Fair	Good	Good
Schist	Fair	Fair	Good	Fair
Slate	Good	Fair	Fair	Fair
Quartzite	Good	Fair	Good	Good
Marble	Poor	Good	Fair	Fair
Serpentine	Good	Fair	Fair	Fair

† Aggregates that are hydrophilic (water-loving) tend to strip more readily since water more easily replaces an asphalt film. Freshly crushed aggregates with many broken ionic bonds also tend to strip more readily.

Aggregates need to be relatively clean to promote a good bond with asphalt cement. HMA mixtures manufactured with aggregates having soft particles, clay lumps, and excessive dust are particularly prone to moisture-related damage. The standard tests conducted to determine the cleanliness of aggregates include:

- AASHTO T 176, “Plastic Fines in Graded Aggregates and Soils by Use of the Sand Equivalent Test” (currently specified by ODOT).
- AASHTO T 122, “Clay Lumps and Friable Particles in Aggregate”
- AASHTO T 11, “Materials Finer Than 75-μm (No. 200) Sieve in Mineral Aggregates by Washing”
- AASHTO TP 57, “Methylene Blue Value of Clays, Mineral Fillers, and Fines”

6.4.2 Asphalt Cements

Asphalt cements are comprised of principally non-polar hydrocarbons but also contain polar organic compounds (organic acids and bases). In general, most asphalt cements (and particularly naphthenic asphalt cements) contain a higher proportion of acidic compounds. Although acidic asphalt cements can coat acidic aggregates (with sufficient heat and agitation), chemical bonds cannot be formed between the acidic components of asphalt cements and the acidic sites on aggregate surfaces and, hence, water can displace the asphalt cement from the aggregate surface at these sites (i.e., initiate stripping). Once the process has begun, the presence of water can accelerate stripping. ASTM D 664 and ASTM D 2896 can be used to determine the relative proportions of acids and bases, respectively, in asphalt cements.

6.4.3 HMA Mixtures

Ideally, basic asphalt cements should be used with acidic aggregates and, conversely, acidic asphalts should be used with basic aggregates. However, in many (if not most) cases, HMA is produced with a blend of locally available aggregates of different geological types and, consequently, different chemical compositions. For example, an HMA mixture produced with crushed or partially crushed river run gravel for the coarse fraction combined with a natural sand fine fraction would likely contain siliceous aggregates in either fraction. Hence, stripping could occur in either fraction assuming that the aggregates are combined with acidic asphalt cement.

Due to difficulties in matching chemical compatibility of asphalt cements and aggregates, many state highways agencies simply avoid aggregate sources known to be prone to stripping when used to produce HMA mixtures, or treat the aggregates with an anti-stripping additive (e.g., lime or liquid anti-stripping agents). Some agencies require anti-stripping additives independent of aggregate source; however, this may add unnecessary expense when required in mixtures with aggregates not prone to stripping.

Whether or not anti-stripping additives are used, most state highway agencies conduct moisture susceptibility tests on the HMA mixture. AASHTO T 283 (or modifications to this procedure) is the most commonly used test by state highway agencies, including ODOT. Several agencies use AASHTO T 165 and at least two use the Hamburg wheel tracker in conjunction with AASHTO T283.

In a recently completed project (NCHRP Project 9-34) undertaken to develop an improved method for determining the moisture susceptibility of HMA (*Solaimanian et al. 2007*), the researchers evaluated the Environmental Conditioning System (ECS) conditioning regime coupled with dynamic modulus testing in comparison with ASTM D 4867 (essentially the same as AASHTO T 283) and the Hamburg wheel tracking device. It was shown that the ECS/dynamic modulus procedure provided the best correlation with field performance. However, the researchers indicated that further work was needed to reduce the duration of the conditioning and loading regime as well as the time of conditioning and magnitude of loading before the procedure would likely be adopted for routine use.

6.4.4 Additives

Additives can be used to correct the chemical incompatibility between aggregates and asphalt cement so as to enhance the bond between the aggregate and asphalt cement and, hence, improve the resistance of the mixture to moisture-induced damage. This can be accomplished chemically by use of lime or a liquid anti-stripping additive, or physically through a polymeric aggregate treatment. Best practices for use of these materials are provided as follows:

- Lime. Quick (ordinary) lime (CaO – calcium oxide) or hydrated lime ($Ca(OH)_2$ – calcium hydroxide) can be used to coat the aggregate surface to make it alkaline such that it chemically bonds with acidic asphalt cements. The three most commonly used methods to add lime include (in order of increasing effectiveness and cost): 1) dry lime on dry aggregate, 2) dry lime on moist aggregate, and 3) lime slurry on dry aggregate. ODOT currently specifies the second method whereby dry lime is added to aggregate in a surface damp condition and mixed in a pugmill prior to mixing with asphalt cement.

- Liquid anti-stripping agents can be added to modify acidic asphalt cements so that they will chemically bond with acidic aggregates. The ODOT Standard Specifications do not explicitly disallow liquid anti-stripping agents (provided that the minimum AASHTO T 283 tensile strength ratio is satisfied), but none are listed in the Qualified Products List (QPL). The typical method for adding a liquid anti-stripping agent at an HMA plant is to inject it into asphalt cement using a metered system immediately prior to the asphalt cement entering the drum or batch mixer.

- Polymeric aggregate treatment (PAT). A polymeric aggregate treatment is used to create a physical, protective barrier on the aggregate surface that waterproofs the aggregate and improves its bond with asphalt cement. The ODOT Standard Specifications do not explicitly disallow the use of polymeric aggregate treatment (provided that the minimum AASHTO T 283 tensile strength ratio is satisfied), but none are listed in the Qualified Products List (QPL). The typical method for application of the treatment is to apply a diluted solution of the product onto the aggregate immediately before it enters the drum dryer.

7.0 CONCLUSIONS AND RECOMMENDATIONS

This document provides details of the efforts undertaken to identify sources of moisture and other conditions that led to rutting problems on four projects along Interstate 5 and one along Interstate 84 in Oregon. These efforts involved ODOT maintenance personnel interviews, records reviews, site visits, non-destructive and destructive field tests, and laboratory tests on cores obtained from the projects.

It also documents the efforts undertaken to evaluate design, construction, and materials requirements that will minimize the risk of such failures for future rehabilitation projects. These efforts involved reviewing literature concerned with moisture damage in HMA pavements, forensic investigations of HMA pavement failures, and best practices for selection and testing of HMA materials as well as for design and construction of HMA pavements, and documenting the findings.

Section 1.0 provides a brief description of the projects investigated while Sections 2.0 – 6.0 documents the efforts undertaken to satisfy the principal objectives of the study. This section provides conclusions that can be drawn from the study and recommendations for further research.

7.1 CONCLUSIONS

Based on the findings from this study, the following conclusions appear warranted:

- Moisture can weaken asphalt cement and/or the bond between the asphalt cement and the aggregate surface through several mechanisms, summarized herein, causing a reduction in strength and stiffness of HMA mixtures.

- Moisture can enter pavements by means of: a) capillary action, b) infiltration from the surface, and c) seepage from surrounding areas. Moisture can occupy pavements as liquid water or moisture vapor above the capillary fringe.

- Review of the literature revealed that the best defense against moisture-induced damage in HMA pavement structures is to provide sufficient drainage so as to prevent the accumulation and prolonged storage of water in the structure.

- Damage due to moisture in pavements usually initiates at or near the bottom of the asphalt bound layers or at interfaces between layers. Advanced moisture damage in HMA pavements can lead to rutting, shoving, corrugations, fatigue cracking, raveling, flushing, and pot holes.

- Numerous factors influence moisture damage in HMA pavements including: a) aggregate properties, b) asphalt cement properties, c) HMA mixture characteristics, d) environmental factors, e) traffic, f) construction practices, and g) design practices. A discussion of each of these factors is provided herein.

- Numerous laboratory tests, identified herein, have been developed to assess the moisture susceptibility of HMA mixtures, but none have been universally accepted as one that can correctly identify a moisture-susceptible mix in all cases. Nevertheless, AASHTO T 283 (the modified Lottman test) was the most commonly used test (as of 2002) among state highway agencies in the United States.

- Information derived from forensic investigations of distressed pavements can be very useful in determining the cause of the distress as well as aiding in the development and selection of appropriate rehabilitation strategies. Forensic investigations, while not usually necessary for pavements requiring only routine maintenance, should be considered when pavements are deteriorating more rapidly than expected, when distresses occur shortly after construction, when the cause of distress is unknown, or when disagreement regarding the distress mechanism occurs so that the root cause of the distress mechanism can be effectively mitigated.

- According to the literature reviewed, the key tasks involved in conducting forensic investigations include: a) records review, b) site observations and condition survey, c) field testing, d) laboratory testing, e) data analysis, and f) reporting the findings. Key components of each task have been included herein. Guidelines for pre-construction site investigations to identify the potential for moisture-related problems are included in Section 6.1.

- The review of the literature, particularly case studies, revealed that forensic investigations of distressed pavements commonly include both non-destructive and destructive testing techniques. Typical non-destructive techniques have included field/condition surveys, falling weight deflectometer (FWD) surveys, ground penetrating radar (GPR) surveys, Automated Road Analyzer (ARAN) surveys, and Portable Seismic Pavement Analyzer (PSPA) surveys. Destructive testing has typically included coring, dynamic cone penetration tests, and trenching operations.

- HMA overlays are presently the most widely-used method of rehabilitation of HMA pavements in the United States due their ability to provide a relatively fast and cost-effective means of improving the functional and/or structural characteristics of a pavement.

- The literature reviewed concerning best practices for the design of overlays to minimize the risk of moisture damage indicated that structural design, drainage design, and mixture design are all vital components of the overall design process. Key elements of each were identified and summarized herein. Guidelines for pavement design best practices to reduce the risk of moisture-related problems in HMA pavements are included in Section 6.2.

- The literature review revealed that moisture susceptibility of HMA mixtures can be influenced by numerous construction practices. Best practices for construction processes, with the objective of minimizing risk of moisture damage, were identified and summarized herein. Guidelines for construction best practices to reduce the risk of moisture-related problems in HMA pavements are included in Section 6.3.

- Proper material selection and testing was found to be, according to the literature reviewed, critical to obtain an HMA mixture that is resistant to moisture-induced damage. Key aspects of these endeavors were identified and summarized herein. Guidelines for materials selection and testing best practices to reduce the risk of moisture-related damage in HMA pavements are included in Section 6.4.

- Forensic investigations were conducted on four projects along Interstate 5 and one project along Interstate 84 that had exhibited significant rutting problems shortly after a rehabilitation activity. The investigations involved site visits, windshield condition surveys, personnel interviews, field investigations, laboratory investigations, analysis of the data obtained from the field and laboratory investigations, and reporting of the findings as provided herein.

- Maintenance personnel interviews and records reviews provided key information for the forensic investigations, particularly with regard to historical performance observations and maintenance activities.

- Site visits were extremely informative with regard to verifying information obtained during the maintenance personnel interviews and record reviews, determining current pavement conditions, and scouting potential locations for further evaluations such as coring and trenching activities.

- Visual examination of cores obtained from the projects provided valuable information regarding layer depths and conditions. However, the presence of moisture damage was not always identified from these observations suggesting a more thorough evaluation may be advisable.

- Trenching operations revealed key information that was not readily (or, for certain observations, could not be) obtained from cores alone.

- Observations made in the field during the trenching operations of three of the five projects (four trenches total) clearly revealed that water was seeping through the pavement structures, and in all cases, through or between lifts (layers) of dense-graded material.

- Findings from the forensic investigations provided substantial evidence to indicate that moisture damage had occurred on all of the projects investigated in this study and that such damage was the most likely cause (in combination with traffic loading) of the rutting problems observed along the projects.

- Findings from laboratory investigations generally supported those from the field investigations. Results of air void analyses of the pavement layers correlated reasonably well with observations made from the trenches in that layers with relatively high air void contents were observed to conduct water, whereas layers with relatively low air void contents did not. Similarly, results of permeability tests of the pavement layers correlated reasonably well with observations made from the trenches; that is, layers with relatively high permeability values were observed to conduct water, whereas layers with relatively low permeability values did not.

- As part of the rehabilitation activity on all projects investigated, a cold milling operation was performed to remove a portion of the existing HMA pavement prior to placement of an overlay. Such an operation exposes aggregate that had been coated with asphalt binder (assuming moisture damage had not already occurred) and consequently disrupts the bond between the asphalt binder and the aggregate affected by the milling operation, potentially providing sites for initiation of moisture damage at the milled surface. The observation of water seepage at layer interfaces between the inlay and underlying, cold-milled HMA along the Pleasant Valley – Durkee project and between the overlay and underlying, cold-milled HMA along the Vets Bridge – Myrtle Creek project points to the milling operation as being a contributing factor for the observed moisture damage in the existing (milled) HMA layers along these projects. The delaminated, dense-graded HMA inlay observed at the trenches along the Pleasant Valley – Durkee project corroborates this assertion.

- The SMA wearing course placed along the Pleasant Valley – Durkee project appeared to trap water within the pavement as evidenced by water seepage at the interface between the SMA layer and underlying dense-graded HMA layer. In addition, the SMA layer delaminated from the dense-graded HMA layer during excavation of the trench, providing evidence of water damage at the interface. Water seepage out of construction joints was also observed in the vicinity of the trenches. These observations indicate that placing an overlay with lower permeability than the underlying layer(s) likely impedes rapid drainage of water from the underlying layer(s), thus increasing the time water remains within the pavement and thereby increasing the opportunity for the occurrence of moisture damage.

- Tests on the core slices of the open-graded overlay placed along the Garden Valley – Roberts Creek project indicated an air void content of approximately 8-10%, well below the design content of 13½-16% and essentially the same as that of the underlying dense-graded mixture. The permeability of the open-graded mixture was also similar in magnitude to that of the underlying dense-graded mixture. Observations made while the trench along this project was open indicated that the open-graded mixture appeared to have about the same moisture content as that of the underlying dense-graded mixture. These observations indicate that the open-graded mixture was not functioning as a free-draining layer and was likely promoting retention of water within the pavement layers.

- Overall, improper tack coat or failure, permeable dense-graded layers, inadequate drainage, and, possibly, inadequate compaction of dense-graded material, were all identified as the likely root causes of the observed moisture damage and consequential rutting problems on the projects that were trenched.

7.2 RECOMMENDATIONS

Based on the findings from this study, the following recommendations appear warranted:

- Although Chapter 4 of the ODOT Pavement Design Guide (*2007*) provides guidance regarding pre-construction site investigations, the guidelines contained in Section 6.1 of this document, which are geared toward minimizing the risk of moisture damage in rehabilitated pavements, should be implemented as part of routine investigation activities. In particular, a more extensive evaluation of cores, which are routinely obtained, should be implemented to detect moisture damage in the existing pavement layers (see Section 6.1.4). In this regard, further research into the development and/or implementation of a suitable permeability test should be undertaken. Also, trenches should be seriously considered as a routine investigative technique. Finally, GPR surveys were shown to provide valuable information regarding the thickness of pavement layers and locations of non-uniform pavement that could be indicative of moisture damage. Although conclusive validation of the use of a GPR survey for detecting moisture-damaged pavement was not obtained in this study, the technique should be further investigated for routine use during pre-construction site investigations.

- The guidelines for structural design techniques presented in Section 6.2 of this document were developed to supplement the guidance provided in the ODOT Pavement Design Guide (*2007*) with particular emphasis on rehabilitation strategy selection and design techniques to minimize the risk of moisture damage in rehabilitated pavements. Hence, it is recommended that the guidelines contained herein be integrated into the ODOT Pavement Design Guide (*2007*). In addition, given that water was found to be seeping through existing, dense-graded HMA layers, ODOT should investigate the efficacy of installing surface drains within the existing, milled surface (similar in concept to wearing course surface drains) spaced at regular intervals along a project to provide transverse drainage paths for water that tends to flow longitudinally through the pavement.

- The guidelines for construction techniques presented in Section 6.3 were developed to assist ODOT personnel who are responsible for oversight of construction activities during the rehabilitation of HMA pavements with emphasis on techniques that reduce the risk of failure in the rehabilitated pavement due to moisture damage. The guidelines not already routinely implemented by ODOT should be adopted. Given the findings documented herein regarding water seepage between paving layers, ODOT personnel should pay particular attention to ensuring that the surface to receive an overlay is properly milled (cold planed), properly cleaned, and that the tack coat is correctly applied. Implementation of this latter recommendation should include a review of the specifications for milling and cleaning the surface and for application of the tack coat to ensure that these are adequate, as well as ensuring these activities are performed to specification through inspection activities. In addition, given that a major source of water within a pavement is from water infiltration through its surface and that longitudinal butt joints (currently specified by ODOT) are notoriously difficult to compact to the same density as the interior portion of an HMA mat, it is likely that longitudinal construction joints have higher permeability than the denser, interior portion of the mat and, thus, more readily conduct water into the pavement. Hence, it is recommended that ODOT investigate the use of lap joints as a possible alternative to butt joints, or investigate the use of a permeability criterion for specifying longitudinal butt joint quality.

- The guidelines for materials selection and testing techniques contained in Section 6.4 were developed to assist ODOT materials engineers select materials for HMA mixtures that will reduce the risk of moisture-related problems in rehabilitated HMA pavements that incorporate the materials. Although ODOT currently implements the spirit of the guidelines presented herein, the use of the methylene blue (AASHTO TP 57) test should be investigated as a potential replacement for the sand equivalent (AASHTO T 176) test due to the improved ability of the methylene blue test in identifying fine aggregates that have the propensity for causing stripping in HMA pavements. Similarly, ODOT should investigate the use of the ECS/dynamic modulus test method developed under NCHRP Project 9-34 as a potential replacement for the modified Lottman (AASHTO T 283) test due to the improved correlation of the ECS/dynamic modulus test results with field performance.

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