CALIBRATION OF THE AASHTO PAVEMENT DESIGN GUIDE TO SOUTH CAROLINA CONDITIONS – PHASE I

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14. Abstract

The primary objective of this research was to identify existing historical data (i.e., climate, traffic, pavement design information, material properties, and pavement performance) within the SCDOT for use in the local calibration of the Mechanistic Empirical Pavement Design Guide (MEPDG) for South Carolina. Based on all of the compiled information, 20 in-service pavement sections (i.e., 14 asphalt concrete (AC) sections and 6 Portland cement concrete (PCC) sections) were identified as suitable for this study, and information gaps were identified. A comprehensive subgrade sampling and testing plan was developed, and field and laboratory tests were performed to study the subgrade modulus at 3 sites. The data collected for the 20 sections was used to perform a preliminary analysis of the MEPDG AC rutting models, AC fatigue cracking models, AC transverse cracking model, and the JPCP transverse cracking model. Level 1 (project specific), Level 2 (region specific), or Level 3 (default) inputs were used, depending on data availability. The preliminary calibration factors found from the analysis should not be used for design until further studies are performed. A plan for future phases, including the additional high priority, high quality data needed to complete the MEPDG analysis for the local calibration and a plan for special pavement test sections, is proposed.

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EXECUTIVE SUMMARY

This report presents the findings from a study undertaken to identify existing historical data within the South Carolina Department of Transportation (SCDOT) for use in the local calibration of the Mechanistic Empirical Pavement Design Guide (MEPDG) for South Carolina (SC). Priority was given to identifying and reviewing pavement performance data collected from high traffic primary and interstate routes across SC. The review process focused on pavements constructed between 1985 and 2000 to best represent SCDOT's current design, materials, and construction practices. Historical data for both asphalt concrete (AC) and Portland cement concrete (PCC) pavement sections located within the SCDOT Office of Materials and Research, Division of Traffic Engineering, and Division of Maintenance were reviewed, and information gaps were identified. The existing historical data found to be compatible with the MEPDG protocol were compiled and 20 in-service pavement sections - 14 AC sections with lengths ranging from 1.0 to 24.35 miles and 6 PCC sections with lengths ranging from 1.47 to 14.17 miles - were selected from 15 counties. The major categories of data include climate, traffic, pavement structure and materials, and pavement performance. For 3 of these sections (i.e., 1 in the Piedmont Region and 2 in the Coastal Plain), field sample collection, Falling Weight Deflectometer tests, soil classification, and resilient modulus tests were performed to determine project specific material inputs.

The data collected for the 20 pavement sections was used to perform a preliminary analysis of the MEPDG AC rutting models, AC fatigue cracking models, AC transverse cracking model, and the JPCP transverse cracking model. Inputs for the analysis were from all 3 hierarchical categories: Level 1 (project specific), Level 2 (region specific), and Level 3 (national or default values). Level 2 and Level 3 inputs were used for many of the material property inputs due to their unavailability in the SCDOT files and databases for the selected 20 pavement sections. SCDOT measures IRI, rutting, fatigue cracking, longitudinal cracking, and transverse cracking for AC pavements; however, the cracking and rutting data cannot be implemented into MEPDG with the highest confidence level because bottom-up and top-down cracking are not clearly distinguished by their visual inspection procedure and only the total rut depth is measured. Because not all of the necessary data was available in the SCDOT files and databases, and the quality of the distress data is uncertain, the local calibration factors presented herein are preliminary, and are <u>not recommended to be used for design</u> until further research is performed in a Phase II study to obtain high quality, high priority data.

Tasks that need to be performed as part of a Phase II study before the MEPDG local calibration can be performed for SC with confidence include: (1) Identify additional pavement sections (i.e., AASHTO (2010) recommends using data from 30 pavement sections to calibrate load related cracking models); (2) Collect distress survey data and perform trench studies (i.e., distinguish between top-down and bottom-up cracking and measure rut depth of individual pavement layers); (3) Collect high priority materials data for AC, PCC and unbound base pavement layers; (4) Install portable WIM stations to obtain load spectra; and (5) Study the seasonal variation of subgrade modulus. Then, after the models are calibrated for SC conditions in a Phase II study, a Phase III study will need to be performed to identify additional pavement sections to validate the models. Fully instrumented test sections will need to be constructed to monitor long-term pavement performance.

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CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

The pavement design method currently used by the South Carolina Department of Transportation (SCDOT) is based on updates to the original 1961 procedure and South Carolina (SC)-specific local calibration studies conducted at University of South Carolina (USC) and Clemson University from approximately 1964 to 1973. However, the original procedure was never intended for very high volumes of truck traffic and new materials (e.g., polymer-modified asphalt binders introduced in the 1990s and later). As a result, the pavement design procedures being used today are not necessarily accurate for certain conditions. It is believed that the current design method overestimates the pavement thickness necessary for high-volume interstate traffic and does not fully account for the benefits of new materials.

In 2008, the American Association of State Highway and Transportation Officials (AASHTO) released the first all-new pavement design method (i.e., the *Mechanistic-Empirical Pavement Design Guide* (MEPDG)). The new design method was developed using data from the Strategic Highway Research Program (SHRP) Long-Term Pavement Performance (LTPP) study started in the mid-1980s. This design method requires the engineer to enter data for traffic, climate, materials characteristics, and a proposed pavement structure into a computer program through one of the three hierarchical levels. The program then makes forecasts of various distresses over the design life of the pavement and the engineer can then decide if the pavement performance is satisfactory. The models used within the program were calibrated using a national database of pavement performance. Because the calibration included data from areas that have significant differences in materials, climate, and construction practices from SC, the procedure may not be accurate for SC conditions. For this reason, AASHTO strongly urges states that use the new procedure to perform local calibration and has designed the pavement design software to be adjustable for local conditions.

In 2010, State Planned Research Project 671, entitled "*Mechanistic/Empirical Design Guide Implementation*", was completed. The final report (Baus and Stires 2010) recommended, among other things, to locally calibrate and validate distress predictions through the establishment of a minimum of 20 pavement test sections. The report also recommended that a comprehensive study be conducted to determine in-situ modulus values for SC subgrade soils and provide comprehensive information on seasonal variation. Anecdotal observation of pavement performance predictions on well-established flexural pavement designs has indicated that the predictions greatly overestimate the amount of permanent deformation compared to SCDOT experience. Consequently, special emphasis on the subgrade modeling contained within the AASHTO procedure and its applicability to SC conditions is highly desired.

Following the above problem statement, this research aims to reduce design bias and increase precision of the model predictions by calibrating the new AASHTO pavement design guide with full consideration of SC local conditions. The research presented herein is the first phase of a multi-phase study to achieve this goal.

1.2 RESEARCH OBJECTIVES AND DELIVERABLES

The primary objective of this research was to identify data within the SCDOT (i.e., climate, traffic, pavement design information, material properties, and pavement performance) for calibration of the MEPDG procedure for new flexible and rigid pavements. Based on this data, in-service pavement sections suitable for calibration studies were selected, and information gaps were identified. This research focused on higher traffic primary and interstate routes rather than low volume local sections. A comprehensive subgrade sampling and testing plan was developed, and field and laboratory tests were performed to study the subgrade modulus. Performance analyses were conducted on the identified in-service test sections using Level 1 (project specific), Level 2 (region specific), or Level 3 (default) inputs, depending on data availability. Preliminary calibration factors were found using limited amounts of data. High priority data needs were identified. A plan for future phases, including the additional data needed to complete the MEPDG analysis for the local calibration and a plan for special pavement test sections, is proposed.

1.3 SPECIFIC RESULTS AND POTENTIAL BENEFITS

It is anticipated that the information from this research will be used for a Phase II project where the identified sites are sampled and analyzed to determine their actual material characteristics and field performance. A Phase III study would then use the data collected in Phases I and II to perform the calibration and propose the final adjustments to the AASHTO design method for use by SCDOT. The potential benefit of this research is to enable the SCDOT to better allocate the billions of dollars it spends on pavement through more precise pavement designs than are currently used.

1.4 IMPLEMETATION

Because this is a Phase I study, it will not be immediately implementable. However, after Phases II and III are conducted, the resulting calibration will provide the basis for pavement design by fully considering SC conditions.

1.5 PROJECT TASKS

To meet the Phase I project objectives, the project was divided into 7 work tasks spanning a 36-month period. The tasks are listed as follows:

Task 1. Project kick-off meeting
Task 2. Review and identify pavement performance data within SCDOT
Task 3. Identify in-service pavement sections and develop a calibration plan
Task 4. Perform analysis on the identified test sections and prioritize data needs
Task 5. Propose special pavement test sections
Task 6. Study subgrade modulus and develop a subgrade sampling and testing plan
Task 7. Final project report and meeting

1.6 ORGANIZATION OF THE REPORT

This report includes 6 chapters. Following the introduction to the project presented here in Chapter 1, the findings from Tasks 2 and 3 are presented in Chapter 2. All of the historical data for climate, traffic, materials and structure, and pavement performance that were discovered in the SCDOT files and found to be compatible with the MEPDG protocol were compiled and twenty pavement sections were selected for in-depth study. The field and laboratory investigations that were performed on 3 of the 20 pavement sections to characterize the subgrade soil (Task 6) are also presented. The hierarchical level for each data input is put forth. Chapter 3 presents the preliminary MEPDG analyses for the 20 identified in-service pavement sections using global calibration factors and the data available from Task 2. The preliminary analysis was performed for the AC rutting models, AC fatigue cracking models, AC transverse cracking model, and the JCPC transverse cracking model. Because the MEPDG analyses using global calibration factors showed extensive bias for all the distress prediction models except AC transverse cracking, preliminary local calibration was performed. From these results, preliminary local calibration coefficients for the pavement distress models were determined and are presented in Chapter 4. Conclusions and recommendations are presented in Chapter 5. A plan for future phases, including the additional data needed to complete the MEPDG analysis for the local calibration and a plan for special pavement test sections (Task 5), is presented in Chapter 6. Recommendations for instrumentation and field and laboratory testing needs are presented.

CHAPTER 2

REVIEW AND IDENTIFY DATA FOR MEPDG

2.1 INTRODUCTION

Existing historical data within the SCDOT was identified and reviewed as part of Phase I to implement the new pavement design procedures of the Mechanistic Empirical Pavement Design Guide (MEPDG) for South Carolina (SC). Priority was given to identifying and reviewing pavement performance data collected from high traffic primary and interstate routes across SC. The review process focused on pavements constructed between 1985 and 2000 to best represent SCDOT's current design, materials, and construction practices. Historical data for both asphalt concrete (AC) and Portland cement concrete (PCC) pavement sections located within the SCDOT Office of Materials and Research, Division of Traffic Engineering, and Division of Maintenance were reviewed. The existing historical data found to be compatible with the MEPDG protocol were compiled and twenty pavement sections were selected for in-depth study. The major categories of the compiled data include climate, traffic, pavement structure and materials, and pavement performance. The compiled data will be used for preliminary MEPDG analysis with global calibration factors in Chapter 3, preliminary MEPDG analysis with local calibration factors in Chapter 4, and ultimately to develop a comprehensive calibration plan that will be implemented as part of a Phase II study.

2.2 IDENTIFY IN-SERVICE PAVEMENT SECTIONS

A minimum of 20 pavement test sections was recommended by Baus and Stires (2010) for calibrating and validating distress predictions. To select the pavement sections for this project, the following guidelines were considered.

1. The pavement sections are primary or interstate routes located in Coastal Plain and Piedmont Regions in SC.
2. Both flexible and rigid pavements with typical layer configuration and material selection, including traditional and new materials, are included.
3. Different service times for different types of pavements are included.
4. Priority is given to the initially selected sections with historical data, including climate, materials, traffic, and performance data.
5. Selected sections are not overlaid or rehabilitated, and are suitable for MEPDG local calibration.

Pavement sections were selected by carefully reviewing the pavement design files available at the SCDOT Office of Materials and Research and consulting with the SCDOT Pavement Design Group. For each pavement design file considered to be a potential candidate for analysis, the cross sections and pavement structure information were collected from the plan library of the SCDOT intranet. The SCDOT intranet also provided the traffic open dates and pavement condition data from the Director Card File and Pavement Viewer, respectively. After reviewing the data from the Office of Materials and Research, and SCDOT intranet, and conducting frequent meetings with the SCDOT Pavement Design Group, 20 pavement sections were selected from 15 counties in SC to serve as a representative sample for MEPDG analysis—14 AC sections with lengths ranging from 1.0 to 24.35 miles (average length of 5.3 miles) and 6 PCC sections with lengths ranging from 1.47 to 14.17 miles (average length of 5.8 miles). Table 2.1 lists the selected pavement sections with their location, pavement type, length, let date (i.e., traffic opening date), and design file number. Note that two additional sections on I 85 (per Design File No. 4.117B: north bound travel lane is an AC section and the south bound travel lane is a PCC section) were identified as potential candidates for study due to availability of WIM and IRI information; however, these sections were not used herein because of insufficient data in other categories.

Table 2.1 Selected Pavement Sections

County	Location	Type	Length (miles)	Let date	Design File No.
Aiken	I-520	PCC	5.35	7/25/2008	2.140B
Beaufort	US-278	AC	1.56	3/13/1998	7.558
Charleston	SC-461	AC	2.48	5/21/1996	10.195A
Charleston	I-526	PCC	2.39	6/25/1991	810.482
Chester	SC-9	AC	7.12	10/1/1999	12.606
Chesterfield	SC-151	AC	5.36	12/15/1999	13.585
Fairfield	I-77	PCC	14.17	10/21/1980	20.437
Florence	SC-327	AC	5.09	2/25/1992	21.873
Florence	US-301	AC	2.38	9/30/2003	21.147A
Georgetown	US-521	AC	4.07	6/1/2003	22.619
Greenville	I-385	AC	7.65	8/28/2000	23.038621
Greenville	I-85	AC	1.00	8/31/2005	23.474A
Horry	SC-22	AC	24.35	10/12/2001	26.856
Horry	SC-31	AC	3.98	1/31/2005	26.986
Laurens	SC-72	AC	5.99	3/1/2002	30.694
Lexington	S-378	PCC	1.47	11/1/2001	32.128A
Orangeburg	US-321	AC	6.17	7/1/2004	38.157A
Pickens	SC-93	AC	1.34	4/10/2001	39.730
Spartanburg	SC-80	PCC	3.30	6/1/2000	42.108B
Spartanburg	I-85	PCC	6.29	6/11/1997	42.146A.1

Note: I, US, and SC represent Interstate highways, United States routes, and South Carolina routes, respectively.

The locations of the pavement sections are shown in Figure 2.1 and represent both the Piedmont Region and Coastal Plains of SC. Soils in SC have been divided into two regions separated by the geological fall line: (i) Upstate Area or Blue Ridge and Piedmont Region (Type A), and (ii) Coastal Plain and Sediment Region (Type B) (SCDOT, 2010). Type A soils are described as micaceous clayey silts and micaceous sandy silts, clays, and silty soils in partially drained condition; Type B soils include fine sand that is difficult to compact. In terms of AASHTO classifications, Type B soils are primarily A-1 to A-4 and Type A soils are predominately A-5 or higher (Pierce et al., 2011). The AASHTO system classifies soils into eight groups: A-1 through A-8 where A-1 to A-3 are granular soils, A-4 to A-7 are fine grained soils, and A-8 represents organic soils (AASHTO M 145-03).

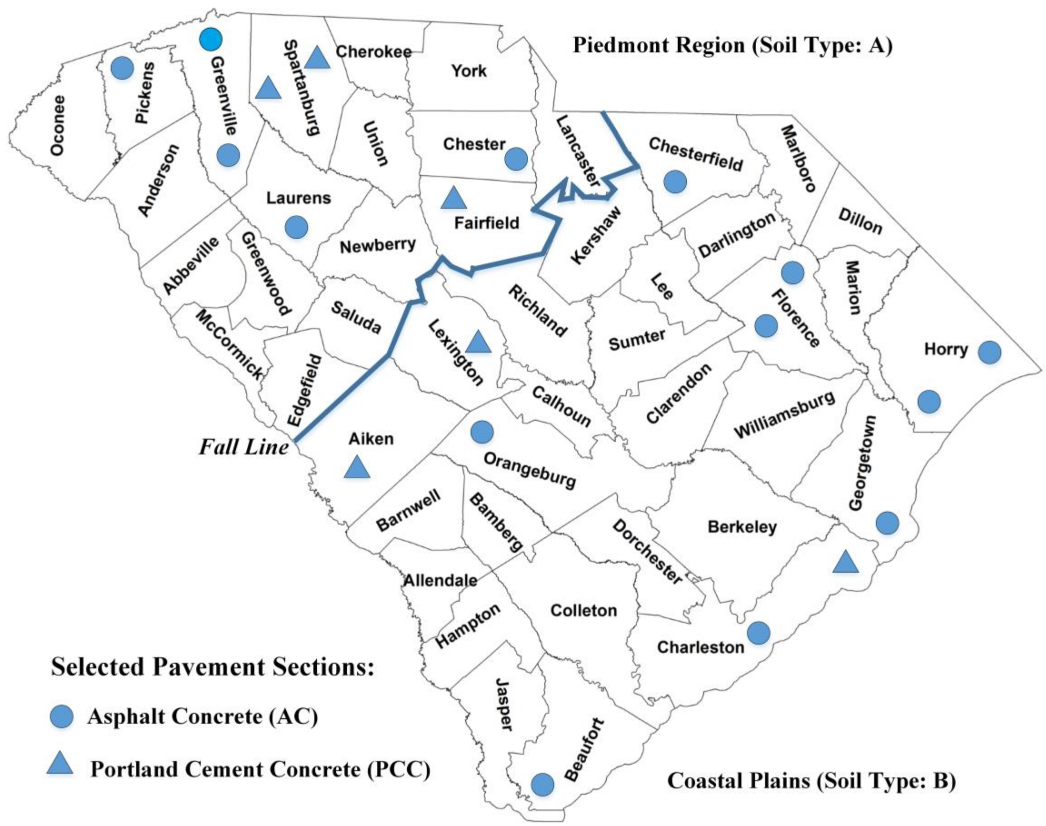


Figure 2.1 Selected Pavement Sections

Once the 20 in-service pavement sections were selected as candidates for pavement analysis, the sites were visited to verify the condition of each section. Site visits were performed to answer the following questions:

- Does the pavement section have the same pavement type (AC or PCC) as mentioned in the design file?
- Does the let date or the date of construction match with the pavement condition after visual inspection?

- Is it a new flexible or rigid pavement? Is there any overlay or reconstruction?

A summary of the findings is presented in Appendix A. Most of the selected pavement sections were found as new flexible or rigid pavements with no overlay, and the let date and pavement type matched the design file. Two exceptions included SC 9 from Chester county that looked newer than its let date and US 301 from Florence county that showed overlay on some of its segments. However, both pavement sections were still considered for the analysis because, based on a review of the pavement distress data over last few years, both pavement sections showed a decreasing trend in pavement condition up to 2014. It is possible that the overlay/or resurfacing took place very recently which would not affect the analysis because distress data after the resurfacing were not considered. Hence, as none of the 20 pavement sections showed any major disagreement with the design files, all were selected for the detailed data collection and MEPDG analysis.

2.3 REVIEW AND IDENTIFY DATA WITHIN SCDOT FOR MEPDG

MEPDG requires great quantity and quality of input data in four major categories: climate, traffic, materials, and pavement performance. Existing historical data within the SCDOT in these four categories were reviewed and have been compiled herein for each of the 20 selected pavement sections.

2.3.1 Climate

In its current version, the MEPDG program (AASHTOWare, 2016) includes weather station data across the U.S.; including 12 stations in SC. Pavement sections were assigned to weather stations located in the same county or in the nearest adjacent county. If no weather stations were available in the same county or in the adjacent county, or two weather stations were available in two adjacent counties, then a virtual weather station was created by averaging two weather stations located in the two nearest counties on opposite sides of the pavement section. Weather stations outside SC were not considered. The assigned weather stations for each pavement section are shown in Table 2.2.

Table 2.2 Weather Stations Assigned to the Selected Pavement Sections

County	Location	Type	Weather Station	Type
Aiken	I-520	PCC	Orangeburg, Columbia	Virtual
Beaufort	US-278	AC	Charleston	Actual
Charleston	SC-461	AC	Charleston	Actual
Charleston	I-526	PCC	Charleston	Actual
Chester	SC-9	AC	Rock Hill	Actual
Chesterfield	SC-151	AC	Rock Hill, Florence	Virtual
Fairfield	I-77	PCC	Rock Hill, Columbia	Virtual
Florence	SC-327	AC	Florence	Actual
Florence	US-301	AC	Florence	Actual
Georgetown	US-521	AC	Charleston, North Myrtle Beach	Virtual
Greenville	I-385	AC	Greenville	Actual
Greenville	I-85	AC	Greenville	Actual
Horry	SC-22	AC	North Myrtle Beach	Actual
Horry	SC-31	AC	North Myrtle Beach	Actual
Laurens	SC-72	AC	Greer	Actual
Lexington	S-378	PCC	Columbia	Actual
Orangeburg	US-321	AC	Orangeburg	Actual
Pickens	SC-93	AC	Clemson	Actual
Spartanburg	SC-80	PCC	Greer	Actual
Spartanburg	I-85	PCC	Greer	Actual

2.3.2 Traffic

Traffic data required by MEPDG includes: vehicle classification distribution, truck volume, number of axles per truck, axle configuration, axle load distribution factors, lateral traffic wander, hourly and monthly traffic volume adjustment factors, and traffic growth factors. In SC, traffic data is collected using Automatic Traffic Recorders (ATRs) and Weigh-in-Motion (WIM) stations. There are more than 100 active ATRs in SC that are monitored by the SCDOT. There are 2 WIM stations that are regularly monitored by the State Transport Police of the SC Department of Public Safety (SCDPS).

For this project, the primary source of traffic data was the SCDOT Division of Traffic Engineering. They provided the traffic data collected through traffic counts by ATRs. ATRs provide historical traffic counts, real-time counts and average speeds on the highway system in SC, but do not provide Weigh-in-Motion (WIM) data or the axle load spectra. The ATR data was primarily used to extract vehicle class distribution data. The vehicle class distribution data that was of acceptable quality (agreed with the predicted traffic in the construction file) were compiled into a standardized format according to the new MEPDG requirements and presented in Table 2.3. This data is used as Level 1 input. For the missing traffic data (i.e., hourly and monthly traffic distribution), default values calibrated on the national level (i.e., Level 3) were used.

Table 2.3 Vehicle Class Distribution by Percentage

Location	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
I-520*	1.33	21.00	0.00	1.33	6.00	66.00	1.33	1.33	1.33	1.33
US-278	4.24	60.37	14.80	1.94	14.60	3.63	0.09	0.32	0.00	0.02
SC-461	13.15	39.24	10.62	0.88	31.18	3.47	0.09	1.21	0.03	0.13
I-526*	1.33	21.00	0.00	1.33	6.00	66.00	1.33	1.33	1.33	1.33
SC-9	5.12	41.77	7.35	0.91	10.82	32.87	1.03	0.11	0.01	0.00
SC-151*	1.33	21.00	0.00	1.33	6.00	66.00	1.33	1.33	1.33	1.33
I-77*	1.33	21.00	0.00	1.33	6.00	66.00	1.33	1.33	1.33	1.33
SC-327	4.16	14.50	17.98	3.77	8.84	41.96	5.14	1.47	0.59	1.58
US-301*	1.17	44.00	8.00	1.17	5.00	36.00	1.17	1.17	1.17	1.17
US-521	3.41	28.56	5.51	1.95	15.16	44.85	0.58	0.00	0.00	0.00
I-385*	1.33	21.00	0.00	1.33	6.00	66.00	1.33	1.33	1.33	1.33
I-85*	1.33	21.00	0.00	1.33	6.00	66.00	1.33	1.33	1.33	1.33
SC-22	6.36	78.51	2.76	0.13	8.77	3.15	0.00	0.31	0.00	0.00
SC-31	7.98	69.67	2.33	0.03	14.21	4.67	0.09	1.00	0.03	0.00
SC-72	3.57	6.81	50.57	1.22	4.22	12.94	12.94	0.08	0.04	7.62
S-378	4.52	74.84	4.55	0.27	9.75	5.82	0.24	0.00	0.03	0.00
US-321	4.62	30.15	7.49	1.56	13.88	38.45	2.64	0.52	0.07	0.61
SC-93	4.89	28.99	10.59	6.40	8.38	9.08	27.47	0.47	0.35	3.38
SC-80	9.30	61.90	8.04	0.48	16.99	2.61	0.00	0.59	0.00	0.09
I-85*	1.33	21.00	0.00	1.33	6.00	66.00	1.33	1.33	1.33	1.33

Note: *Vehicle class distribution from historic road group was used

For MEPDG, vehicle class distribution of the 10 vehicle classes are required (Class 4-Class 13). Of the 20 pavement sections, vehicle class distribution data was available from the SCDOT Division of Traffic Engineering for only 12 sections and not available for the other 8 sections. Therefore, vehicle class distribution data for different historic road groups were used for those 8 sections. Road groups are derived based on the vehicle classification data of corresponding road sections. Truck type distributions are provided for 16 road groups (Road Group A to Road Group P) per the load data table in the SCDOT Pavement Design Guideline (SCDOT, 2008). However, the percent trucks by class are only divided for Class 5, Class 6, Class 8, and Class 9; all other classes are combined (Table 2.4). Therefore, to convert those data to MEPDG format, the percent trucks by class data grouped as "all others" were equally distributed to the missing classes.

Table 2.4 Truck Type Distribution for Various Road Groups per SCDOT (2008)

Road Group	% Trucks By Class				
	Class 5's	Class 6's	Class 8's	Class 9's	All Others
A	94	0	0	0	6
B	90	5	0	4	1
C	81	5	5	7	2
D	73	6	6	10	5
E	68	6	8	12	6
F	64	6	7	15	8
G	59	8	5	19	10
H	54	6	7	25	9
I	48	7	5	31	8
J	44	8	5	36	7
K	40	7	6	41	7
L	33	7	6	49	6
M	27	7	6	55	5
N	24	3	6	60	7
O	21	0	6	66	8
P	12	3	4	72	9

Average Annual Daily Truck Traffic (AADTT) data for the selected pavement sections were compiled for MEPDG analysis and summarized in Table 2.5. The data includes two way

AADTT, number of lanes, percent trucks in the design direction, percent trucks in the design lane, operational speed in the roadway, and traffic compound growth rate. Growth rate was estimated from the base year traffic and predicted future traffic found in the pavement design files. Two-way AADTT was estimated from the base year average annual daily traffic (AADT) and the percent truck information collected from the Traffic Data for Pavement Loading files obtained from the SCDOT Office of Materials and Research.

Table 2.5 Average Annual Daily Truck Traffic (AADTT) Data

County	Location	Two-way AADTT	No. of Lanes	% Trucks Design Direction	% Trucks Design Lane	Operational Speed (mph)	Growth Rate (%)
Aiken	I-520	2150	2	50	80	60	2.36
Beaufort	US-278	915	2	50	80	55	2.76
Charleston	SC-461	2400	2	50	80	55	2.00
Charleston	I-526	2700	2	50	80	60	2.36
Chester	SC-9	640	2	50	80	45	2.70
Chesterfield	SC-151	516	2	50	80	55	3.12
Fairfield	I-77	2000	2	50	80	60	2.00
Florence	SC-327	710	2	50	80	60	2.00
Florence	US-301	1144	1	50	100	55	2.48
Georgetown	US-521	368	2	50	80	60	3.03
Greenville	I-385	12000	3	50	65	65	2.00
Greenville	I-85	15440	1	50	100	60	2.00
Horry	SC-22	1770	2	50	80	65	2.00
Horry	SC-31	1520	3	50	65	65	3.11
Laurens	SC-72	472	2	50	80	55	2.36
Lexington	S-378	736	2	50	80	45	2.00
Orangeburg	US-321	720	2	50	80	55	1.86
Pickens	SC-93	490	2	50	80	35	2.06
Spartanburg	SC-80	888	2	50	80	55	2.60
Spartanburg	I-85	20303	2	50	80	70	4.26

Table 2.6 AADTT Estimated from the Base Year AADT

County	Location	Base Year AADT	% Truck	AADTT
Aiken	I-520	21500	10	2150
Beaufort	US-278	18290	5	915
Charleston	SC-461	24000*	10	2400
Charleston	I-526	22500	12	2700
Chester	SC-9	6400	10	640
Chesterfield	SC-151	4300	12	516
Fairfield	I-77	20000*	10	2000
Florence	SC-327	7100	10	710
Florence	US-301	11440	10	1144
Georgetown	US-521	2300	16	368
Greenville	I-385	60000	20	12000
Greenville	I-85	77200	20	15440
Horry	SC-22	29500	6	1770
Horry	SC-31	19000	8	1520
Laurens	SC-72	5900	8	472
Lexington	S-378	9200	8	736
Orangeburg	US-321	7200	10	720
Pickens	SC-93	9800	5	490
Spartanburg	SC-80	11100	8	888
Spartanburg	I-85	58009	35	20303

Note: *Base year AADT was back-calculated from the current year AADT

The AADTT data are summarized in Table 2.6 and were estimated from the base year AADT data. Base year AADT was not found in the title sheet of the design file for SC 461 of Charleston County and I 77 of Fairfield County. Therefore, base year AADT was back predicted from the current year AADT and the growth rate for those two sections.

WIM data were collected from the SCDPS State Transport Police for the two active stations: one is in Townville (Anderson County) on I-85 N, mile marker 9 and the other is in St. George (Dorchester County) on I-95 N, mile marker 74. For these two stations, WIM data were collected for the last five years for different single, tandem, tridem and quadrem axles. Neither of these two stations is located near any of the selected 20 pavement sections; however, weight data from these two stations were used to produce Level 2 (i.e., state specific) WIM data. The average axle load spectra for different number of axles were estimated using WIM data collected from years 2011 to 2015.

The average axle load distributions for the single axle are shown in Table 2.7(a), and for the tandem and tridem axle in Table 2.7(b) and Table 2.7(c), respectively. No quadrem axle was experienced at the WIM stations during the time period. Therefore, no distribution was used for quadrem axle. The data is shown as a function of class as per the required MEPDG format. However, the collected WIM data were provided as ranges of different classes (e.g. class 2-4) which required the data to be distributed among each class (e.g., class 2, class 3, and class 4). Moreover, the collected WIM data was reported in terms of the total number of vehicles, thus the data was converted to percentages as required for MEPDG.

Table 2.7 (a) Axle Load Distribution for the Single Axle

Weight (kips)	Single Axle Load Distribution									
	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
0 - 2	0.89	0.03	0.00	1.55	17.79	0.79	0.03	0.04	16.56	4.37
2 - 4	0.75	42.12	0.00	2.72	20.62	8.67	0.26	0.12	16.39	6.51
4 - 6	0.21	22.50	0.00	6.60	13.66	11.78	7.95	6.37	12.37	7.34
6 - 8	3.38	12.78	0.00	8.80	10.18	7.60	18.65	12.02	12.71	5.60
8 - 10	19.28	9.43	0.00	7.37	8.51	6.16	16.02	12.42	9.20	7.26
10 - 12	28.50	5.39	0.00	8.15	7.09	6.20	22.07	19.44	7.69	9.88
12 - 14	19.20	3.46	0.00	14.88	7.08	7.47	20.60	20.02	8.03	14.48
14 - 16	13.35	2.05	0.00	17.08	6.00	15.14	9.85	15.33	6.19	17.86
16 - 18	8.10	1.18	0.00	13.84	4.51	25.90	3.42	9.07	4.52	12.50
18 - 20	4.26	0.67	0.00	11.77	3.17	9.16	0.91	4.20	3.68	6.63
20 - 22	1.50	0.27	0.00	4.79	1.14	0.97	0.19	0.90	1.17	3.61
22 - 24	0.38	0.08	0.00	1.29	0.21	0.12	0.04	0.09	1.00	2.02
24 - 26	0.11	0.02	0.00	0.91	0.03	0.03	0.01	0.00	0.33	1.55
26 - 28	0.05	0.01	0.00	0.00	0.01	0.01	0.00	0.00	0.17	0.24
28 - 30	0.02	0.00	0.00	0.26	0.00	0.00	0.00	0.00	0.00	0.08
30 +	0.03	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.08

Table 2.7 (b) Axle Load Distribution for the Tandem Axle

Weight (kips)	Single Axle Load Distribution									
	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
0 - 2	0.00	0.00	0.00	0.12	0.76	0.00	0.00	1.37	2.19	0.15
2 - 4	0.00	0.00	0.04	0.04	6.88	0.01	0.01	3.15	8.59	1.53
4 - 6	0.03	0.00	3.05	0.04	16.86	0.19	0.03	6.85	6.23	4.01
6 - 8	1.51	0.00	35.83	0.16	13.63	0.92	0.18	8.08	4.88	5.39
8 - 10	3.60	0.00	26.09	0.36	12.14	4.34	3.40	13.84	2.19	4.72
10 - 12	2.61	0.00	6.62	0.93	14.54	8.16	10.05	9.32	3.87	4.63
12 - 14	3.01	0.00	7.44	1.81	9.63	7.94	8.83	7.81	4.55	7.84
14 - 16	2.76	0.00	4.87	4.72	6.79	6.54	10.28	5.34	4.38	7.41
16 - 18	2.66	0.00	2.27	6.25	5.23	6.05	16.45	2.60	4.55	11.27
18 - 20	2.67	0.00	1.44	7.54	3.73	5.96	20.08	4.79	5.39	10.02
20 - 22	3.77	0.00	1.38	8.19	2.85	5.52	12.02	4.25	4.21	7.66
22 - 24	6.62	0.00	1.44	9.92	2.46	5.09	6.41	5.48	3.54	4.11
24 - 26	13.14	0.00	1.51	11.53	2.12	4.85	3.72	6.03	6.23	2.97
26 - 28	17.55	0.00	1.31	13.91	1.25	5.24	2.26	4.52	6.40	3.34
28 - 30	18.23	0.00	1.24	11.73	0.55	7.20	1.76	6.44	4.55	4.32
30 +	21.84	0.00	5.46	22.74	0.59	31.98	4.53	10.14	28.28	20.62

Table 2.7 (c) Axle Load Distribution for the Tridem Axle

Weight (kips)	Single Axle Load Distribution									
	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
0 - 2	0.00	0.00	0.00	0.00	0.00	0.16	0.00	0.37	0.00	0.06
2 - 4	0.00	0.00	0.00	0.00	0.00	2.84	0.11	3.46	0.00	0.67
4 - 6	0.00	0.00	0.00	0.00	0.00	13.67	0.91	13.66	0.12	2.95
6 - 8	0.00	0.00	0.00	0.00	0.00	9.60	1.29	11.94	0.00	4.59
8 - 10	0.00	0.00	0.00	0.00	0.00	11.19	1.75	13.19	0.61	6.87
10 - 12	0.00	0.00	0.00	0.46	0.00	17.49	6.05	19.48	0.61	8.39
12 - 14	0.00	0.00	0.00	0.15	0.00	15.98	16.93	17.75	0.97	6.14
14 - 16	0.00	0.00	0.00	0.31	0.00	11.67	9.42	11.04	1.58	3.86
16 - 18	0.00	0.00	0.00	0.69	0.00	7.31	3.97	4.38	1.70	2.40
18 - 20	0.00	0.00	0.00	0.69	0.00	3.73	2.89	1.83	1.46	1.61
20 - 22	0.00	0.00	0.00	0.53	0.00	2.32	2.28	0.94	1.22	1.16
22 - 24	0.00	0.00	0.00	1.07	0.00	1.45	2.41	0.37	1.58	1.46
24 - 26	0.00	0.00	0.00	2.36	0.00	1.00	2.70	0.12	1.58	1.67
26 - 28	0.00	0.00	0.00	2.90	2.08	0.65	2.97	0.12	2.07	1.40
28 - 30	0.00	0.00	0.00	2.82	0.00	0.46	3.05	0.10	3.41	1.52
30 +	0.00	0.00	0.00	88.02	97.92	0.49	43.27	1.26	83.09	55.24

2.3.3 Materials

Table 2.8 shows the pavement layer information (e.g., thickness and type of material used for each layer) for the 20 selected AC and PCC pavement sections. The subgrade soil regions are also shown. The type and thickness of the surface layer (some with binders), base layer, and subgrade layer were obtained from the pavement design files located in the SCDOT Office of Materials and Research.

Table 2.8 Pavement Layer Information

Location	Surface Type	Surface & (binder) Thickness (in.)	Top Base Type	Top Base Thickness (in.)	Bottom Base Type	Bottom Base Thickness (in.)	Subgrade Soil Region
I-520	PCC	11	AA	1.5	GAB	8	Mid-zone
US-278	AC	1.6(2)	AA	3.2	GAB	6	Coastal
SC-461	AC	3.4 (2.3)	AA	2.7	SAB	8	Coastal
I-526	PCC	11	CSM	6	CMS	6	Coastal
SC-9	AC	3.4(2.7)	GAB	8	CMS	6	Piedmont
SC-151	AC	1.6(2.3)	AA	2.7	Sand Clay	8	Mid-zone
I-77	PCC	10	Lean Concrete	6	CMS	6	Piedmont
SC-327	AC	2.8(4.1)	Macadam	8	-	-	Coastal
US-301	AC	1.8(2)	GAB	8	CMS	6	Coastal
US-521	AC	1.8(2)	GAB	8	CMS	6	Coastal
I-385	AC	4.6(12)	CSM	6	-	-	Piedmont
I-85	AC	1.6 (2.3)	AA	7.7	-	-	Piedmont
SC-22	AC	1.8 (2)	AA	5.5	GAB	8	Coastal
SC-31	AC	1.6 (2.2)	AA	2.7	GAB	8	Coastal
SC-72	AC	1.8 (1.8)	AA	6.8	-	-	Piedmont
S-378	PCC	9	GAB	6	-	-	Mid-zone
US-321	AC	1.8 (3.8)	GAB	6	-	-	Mid-zone
SC-93	AC	1.6 (1.8)	AA	5.8	-	-	Piedmont
SC-80	PCC	10	GAB	5	-	-	Piedmont
I-85	PCC	12	AA	4	CMS	6	Piedmont

Note: AC = Asphalt Concrete, PCC = Portland cement Concrete, GAB = Graded Aggregate Base, AA = Asphalt Aggregate Base, SAB = Stabilized Aggregate Base, CSM = Cement Stabilized Macadam, CMS = Cement Modified Subbase

2.3.3.1 Hot Mix Asphalt Properties

Binder grades, air voids, effective binder content, and mix gradations are required inputs for MEPDG. Site-specific mix design information was not available for the 20 selected pavement sections; thus, 1) the SCDOT 'Standard Specifications for Highway Construction' versions 1986, 2000 and 2007 and 2) asphalt mix design information obtained for various job mixes from laboratory test reports for the period from 2012 to 2014 were reviewed. In addition, existing laboratory test data for the binder and mixture properties of hot mix asphalt (HMA) were compiled into a catalog of typical design inputs (see Table 2.9). Guidelines for asphalt mixture selection (SCDOT, 2013) are shown in Table 2.10 and asphalt mix design of a typical job mix of asphalt surface course of Type A and intermediate course Type B are shown in Appendix B.

Table 2.9 SCDOT Typical Asphalt Mix Design (2011)

HMA Properties	Target
Percent Binder	5
Maximum Specific Gravity	2.446
Bulk Specific Gravity	2.362
% Air Voids in Total Mix	3.4
% VMA	14.8
% Voids Filled	76.8
Effective Specific Gravity	3.635
Grade of Binder	PG 76-22
Binder Specific Gravity	1.037

From Table 2.10, binder grades PG 64-22 and PG 76-22 were used as Level 2 (regional) inputs for the intermediate and surface course, respectively, for all selected pavement sections. For each of these binder grades, Level 3 (default) values of unit weight, effective binder content and air voids were used. Information from Table 2.9 and Appendix B was not used because this mix design information is from different job mixes but not for the specific binder type (PG grade) used in the selected locations. Note that the Level 3 (default) value of air voids is 7% which is equal to the field-derived average air voids for SCDOT pavements in Appendix B. The average air voids of 3.6% reported from laboratory tests (see Appendix B) is about 50% less than the average field value and was not used herein. For Phase I of the project, it was deemed acceptable to use the Level 2 binder grade type and Level 3 mix design information. However, it is recommended to use Level 1 or Level 2 binder grades and mix design information in Phase II.

Table 2.10 Guidelines for Asphalt Mixture Selection (SCDOT, 2013)

Type	Type Facility	AADT	Mix Design Type	Est. % Binder	Binder Grade	Recom. Rate (lbs/SY)
Surface	Interstate		A	5	PG 76-22	200
	High volume primary & secondary	>=5000	B	5.3	PG 64-22	150-200
	Low volume primary & secondary	1500-5000	C	6	PG 64-22	150-175
	Low volume secondary	<1500	D	6.3	PG 64-22	125-150
	Multiple facility usage		E	6.5	PG 64-22	45-80
Intermediate	Interstate & high volume primary		A	4.6	PG 64-22	250-300
	High volume primary & secondary	>=5000	B	5.1	PG 64-22	200
	Primary to low volume secondary		C	4.8	PG 64-22	200-300
Base	Interstate & problem areas	>=5000	A	4.5	PG 64-22	300-450
	Primary & secondary	<5000	B	4.5	PG 64-22	300-450
	Special		C	5.5	PG 64-22	200-300
	Special		D	5	PG 64-22	200-300
Special Mixes	Interstate		OGFC	6.5	PG 76-22	110-150
	Primary & secondary		PMTLSC	6	PG 64-22	0.75"
	Primary & secondary		Widening	5	PG 64-22	400-600

Some dynamic modulus data were recently collected from project SPR 720, "Characterization of Asphalt Concrete Dynamic Modulus in South Carolina" funded by the FHWA/SCDOT and are shown in Figure 2.2. The dynamic modulus is shown for a range of frequencies at different temperatures (i.e., 20°, 4° and 40°) for three aggregate surface combinations (i.e., Aggregate type A and surface type B, C and D). These results are compatible with the MEPDG requirements, and could be used as a Level 2 input. However, the data was not used herein because it only became available at the end of the project. Additional data is expected to be available for the Phase II study.

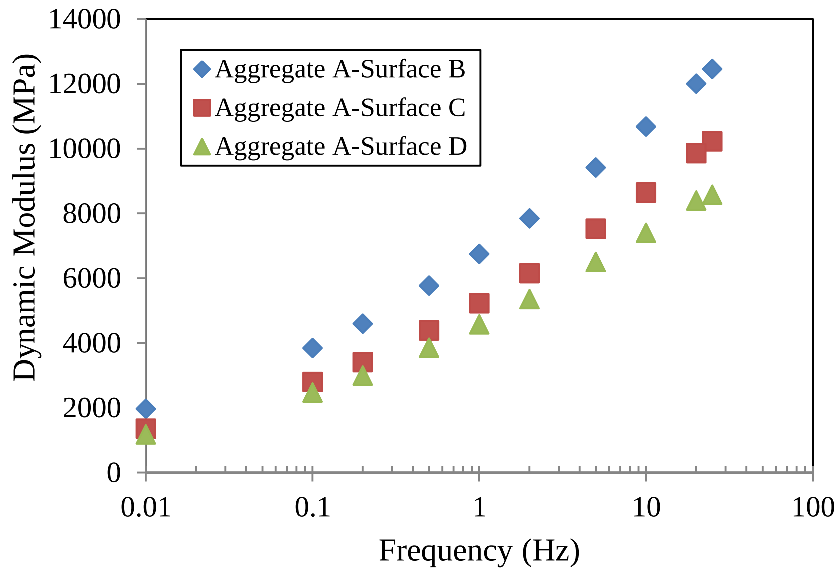


Figure 2.2(a) Dynamic Modulus Test Results for 20° C (SCDOT, 2016)

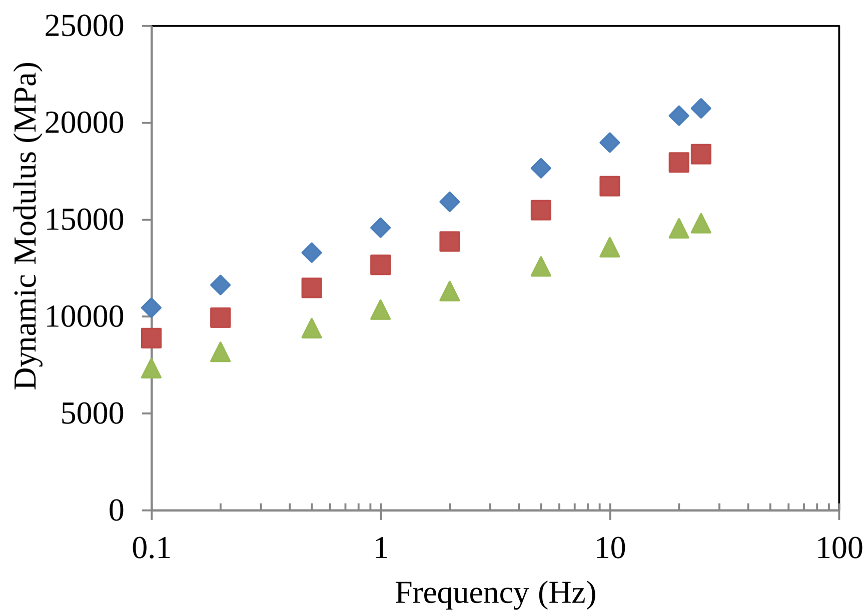


Figure 2.2(b) Dynamic Modulus Test Results for 4° C (SCDOT, 2016)

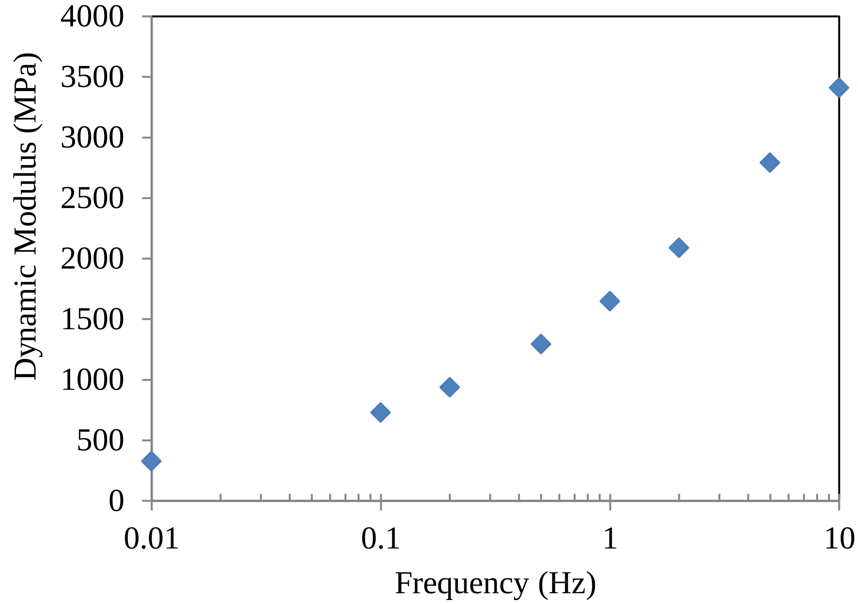


Figure 2.2(c) Dynamic Modulus Test Results for 40° C (SCDOT, 2016)

2.3.3.2 Portland Cement Concrete Properties

The required PCC properties for MEPDG are elastic modulus, Poisson's ratio, flexural strength, unit weight, compressive strength, coefficient of thermal expansion, thermal conductivity, heat capacity, cement type, cementitious material content, water-cement ratio, aggregate type, and ultimate shrinkage. Baus and Stires (2010) recommended giving priority to obtaining Level 1 inputs for the elastic modulus, compressive strength, and flexural tensile strength. In addition, the coefficient of thermal expansion (CTE) has been identified by Tanesi et al., (2007) as an important PCC parameter. However, none of the required properties for PCC pavements were available in the historical data files at the SCDOT, thus Level 3 (default) PCC pavement properties were used for the preliminary MEPDG local calibration presented in Chapter 4. Note that the CTE of PCC is currently being studied under SPR 722, "Characterization of Portland Cement Concrete Coefficient of Thermal Expansion in South

Carolina." Additional studies are needed to obtain the other PCC properties for use in the MEPDG local calibration in Phase II.

2.3.3.3 Unbound Material Properties

The material properties required for unbound layers for MEPDG are Poisson's ratio, coefficient of lateral earth pressure, resilient modulus, gradation, and other engineering properties (i.e., liquid limit, plasticity index, and maximum dry unit weight). Resilient modulus has the greatest effect on MEPDG (Orobio and Zaniewski, 2011), but is not available in the historical files at the SCDOT. Soil classification and California Bearing Ratio (CBR) were the only historical unbound material property data available for some of the pavement sections in the SCDOT files. The available data for SC-93 (Pickens), US-521 (Georgetown), and S 378 (Lexington) are shown in Appendix C.

As discussed by Schwartz (2007), several of the material parameters (e.g., subgrade resilient modulus, M_R) required for the new MEPDG are similar to the inputs for the previous AASHTO design guide. However, some parameters (e.g., soil thermo-hydraulic properties) are not traditionally measured in standard agency laboratories. For the parameters not currently available from the SCDOT, Level 3 (default values) inputs were adopted. For the resilient modulus, laboratory tests (see Section 2.4.2) were performed on Shelby tube samples collected from 3 sites: US-321 in Orangeburg County, US-521 in Georgetown County, and SC-93 in Pickens County; and used as Level 1 inputs for these 3 sites and Level 2 inputs for the other 17 selected sites.

2.3.4 Pavement Performance Data

The pavement performance data required for MEPDG are mainly associated with pavement distresses and roughness. By working closely with the SCDOT Division of Traffic

Engineering and the Pavement Management Group, the record of historical pavement distresses, including fatigue cracking, transverse cracking, longitudinal cracking, rutting and International Roughness Index (IRI) for flexible pavements; and fatigue cracking, transverse cracking, and IRI for rigid pavements were collected. However, faulting data for rigid pavements were not found. The biggest challenge to use the performance data from SCDOT's specific pavement management sections is the incompatibility of the SCDOT pavement data collection protocols with the new MEPDG distress identification protocols. Two possible methods to address performance data are: (1) establish sections from SCDOT's pavement network and collect data for the next 3 to 5 years; or (2) use approximate data conversion techniques to translate SCDOT performance data into the MEPDG format. Due to the time limitation for Phase I, the second approach was tentatively selected to quantify the pavement performance data for local calibration and validation. However, in Phase II of this project, the first approach to collecting high quality performance data will be considered.

In South Carolina, International Roughness Index (IRI) values are derived from wheel path profiles obtained using non-contacting inertial profilers. Figure 2.3 shows the IRI values for the selected pavement sections for the most recent year that were collected from the SCDOT Integrated Transportation Management System (ITMS). The pavement condition indices (Present Serviceability Index (PSI), Pavement Distress Index (PDI), and Pavement Quality Index (PQI)) are available for different mileposts in ITMS. These data were compiled for the selected pavement sections and shown in Appendix D. According to FHWA (2004), IRI values less than 170 in./mi are acceptable and any IRI value less than 95 in./mi indicates good roughness condition of the pavement (Shahin, 2005). The IRI values are heavily dependent on the other distresses calculated by MEPDG and the site factor. The site factor depends on pavement age, plasticity index of soil, fines content, freezing index, and precipitation (AASHTO, 2008).

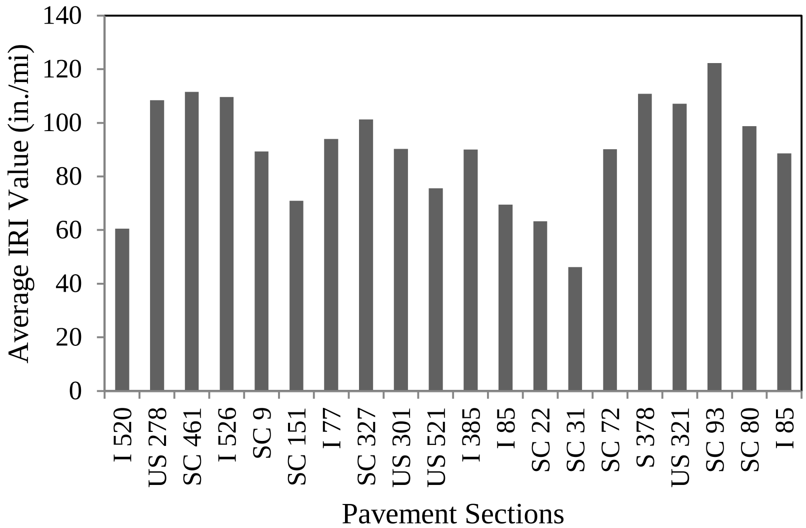


Figure 2.3 Average IRI (Both Directions)

Rutting is a longitudinal surface depression in the wheel path resulting from plastic or permanent deformation in each pavement layer. SCDOT measures rutting using an automated profiler connected to a moving vehicle. Figure 2.4 shows the average rut depth for the 14 selected AC sections. Rutting less than 0.5 inches is acceptable and considered as less severe (Shahin, 2005).

Individual distress data were also compiled for the selected pavement sections. The distress data were collected by a vendor under contract with the SCDOT using a pavement profiler according to the distress identification training manual (FHWA, 2003). The vendor collects the distress data and uses software to process the data to obtain the distress quantity of various severities (i.e., low, medium, or high). To use these distress data for MEPDG, distress quantities of various severities were summed up and the maximum distress from each segment was taken. As the confidence in SCDOT data was very low, it was observed that the maximum distresses showed better trend than the average distresses with age. Therefore, the maximum distress has been used. On the other hand, for rutting and IRI, the average value for each segment were used in this study due to their better trend than the maximum value. In summary, the maximum value for each distress (i.e., fatigue cracking, longitudinal cracking, transverse cracking) were used and average values of IRI and rutting were used.

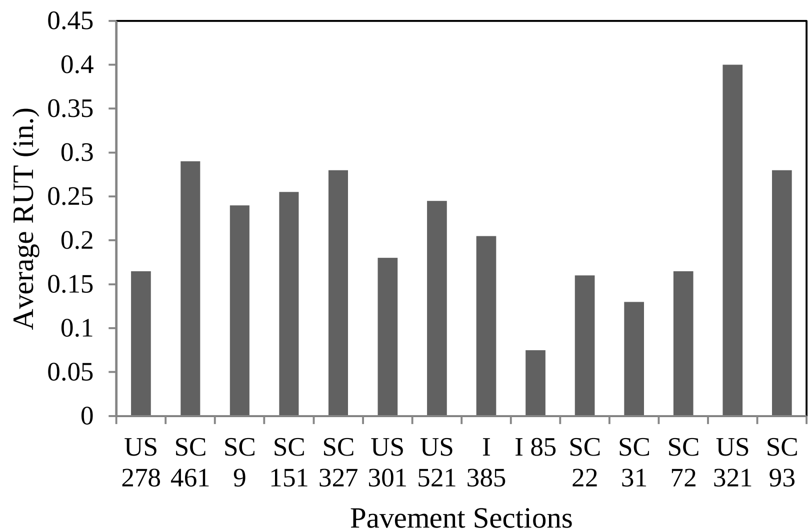


Figure 2.4: Average Rutting (Both Directions)

A comparison of the performance data that are used in MEPDG and are measured by SCDOT are shown in Table 2.11. Note that while SCDOT does measure fatigue cracking and alligator cracking for AC pavements, bottom up and top down cracking are not clearly distinguished by their procedure as they measure distresses only by visual inspection; thus, the data in its current form cannot be implemented into MEPDG with confidence. Therefore, pavement coring and trench studies are recommended for Phase II to measure top down and bottom up cracking. Moreover, SCDOT does not measure PCC pavement cracking as required by MEPDG (i.e. faulting). Detailed pavement inspections for PCC pavements are also recommended for Phase II.

Table 2.11 MEPDG Required and SCDOT Measured Performance Data

Pavement Type	Pavement Performance	Units	MEPDG Used	SCDOT Measured
AC	IRI	in./mi	√	√
	Rut Depth	in.	√	√
	Fatigue/Alligator/Bottom Up (Load)	% lane area	√	√*
	Longitudinal/Top Down (Load)	ft/mi	√	√**
	Transverse (Non-load)	ft/mi	√	√
	Reflection Cracking	% lane area	√	
	Ravelling	% lane area		
	Patching	% lane area		
PCC	IRI	in./mi	√	√
	Transverse Slab Cracking (JPCP)	% slab	√	√
	Mean Transverse Joint Faulting (JPCP)	in.	√	
	CRCP Punchouts	Punchouts/mi	√	

Note: JPCP = Jointed Plain Concrete Pavement, CRCP = Continuously Reinforced Concrete Pavement
*Top-down and bottom-up fatigue cracking are not known
**SCDOT measured longitudinal cracking in %

2.4 NEW DATA FOR MATERIALS INPUTS

Three pavement sections were selected for further study to obtain new data for the materials inputs: US-321 in Orangeburg County, US-521 in Georgetown County, and SC-93 in Pickens County. These sites were selected to represent different soil regions above and below the fall line as shown in Figure 2.5. SC-93 in Pickens County was selected to represent the soils in the Piedmont; whereas, US-521 in Georgetown County represents the Coastal Plain. US-321 in Orangeburg County is in the Coastal Plain, but is located more inland and closer to the fall line than US-521, thus is referred to as being located in the mid-zone. These sites were also selected due to their low traffic activity; and thus were sites where disruptions to traffic flow from lane closures would be minimal. At each of these sites, Falling Weight Deflectometer (FWD) tests were performed, asphalt cores were collected, and soil samples (Shelby tube samples and bulk samples) were collected. The spacing and number of tests and samples for each are shown in Table 2.12. Maps showing the location of each pavement section, the locations of the FWD tests and boring locations along each pavement section, and a photograph showing the surface pavement conditions at each of the three sites are shown in Figure 2.6.

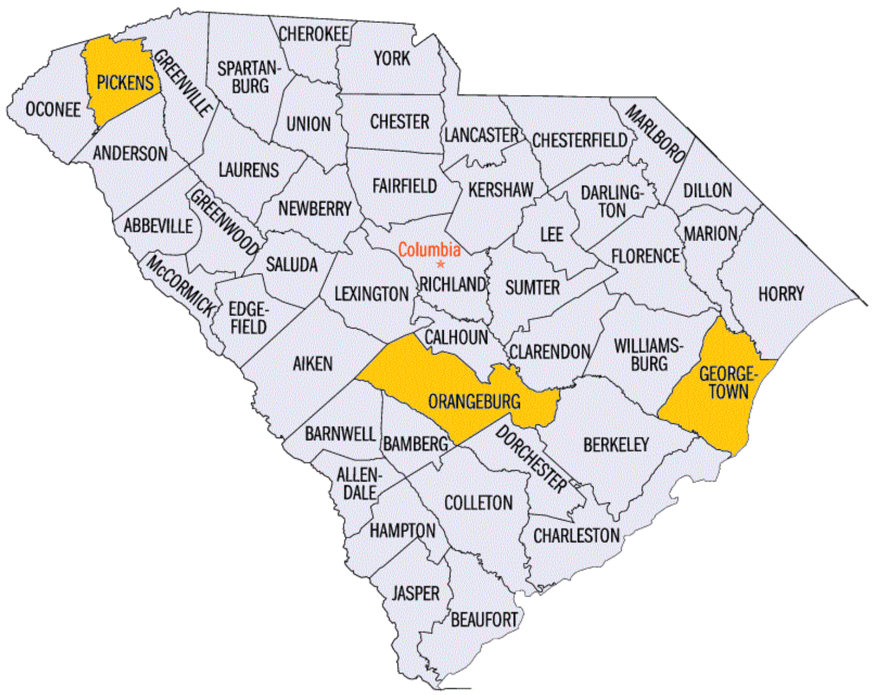


Figure 2.5 Selected Sections for Pavement Coring

Table 2.12 Sample Collection and FWD Testing

Site	County	Region	Length (mi)	Boring Spacing (ft)	Total No. of Boreholes	No. of Shelby Tube Samples	Bags of Bulk Soils	No. of Asphalt Cores	No. of FWD Tests
US-321	Orangeburg	Mid-zone	6.17	1500-3000	13	37	13	13	21
US-521	Georgetown	Coastal	4.07	3000	7	19	7	7	13
SC-93	Pickens	Piedmont	1.34	3000	5	26	5	5	9

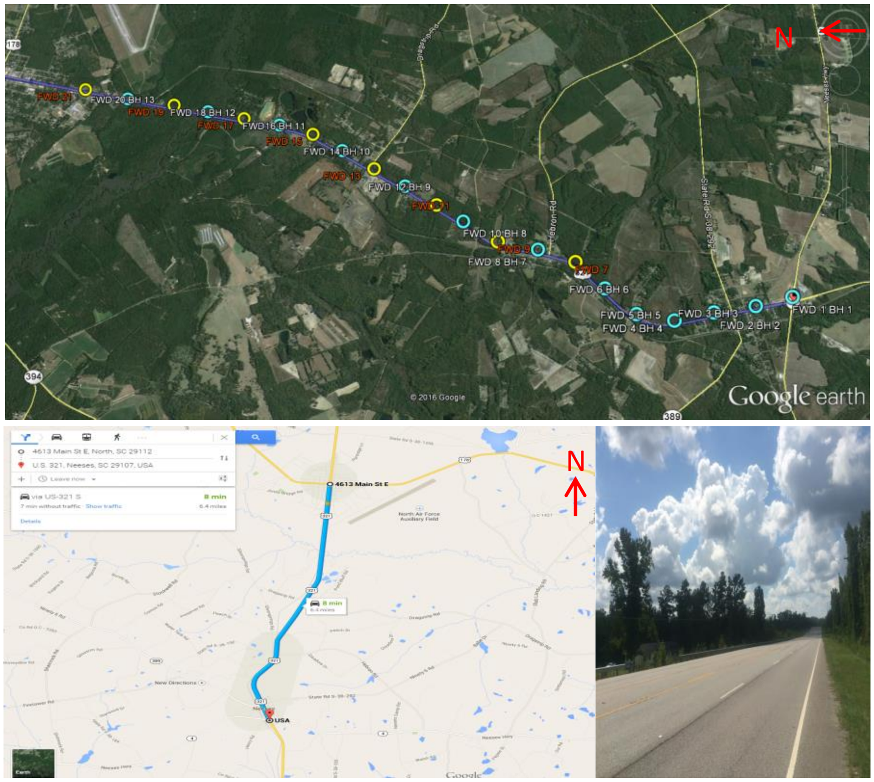


Figure 2.6 (a) FWD Testing and Borehole Locations (US-321, Orangeburg)

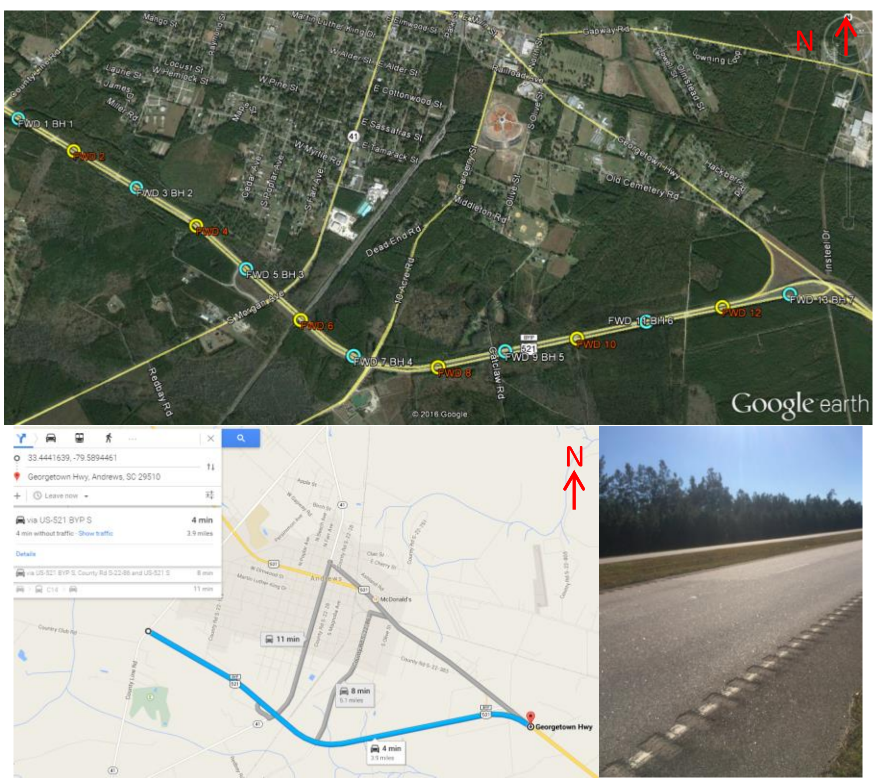


Figure 2.6 (b) FWD Testing and Borehole Locations (US-521, Georgetown)

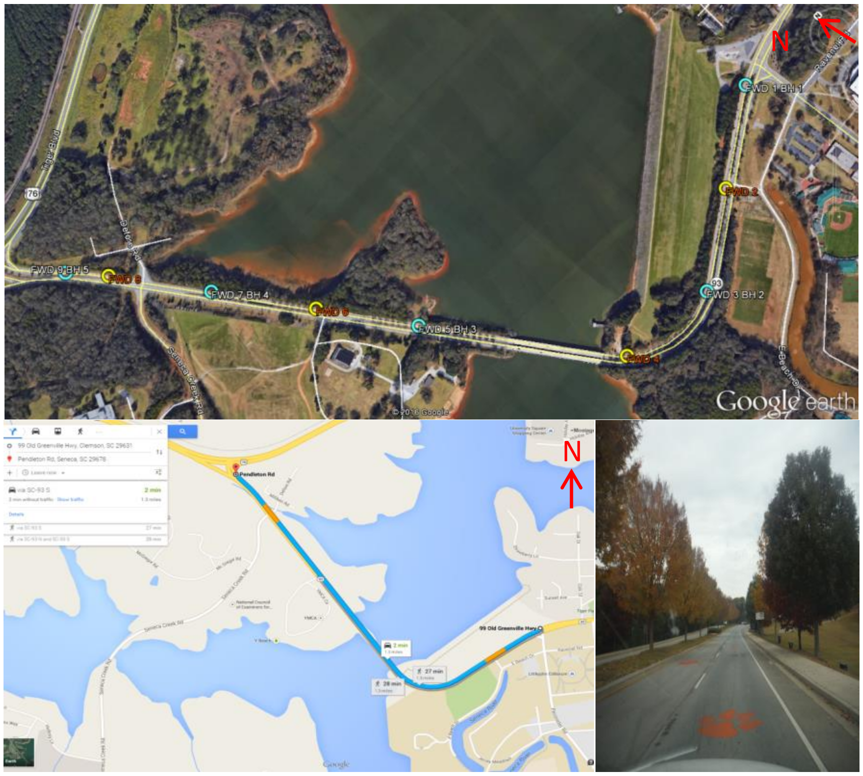


Figure 2.6 (c) FWD Testing and Borehole Locations (US-93, Pickens)

2.4.1 Hot Mix Asphalt Properties

Cores of the HMA were collected from each of the three pavement sections. The location and number of samples collected are shown in Table 2.12. These samples are currently stored at the USC Geotechnical Laboratory and can be used for testing to obtain material properties, such as dynamic modulus, in Phase II.

Figure 2.7 FWD Testing Equipment

FWD tests were performed using the Dynatest FWD equipment shown in Figure 2.7. The equipment consists of 7 sensors located at 7 different offsets along the loading plate (0.0 in., 7.9 in., 11.8 in., 17.7 in., 23.6 in., 35.4 in., and 47.2 in. from the loading plate). A FWD test is performed by applying load of 4 different magnitudes (6.1 kip, 14.5 kip, 10.9 kip, and 8.6 kip respectively) and collecting deflection data for those loads by the 7 sensors. Deflection data are used to determine the modulus of the subgrade with SCDOT back-calculation software. FWD tests were performed at the center of the design lane (right lane). FWD tests were performed coincident with the sampling locations as summarized in Table 2.12. The FWD test results are shown in Table 2.13(a), Table 2.13(b), and Table 2.13(c) for US-321, US-521, and SC-93, respectively.

Table 2.13(a): FWD Test Results for US-321 of Orangeburg County

Distance (ft)	Structural Number (SN)	Subgrade Modulus (psi)	Subgrade Modulus (MPa)
0	3.44	20671	143
1516	3.31	22296	154
3016	2.76	33498	231
4503	3.08	23333	161
6047	5.24	43588	301
7535	3.68	25909	179
9004	4.07	24177	167
10540	3.51	18519	128
12055	3.64	36176	249
13518	3.42	24178	167
15016	3.64	32677	225
16517	4.28	28949	200
18010	3.69	29351	202
19510	3.56	26730	184
20954	3.22	28897	199
22519	3.03	38157	263
24014	5.21	26340	182
25578	4.52	38854	268
27018	5.38	36101	249
28505	3.44	27903	192
30063	3.44	25556	176

Table 2.13(b): FWD Test Results for US-521 of Georgetown County

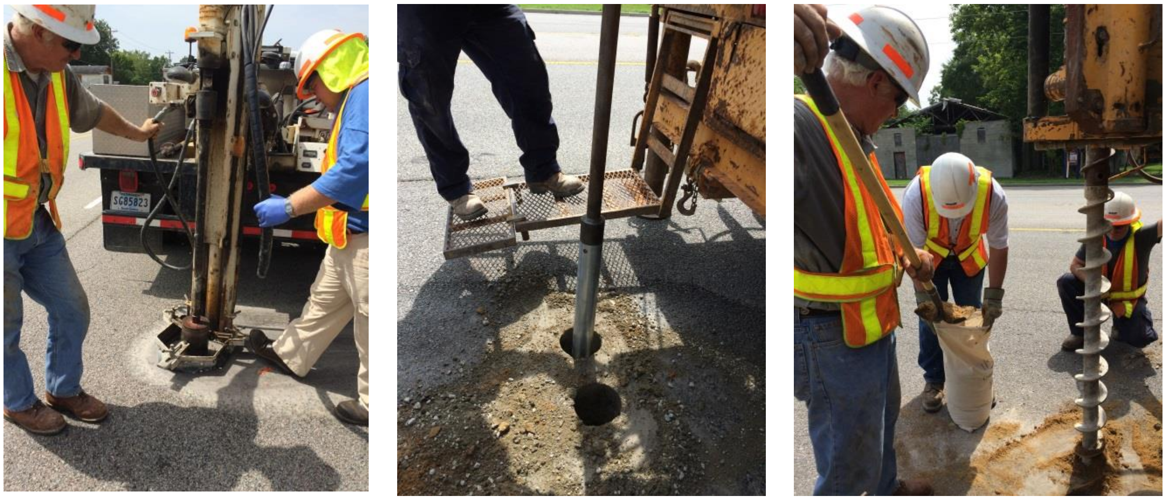
Distance (ft)	Structural Number (SN)	Subgrade Modulus (psi)	Subgrade Modulus (MPa)
0	4.63	33203	229
1507	5.08	36819	254
3101	2.97	22440	155
4529	3.51	24720	170
6032	3.56	29164	201
7455	2.88	16140	111
9057	2.65	20822	144
10534	2.27	29712	205
12088	4.32	35100	242
13529	3.13	23856	164
15113	3.89	25357	175
16519	4.2	34261	236
18138	3.51	27207	188

Table 2.13(c): FWD Test Results for SC-93 of Pickens County

Distance (ft)	Structural Number (SN)	Subgrade Modulus (psi)	Subgrade Modulus (MPa)
0	5.3	13000	90
753	4.23	6454	45
1510	4.09	8194	56
2256	5.39	14119	97
3758	4.58	8910	61
4516	6.14	39808	274
5252	6.53	42129	290
6003	4.77	8980	62
6407	4.69	10745	74

2.4.2 Unbound Material Properties

As part of this Phase I study, resilient modulus (M_R) values of unbound materials from 3 different soil regions of SC were determined though field and laboratory testing. In the laboratory, the resilient modulus was determined by a repeated load triaxial compression test per AASHTO T 307 (2003). To perform a test, a repeated axial cyclic stress of fixed magnitude, load duration, and cycle duration is applied to a 3 in. x 6 in. cylindrical test specimen. During testing, the specimen is subjected to a dynamic cyclic stress and a static-confining pressure provided by means of a triaxial pressure chamber. The total resilient or recoverable axial deformation response of the specimen is measured and used to calculate the M_R (AASHTO, 2003).



(a) Asphalt Coring (b) Shelby Tube (c) Bulk Soil

Figure 2.8 Field Sample Collection

SCDOT and USC personnel worked together to collect both disturbed bulk samples and Shelby tube samples of soils from the different soil regions of South Carolina. Photographs of the coring and sampling are shown in Figure 2.8. The number of soil samples collected for each type is shown in Table 2.12. The plan and profile views of a typical soil boring are shown in Figure 2.9. After the asphalt cores were collected, high quality soil samples were collected in 3 ft long and 3 in. diameter Shelby tubes. There were a total of 25 boring locations in 1500 to 3000 ft spacing for US-321 and 3000 ft spacing for US-521 and SC-93. Bulk soil samples were also collected and used to classify the soil and perform strength tests. Shelby tubes were tightly sealed and stored in the SCDOT concrete curing room (maintain 100% humidity) before being brought to USC for testing.

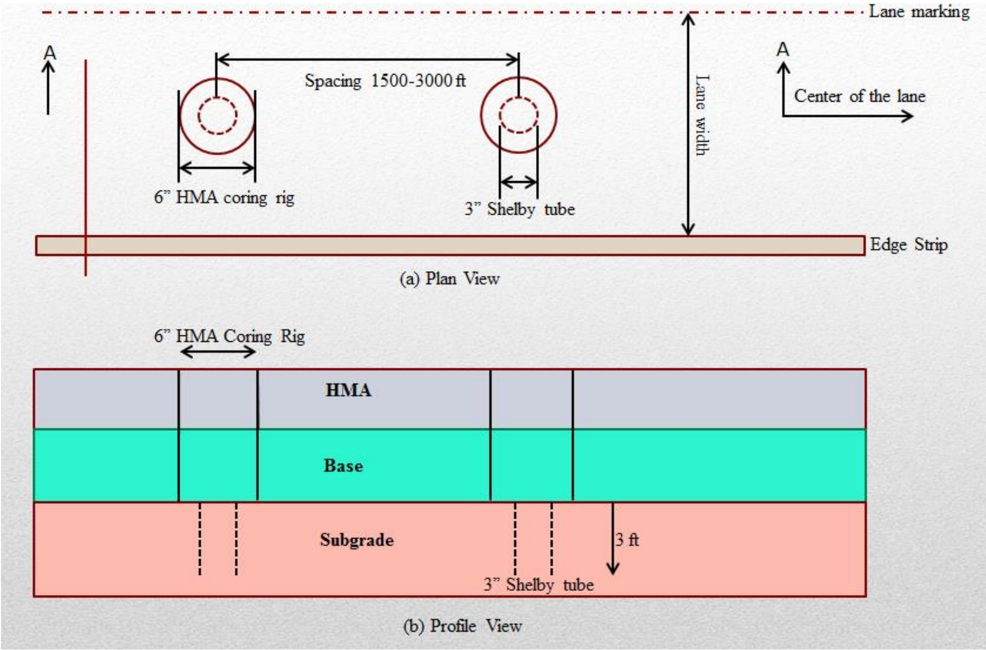
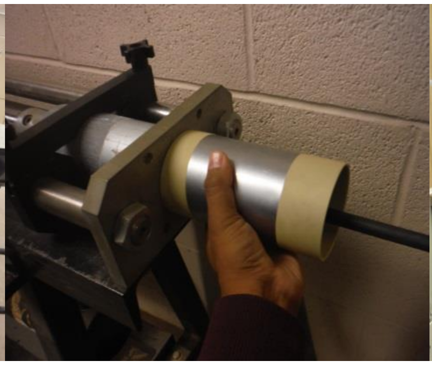
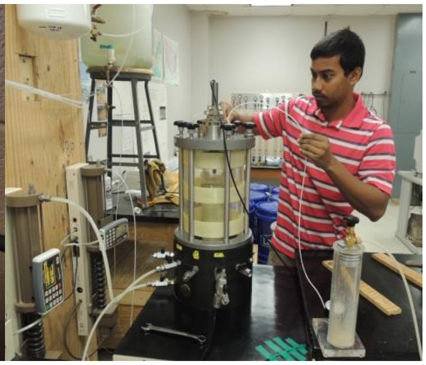


Figure 2.9 Plan and Profile View of Subgrade Sampling

Each Shelby tube was cut into 6 in. long sections. The soil samples were extruded and inserted into a rubber membrane prior to being brought to the USC Geotechnical Laboratory for resilient modulus testing. Photographs showing the soil extrusion and the laboratory testing equipment are shown in Figure 2.10. The current protocol for determination of resilient modulus of soil and aggregate material is AASHTO T 307.





(a) Cutting the Tube (b) Soil Extrusion (c) M_R Testing

Figure 2.10 Soil Extrusion and Laboratory Testing

A total of 82 resilient modulus tests were performed on samples from the 25 boring locations located along the 3 pavement sections. Example test results are shown in Figure 2.11 and Figure 2.12 for Sample No. 1211 (borehole No. 12, Shelby tube No. 1, sample No. 1 from top) from US-321 (Orangeburg). Figure 2.11 shows the resilient modulus increases with increasing cyclic stress and higher resilient modulus is found for higher confining pressure. These results are indicative of granular materials. Figure 2.12 shows the relation between resilient modus and bulk stress for this sample. The coefficient of determination (R^2) was 0.98. Similarly high coefficients of determination (R^2) were observed for most of the other tests.

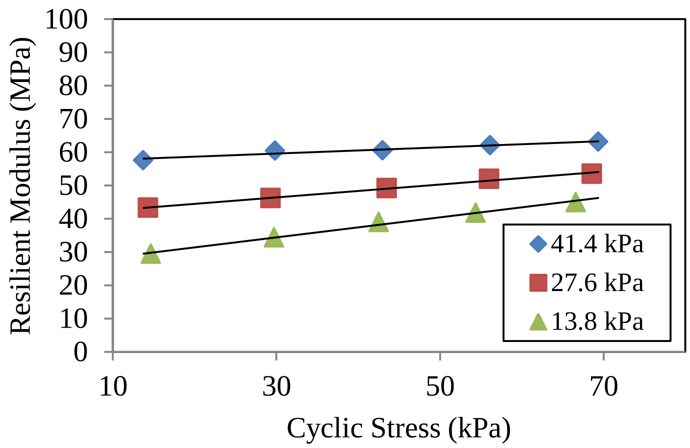


Figure 2.11 M_R Test Results for Sample No.1211 from US-321 (Orangeburg)

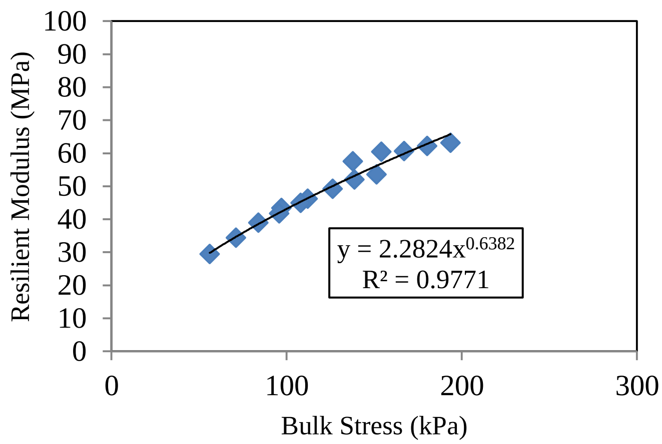


Figure 2.12 M_R versus Bulk Stress for Sample No.1211 from US-321 (Orangeburg)

Relations between M_R and bulk stress were obtained for each of the 37 samples for US-321 (Orangeburg), 19 samples for US-521 (Georgetown), and 26 samples for SC-93 (Pickens). The relations were combined for each of the 3 sites and are shown in Figures 2.13(a), (b), and (c). The best fit line was obtained using the mean of all the test results for each site. The coefficient of determination (R^2) values for US-321 (Orangeburg) (=0.42) and US-521 (Georgetown) (=0.30) are both low and indicate a large variation in M_R found for each of the boring locations along the length of each pavement section. The coefficient of determination (R^2) for SC-93 (Pickens) (=0.06) is even lower.

A significant difference in M_R for each pavement section (different geographic locations) was found. For a representative bulk stress of 154.64 kPa, the average M_R was 52, 50, and 40 MPa for US-321 (Orangeburg), US-521 (Georgetown), and SC-93 (Pickens), respectively with corresponding COV of 0.17, 0.20, and 0.42. This suggests that the variation of M_R along each of the three pavement sections must be considered when selecting a M_R for input to MEPDG.

Model parameters were obtained using the bulk stress models and the AASHTO M-E Pavement Design Models. Model parameters of AASHTO M-E Pavement Design Models can be used directly as inputs to MEPDG for local calibrations which are shown in Table 2.14. The resilient modulus data was used to represent the annual representative values in MEPDG. However, it is recommended to obtain monthly representative values in future phases of this research.

Table 2.14(a) AASHTO M-E Pavement Design Model Parameters (US-321, Orangeburg)

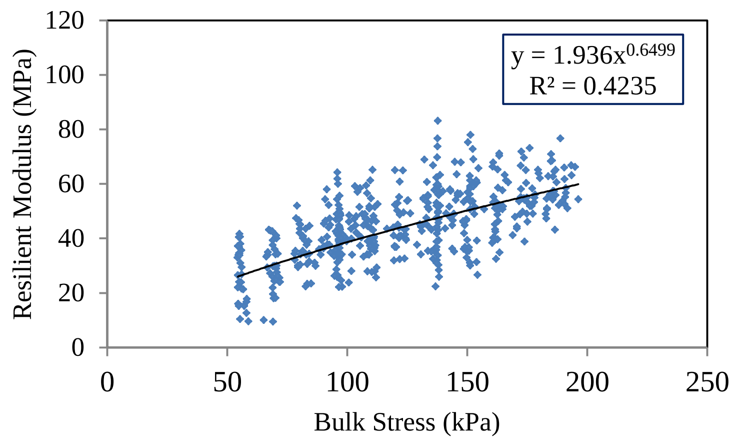
Sample	K1	K2	K3	R2
121	521	0.5815	-0.4697	0.99
211	366	0.0153	1.5230	0.39
221	228	0.7665	1.7602	0.68
223	210	0.3742	2.6134	0.88
312	423	0.3983	0.3155	0.69
313	474	0.5545	-0.7247	0.95
411	301	0.4231	1.1437	0.83
412	295	0.8643	0.5415	0.93
511	226	1.0940	0.6946	0.87
512	374	0.5943	0.4530	0.84
612	239	0.7979	1.7598	0.87
613	169	0.2916	4.4158	0.91
711	214	0.9368	0.5674	0.94
712	288	0.4356	1.4698	0.88
713	203	1.0389	1.0715	0.82
811	188	0.4284	3.2315	0.83
812	332	0.9900	-0.1844	0.94
822	599	0.6676	-1.2901	0.99
912	437	0.7035	-0.5806	0.97
1011	414	0.2389	1.7091	0.64
1012	539	0.6703	-0.3943	0.82
1013	341	0.2068	1.5598	0.63
1014	430	0.8750	-0.3099	0.92
1111	263	0.8419	-0.1828	0.82
1122	704	0.7032	-1.7826	0.98
1123	624	0.6117	-0.6808	0.95
1211	458	0.6821	-0.4018	0.99
1212	383	0.3870	0.2243	0.93
1221	402	0.8087	-0.7142	0.97
1311	377	0.8294	-0.4701	0.99
1312	501	0.4811	-0.1593	0.99
1321	491	0.5862	-0.6179	0.99

Table 2.14(b) AASHTO M-E Pavement Design Model Parameters (US-521, Georgetown)

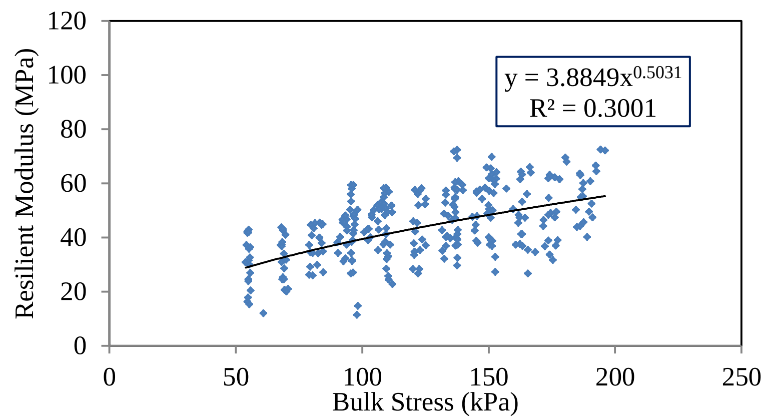
Sample	K1	K2	K3	R2
111	392	0.4136	-0.0870	0.79
112	337	0.3222	0.7625	0.79
113	487	0.3113	0.0333	0.53
114	543	0.6788	-0.7112	0.93
211	256	0.1433	1.7622	0.86
221	391	0.7856	-0.7849	0.90
222	553	0.3476	-0.8293	0.86
223	183	0.7132	1.8977	0.74
311	635	0.5696	-1.4083	0.98
321	618	0.4955	-1.1701	0.96
322	319	0.4635	-0.0896	0.80
411	353	0.2666	1.0594	0.49
511	208	0.4990	2.5897	0.79
512	446	0.8075	-0.1503	0.99
513	247	0.4523	1.6902	0.77
611	484	0.7143	-0.2119	0.99
613	413	0.2853	0.5394	0.68
721	332	0.0425	1.8625	0.87
722	129	0.1933	4.5193	0.76

Table 2.14(c) AASHTO M-E Pavement Design Model Parameters (SC-93, Pickens)

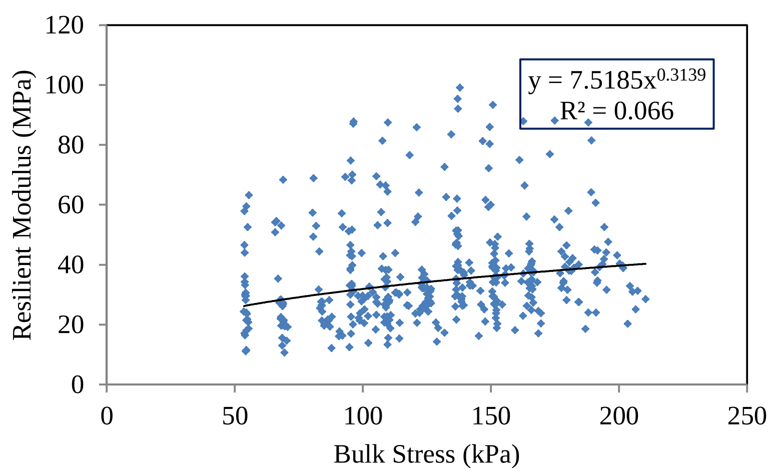
Sample	K1	K2	K3	R2
111	323	0.7742	-1.6326	0.98
112	386	0.3598	-0.8676	0.54
113	313	0.2994	0.0406	0.43
114	162	0.6043	-0.8476	0.80
115	417	0.6279	-1.0916	0.86
211	349	0.6788	-2.1792	0.94
212	232	0.4578	-0.0576	0.75
213	300	0.4220	-0.3387	0.60
214	344	0.7413	-2.0678	0.96
215	444	0.6306	-1.6429	0.95
311	295	0.2736	0.0263	0.51
312	145	0.5578	0.8150	0.81
313	403	0.7879	-2.0498	0.93
314	338	0.7418	-1.5104	0.94
411	825	0.5795	-1.4574	0.82
412	911	0.4199	-1.1576	0.95
413	623	0.1779	-0.5906	0.37
414	567	-0.0271	-0.2383	0.05
415	886	0.4912	-2.9989	0.96
511	395	0.7726	-1.7618	0.87
512	474	0.2371	-1.3406	0.53
521	227	0.5753	-1.0861	0.79
522	222	0.4876	-0.2695	0.58
523	278	0.3417	-0.7560	0.45
524	427	0.4721	-1.6531	0.78
525	434	0.3985	-1.4768	0.67



(a) Combined Bulk Stress Model for US-321 (Orangeburg)



(b) Combined Bulk Stress Model for US-521 (Georgetown)



(c) Combined Bulk Stress Model for SC-93 (Pickens)

Figure 2.13 Combined Bulk Stress Models

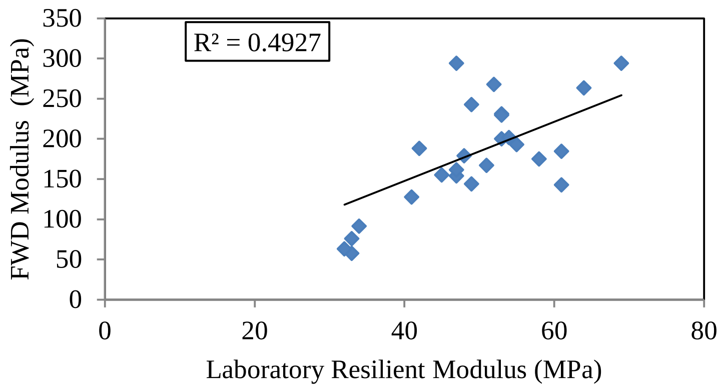


Figure 2.14 Laboratory M_R versus FWD Modulus

In Figure 2.14, the M_R obtained from the laboratory tests (for a representative bulk stress of 154.64 kPa) for each sampling location along all 3 pavement sections is compared to the corresponding FWD modulus tabulated in Table 2.13. This data was used to develop the following relation:

$$M_R(Lab) = 0.27 \times M_R(FWD) \qquad 2.1$$

In addition, grain size analysis (per ASTM D 6913-04, AASTHO T 311-00), Atterberg Limits (per ASTM D 4318-10, AASHTO T 90-15), maximum dry density (per ASTM D 698-12, AASHTO T 99-15), moisture content (per ASTM D 2216-10, AASHTO T-265-15), and unconfined compression (per ASTM D 2166-13, AASHTO T 208-05) tests were performed in the laboratory. The particle size distribution curves for samples from US-321 (Orangeburg), US-521 (Georgetown), and SC-93 (Pickens) are shown in Figure 2.15(a), (b), and (c), respectively. The corresponding soil classification according to USCS (per ASTM D 2488-09) and AASHTO (per AASHTO M 145-03) is shown in Table 2.15. The samples for SC-93 (Pickens) were classified as 3 different soil types: SM, CL or SC (A-4, A-6, A-7). This variation in soil properties between boreholes contributes to the high variation in M_R results for this site. The results for maximum dry density, moisture content, and unconfined compressive strength are summarized in Appendix E.

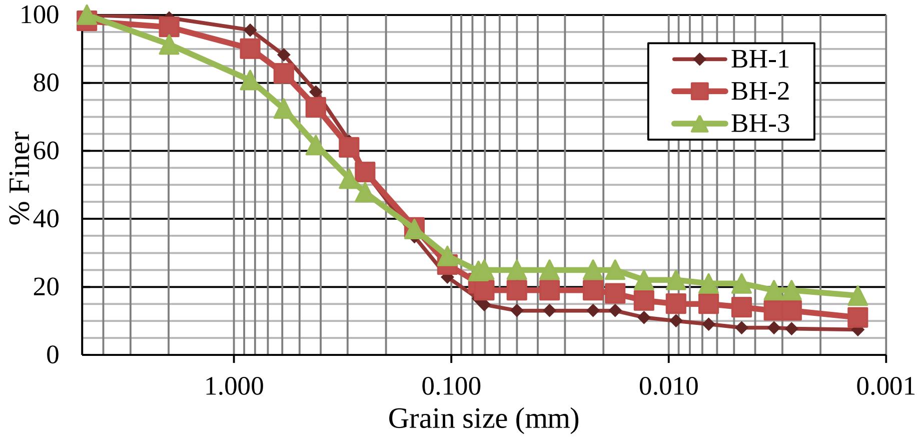


Figure 2.15 (a) Particle Size Distributions (US-321, Orangeburg)

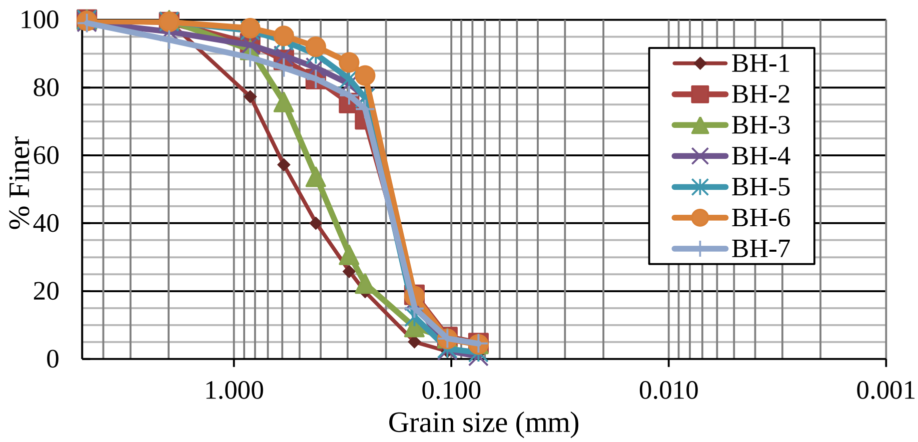


Figure 2.15 (b) Particle Size Distributions (US-521, Georgetown)

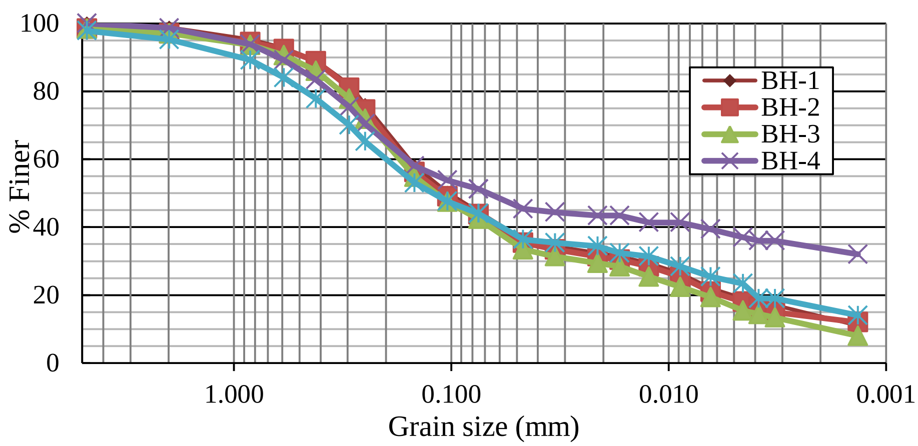


Figure 2.15 (c) Particle Size Distributions (SC-93, Pickens)

Table 2.15 Soil Classification

Site	BH	Soil Classification	
		USCS	AASHTO
Orangeburg	1	SM	A-2-4 (2)
	2	SM	A-2-4 (5)
	3	SC	A-2-4 (0)
Georgetown	1	SP	A-3 (0)
	2	SP	A-3 (0)
	3	SP	A-1-b (0)
	4	SP	A-3 (0)
	5	SP	A-1-b (0)
	6	SP	A-1-b (0)
	7	SP	A-3 (0)
Pickens	1	SM	A-4 (1)
	2	SM	A-4 (0)
	3	SM	A-4 (0)
	4	CL	A-6 (5)
	5	SC	A-7 (4)

2.5 INPUT LEVEL USED FOR PRELIMINARY MEPDG LOCAL CALIBRATION

Based on the collected and reviewed data for each of the 20 pavement sections, input levels that will be used in the Phase I (preliminary) calibration of MEPDG are shown in Table 2.16. Input Level 1 means the input parameter is measured directly and it is site or project specific. This level represents the greatest knowledge about the input parameter for a specific project. Input Level 2 means the input parameter is estimated from correlations or regression equations and it is state or region specific. A Level 3 input parameter is based on best estimated value or MEPDG default values, which are based on national studies (AASHTO, 2008).

Table 2.16: Input Levels Used in Preliminary Calibration

Input Group		Input Parameter	Input Level Used
Traffic		Average Annual Daily Truck Traffic	Level 1
		Vehicle Class Distributions	Level 1
		Lane and Directional Truck Distribution	Level 1
		Axle-Load Distribution	Level 2
		Hourly and Monthly Traffic Distribution	Level 3
		Tire Pressure	Level 3
		Axle Configuration, Tire Spacing	Level 3
		Truck Wander	Level 3
Climate		Temperature, Wind Speed, Cloud Cover, Precipitation, Relative Humidity	Level 1/2
Material Properties	Unbound Layers and Subgrade	Resilient Modulus - All Unbound Layers	Level 1/2/3
		Poisson's Ratio, Coefficient of Lateral Earth Pressure, Liquid Limit, Plasticity Index, Unit Weight	Level 1/2/3
		Classification and Volumetric Properties	Level 1/2/3
		Moisture-Density Relationships	Level 3
		Soil-Water Characteristic Relationship	Level 3
		Saturated Hydraulic Conductivity	Level 3
	HMA	HMA Dynamic Modulus	Level 3
		Creep Compliance and Indirect Tensile Strength	Level 3
		Volumetric Properties	Level 3
		Binger Grades	Level 2
		Air Voids, Effective Binder Content	Level 3
		Coefficient of Thermal Expansion	Level 3
	PCC	PCC Elastic Modulus, Poisson's Ratio	Level 3
		Flexural Strength, Unit Weight, Compressive Strength	Level 3
		Coefficient of Thermal Expansion, Thermal Conductivity, Heat Capacity	Level 3
		Cement Type, Cementitious Material Content, Water-Cement Ratio, Aggregate Type, Ultimate Shrinkage	Level 3
All Materials		Unit Weight	Level 1/2/3
		Poisson's Ratio	Level 3
		Other Thermal Properties	Level 3
Existing Pavement		Condition of Existing Layers	Level 1

CHAPTER 3

PRELIMINARY MEPDG ANALYSIS WITH GLOBAL CALIBRATION FACTORS

3.1 INTRODUCTION

This chapter presents the preliminary MEPDG analysis for the 20 identified in-service pavement sections using global calibration factors and the historical data that was collected and presented in Chapter 2. The performance analysis was performed using AASHTOWare (2016) pavement design software, which builds upon the new MEPDG. AASHTOWare calculates pavement responses (stresses, strains, and deflections) and combines them with the other pavement, traffic, climate, and materials parameters to predict the progression of key pavement distresses and smoothness loss over time. It is important to note that the analysis presented herein is considered to be preliminary because, as discussed in Chapter 2, many Level 3 inputs were used due to lack of site specific or regional data, there are insufficient numbers of pavement sections (only 14 AC sections and 6 PCC sections), and the quality of the distress data is uncertain.

3.2 STEP-BY-STEP PROCEDURE OF MEPDG LOCAL CALIBRATION

As shown in Table 3.1, there are 11 steps in the MEPDG local calibration procedure per the Local Calibration Guide (AASHTO 2010). Steps 1 to Step 7 are demonstrated in this chapter using all collected data from SCDOT described in Chapter 2. Steps 8 to Step 11 will be described in Chapter 4.

Table 3.1 Steps Suggested for Local Calibration (AASHTO, 2010)

Steps	Step-by step Procedure of MEPDG Local Calibration
1	Select hierarchical input level for each input parameter
2	Develop local experimental plan and sampling template
3	Estimate sample size for specific distress prediction models
4	Select roadway segments
5	Extract and evaluate distress and project data
6	Conduct field and forensic investigations
7	Assess local bias from global calibration factor
8	Eliminate local bias of distress and IRI prediction models
9	Assess the standard error of the estimate
10	Reduce standard error of the estimate
11	Interpretations of results, deciding on adequacy of calibration parameters

3.2.1 Step 1: Select Hierarchical Input Level

The first step in the local calibration process is to select the hierarchical input level for the inputs that will be used by the SCDOT for pavement design and analysis. The highest levels of input data available within SCDOT were used for the inputs (see Table 2.16) for the global calibration effort, and resulting standard error of the estimate. For instance, vehicle class distributions were available for most of the selected pavement sections; therefore, input Level 1 vehicle class data were used in the analysis for those sections. However, Weigh in Motion (WIM) axle load spectra data were not available for any of the 20 selected pavement sections, but was available for two other pavement sections in SC. Therefore, the axle load data from those two sites were used for the 20 selected sites (i.e., as a Level 2 input), which gave a better confidence level than just using the default (Level 3) axle load data available in AASHTOWare. For the unbound subgrade layer, Level 1 and Level 2 inputs were used; whereas, for the unbound base layer, Level 3 input was used because data was not available as discussed in Chapter 2. Level 3 inputs were used for the materials properties of the HMA and PCC pavement layers (except for binder grade type which was Level 2) due to lack of data within SCDOT. Level 1 climate data were used because of the availability of weather stations in SC. Moreover, Level 1 inputs were used for the existing pavement layer information (i.e., thickness, type).

3.2.2 Step 2: Develop Local Experimental Plan and Sample Template

The second step is to develop a detailed, statistically sound experimental plan and sample template to refine the calibration of the MEPDG distress and IRI prediction models based on local condition, policies, and materials. Table 3.2 shows the sample template used in this study, along with the number of pavement sections associated with each pavement type. A total of 20 sections were selected based on the review of available historical data (see Section 2.2).

Table 3.2 Sample Template

Pavement Type	Sub Category	No. of Pavement Sections	Total Sections
AC	New Flexible	14	20
	HMA Overlay	-	
PCC	JPCP	6	
	CRCP	-	

Selected pavement sections are primary or interstate routes located in the Coastal Plain and Piedmont regions in SC. Both flexible and rigid pavements with typical layer configuration and material selection, including traditional and new materials are included. Different service times for different types of pavements are included. In selecting pavement sections, priority was given to the initially selected sections with high quantity and quality of historical data, including climate, materials, traffic, and performance data.

3.2.3 Step 3: Estimate Sample Size for Specific Distress Prediction Model

The minimum number of recommended test sections for each distress for the local calibration of the MEPDG per AASHTO (2010) is as follows:

- Distortion (Total Rutting or Faulting) - 20 roadway segments

- Load Related Cracking - 30 roadway segments
- Non-Load-Related Cracking - 26 roadway segments
- Reflection Cracking (HMA surfaces only) - 26 roadway segments

Therefore, the selection of 20 pavement sections (i.e., 14 AC pavement sections and 6 PCC pavement sections) (per the initial recommendation of Baus and Stires, 2010) is not adequate for local calibration of the performance indicators in the list. However, because reviewing and identifying data within SCDOT was the main objective for Phase I of the project, it was deemed acceptable to start with 20 pavement sections with the maximum data availability. For Phase II, it is recommended to follow the AASHTO (2010) recommendation and use at least 30 pavement sections for each of AC and PCC pavements.

3.2.4 Step 4: Select Roadway Segments

This step is used to select roadway projects to obtain maximum benefit of existing information and data to keep sampling and field testing costs to a minimum. For this study, 14 AC pavements and 6 PCC pavements with multiple distress measurements and a sufficient number of observations (i.e., at least 2 years of pavement performance data) were selected as summarized in Section 2.2.

3.2.5 Step 5: Extract and Evaluate Distress and Project Data

This step consists of 4 activities: (1) extract and review available pavement performance data; (2) compare the performance indicator magnitudes to the trigger values; (3) evaluate the distress data to identify anomalies and outliers; and (4) determine inputs for MEPDG analysis.

3.2.5.1 Extract and Review the Pavement Performance Data

Distress and roughness indicator survey data included in the SCDOT ITMS and collected from the SCOT Division of Traffic Engineering were reviewed to determine their consistency with the MEPDG predicted values. Individual distress data were collected for the 20 selected pavement sections (see Section 2.3.4). For each distress (i.e., fatigue cracking, longitudinal cracking, and transverse cracking), quantities of different severities (i.e., low, medium, and high) were summed and the highest magnitude of each distress from each section was taken for MEPDG use. In addition, average IRI and rut information were compiled and reviewed for each pavement section. For AC pavements, IRI and rutting values measured by the SCDOT are assumed to have the same data format with MEPDG, which have units of in./mi and in., respectively. Note that the measured fatigue cracking, longitudinal cracking, and transverse cracking data were collected by visual-manual surveys (i.e., visual survey with tape measurement) and reported as a percent of the lane area. However, it is difficult to distinguish top-down and bottom-up fatigue cracking by visual inspection without performing trench studies or pavement coring. It was deemed acceptable to use the currently available distress data for the Phase I study herein; however, trench studies are recommended to obtain more accurate distress data (i.e., to identify the source of the distress and distress propagation) and distinguish between top-down and bottom-up fatigue cracking, and among AC rutting, base rutting, and subgrade rutting for Phase II of the project.

For PCC pavements, SCDOT measures the same distresses as for AC pavements. Therefore, IRI, fatigue cracking, and transverse cracking information are available for PCC pavements. Transverse joint faulting is not measured by SCDOT, therefore it is recommended to measure PCC pavements distresses (i.e., JPCP transverse slab cracking, JPCP mean transverse joint faulting, CRCP punchouts) in accordance with the MEPDG requirements in the Phase II project.

3.2.5.2 Compare Distress Magnitudes to Trigger Values

According to MEPDG (AASHTO, 2010), the average maximum distress values from the sampling templates should exceed at least 50 percent of the design criteria. If the maximum distress values are significantly lower than the agency's design criteria for that distress, the accuracy and bias for the distress prediction or transfer function may not be well defined at the values that trigger major rehabilitation. Table 3.3 summarizes the average, maximum and minimum distress values for each performance indicator for the 20 selected pavement sections as compared to the trigger values (i.e., design criteria) for major rehabilitation per AASHTO (2010).

Table 3.3 Simplified Sample Templates

Pavement Type	Performance Indicator	Design Criteria per AASHTO (2010)	Maximum Value Measured for Each Section				Probability Exceeding Trigger Value
			Average Max Value	Lowest Max Value	Largest Max Value	Standard Deviation of Max Values	
AC	IRI, in./mi	172	79	19	170	21	0.0
	Rut Depth, in.	0.75	0.18	0.06	0.48	0.07	0.0
	Fatigue Cracking, %	25	13.0	0.0	107.3	17.5	24.5
	Transverse Cracks, ft/mi	1000	151	0	1200	240	0.0
PCC	IRI, in./mi	172	101	51	180	18	0.0
	Transverse Crack, % slabs	15	10.7	0.0	39.3	9.3	32.4

Table 3.3 shows that most of the average maximum measured distress values are significantly less than the AASHTO (2010) design criteria trigger value. The probability of exceeding the trigger values found using the probability distribution test at 95% confidence

interval for each performance indicator shows the probability is zero for all performance indicators except AC fatigue cracking and PCC transverse cracking. This indicates that the trigger values shown in Table 3.3 are too high for four of the performance indicators because the probability of exceeding the trigger value is zero. Thus, the pavement will be overdesigned for those performance indicators. However, since the SCDOT has not defined trigger values for SC pavements, the AASHTO (2010) trigger values were used herein for the Phase I analysis. Evaluating reasonable trigger values for SC should be part of the scope of the Phase II project.

3.2.5.3 Evaluate Distress Data to Identify Outliers

Pavement roughness (IRI) and distress data (i.e., rutting, fatigue cracking, longitudinal cracking, and transverse cracking) were collected for different years between 2001 and 2014 for the 20 selected pavement sections (see Section 2.3.4). The measured distress data for all roadway sections were evaluated and checked for anomalies and outliers. This evaluation was limited to visual inspection of the trends in the data to ensure that the changes in each distress with time were reasonable and there were no irrational trends in the data.

3.2.5.4 Inputs to the MEPDG for Each Input Category

The input data extracted from the SCDOT databases and files for MEPDG was summarized in Section 2.3.1 for climate, Section 2.3.2 for traffic, Section 2.3.3 for materials, and Section 2.3.4 for pavement performance. In addition to this data, one of the critical input parameters required for MEPDG local calibration of the roughness model is the initial IRI after construction. Initial IRI information was not available within the SCDOT; therefore, this value was estimated before the preliminary calibration.

The MEPDG default value for initial IRI is 63 in./mi for both AC and PCC pavements, which has been shown to produce poor calibration results (Souliman et al., 2010). Therefore, the initial IRI values were estimated separately by back-predicting the trend of available IRI information for AC and PCC pavements. Figure 3.1 and Figure 3.2 show the trend of IRI for AC and PCC pavements, respectively, which have been developed using the available IRI information for the 20 selected pavement sections. Therefore, an initial IRI of 60 in./mi and 80 in./mi were taken as preliminary estimates for AC and PCC pavement design, respectively. Further study with additional pavement sections is needed in Phase II to finalize these values. The 100% pay line in the SCDOT specifications, as well as different target values for different road types (i.e., secondary, interstate), will need to be considered. Note that a MEPDG local calibration study performed for Arizona conditions (Souliman et al., 2010) estimated different initial IRI for different pavement sections. However, in this study a single initial IRI was estimated for all pavements of similar type (i.e., AC or PCC) per discussions with the SCDOT Pavement Design Group.

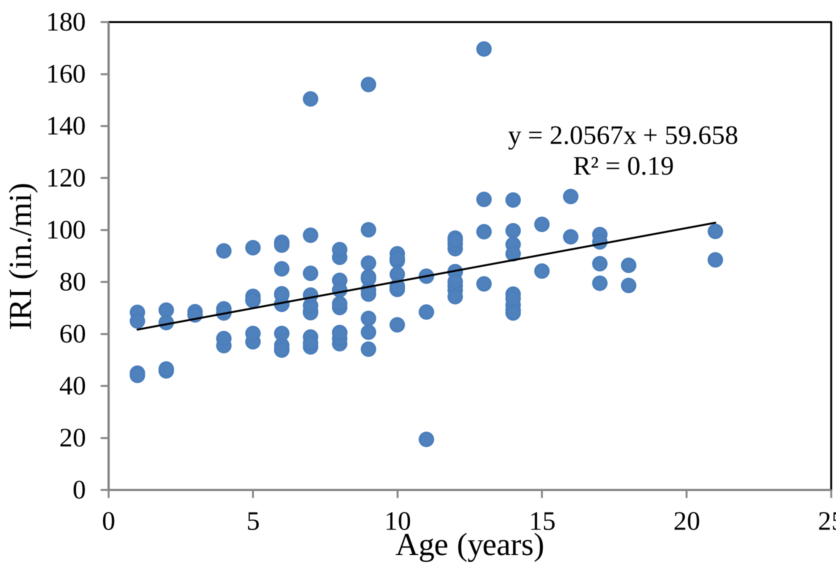


Figure 3.1 Estimation of Initial IRI for AC Pavements

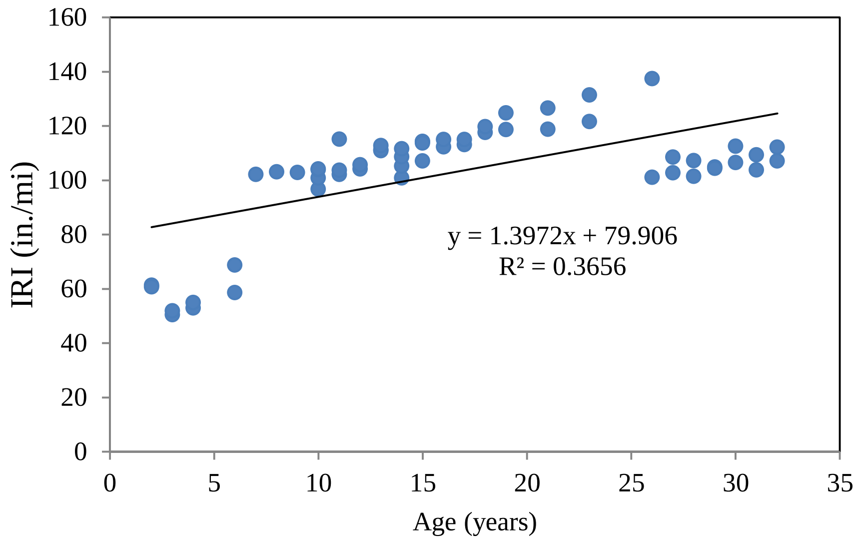


Figure 3.2 Estimation of Initial IRI for PCC Pavements

3.2.6 Step 6: Conduct Field and Forensic Investigation

To measure the rutting of individual layers, and to determine the location of crack initiation or the direction of crack propagation, forensic investigations are required. For the Phase I of the study presented herein, the SCDOT decided to accept the assumptions and conditions included in the MEPDG for the global calibration effort. Therefore, no forensic investigations were conducted beyond the general observations made by SCDOT and USC personnel during site visits to each of the 20 selected pavement sections (see Section 2.2). Figure 3.3 shows one of the site visits where pavement design engineer is looking for cracking in a PCC section. Information gathered from sites visits included pavement type, any overlay or resurfacing, let date compatibility with the design file, number of lanes, and posted speed limit.

3.2.7 Step 7: Assess Local Bias from Global Calibration Factors

For Step 7, preliminary MEPDG analysis was performed using the global calibration values (see Table 3.4) to predict the performance indicators for each of the 20 selected pavement sections. The results from the analysis using the AASHTOWare Pavement ME Design Software for the pavement sections of US-321 (Orangeburg), US-521 (Georgetown), and SC-93 (Pickens) will be presented to illustrate the process. Analysis was performed for 13, 12, and 15 years of design life, respectively. Different analysis periods were used because analysis was run for the period from the construction year of the pavement section to the current year of 2016. A summary of the distress prediction after performing the analysis using the global calibration values and MEPDG default target distresses and reliability values is shown in Tables 3.5(a) for Orangeburg, Table 3.5(b) for Georgetown, and Table 3.5(c) for Pickens. As shown, the predicted distresses are below the specified target for all 3 pavement sections. Furthermore, MEPDG predicted that AC bottom-up fatigue cracking, AC top-down fatigue cracking, and AC thermal cracking were significantly lower than the specified target values at 50% reliability.

Figure 3.3 Site Visit on SC 80 in Spartanburg, SC

Table 3.4 Summary of the Global Calibration Coefficients

Pavement Type	MEPDG Models	Calibration Coefficients	Global Calibration Coefficients
AC Pavement	Rutting Model	β_{r_1}	1
		β_{r_2}	1
		β_{r_3}	1
		β_{GB}	1
		β_{SG}	1
	Bottom-up Fatigue or Alligator Cracking Model	β_{f_1}	1
		β_{f_2}	1
		β_{f_3}	1
		C_1	1.00
		C_2	1.00
		C_4	6000
	Top-down Fatigue or Longitudinal Cracking Model	β_{f_1}	1
		β_{f_2}	1
		β_{f_3}	1
		C_1	7.00
		C_2	3.5
		C_4	1000
	AC Transverse Cracking Model	k_t $(Level\ 1)$	5.0
		k_t $(Level\ 1)$	1.5
		k_t $(Level\ 1)$	3.0
		β_t	1
PCC Pavement	JPCP Transverse Cracking Model	C_1	2.0
		C_2	1.22

Table 3.5(a) Distress Prediction Summary (Orangeburg)

Distress Type	Distress at Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mi)	172.00	92.48	50.00	99.86	Pass
Permanent Deformation- Total Pavement (in.)	0.75	0.52	50.00	99.39	Pass
AC Bottom Up Fatigue Cracking (% Lane Area)	25.00	3.43	50.00	94.19	Pass
AC Thermal Cracking (ft/mi)	1,000.00	1.00	50.00	100.00	Pass
AC Top Down Fatigue Cracking (ft/mi)	2,000.00	958.00	50.00	68.35	Pass
Permanent Deformation - AC only (in.)	0.25	0.18	50.00	86.76	Pass

Table 3.5(b) Distress Prediction Summary (Georgetown)

Distress Type	Distress at 50% Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mi)	172.00	92.94	50.00	99.85	Pass
Permanent Deformation- Total Pavement (in.)	0.75	0.32	50.00	99.00	Pass
AC Bottom Up Fatigue Cracking (% Lane Area)	25.00	0.00	50.00	100.00	Pass
AC Total Fatigue Cracking: Bottom Up + Reflective (% Lane Area)	25.00	0.00	50.00	100.00	Pass
AC Thermal Cracking (ft/mi)	1,000.00	1.00	50.00	100.00	Pass
AC Top Down Fatigue Cracking (ft/mi)	2,000.00	0.00	50.00	100.00	Pass
Permanent Deformation - AC only (in.)	0.25	0.15	50.00	96.67	Pass

Table 3.5(c) Distress Prediction Summary (Pickens)

Distress Type	Distress at Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mi)	172.00	89.38	50.00	99.95	Pass
Permanent Deformation- Total Pavement (in.)	0.75	0.39	50.00	100.00	Pass
AC Bottom Up Fatigue Cracking (% Lane Area)	25.00	0.35	50.00	100.00	Pass
AC Thermal Cracking (ft/mi)	1,000.00	1.00	50.00	100.00	Pass
AC Top Down Fatigue Cracking (ft/mi)	2,000.00	2.16	50.00	100.00	Pass
Permanent Deformation - AC only (in.)	0.25	0.12	50.00	99.88	Pass

The next step is to compare the predicted distress with the original distress for a greater number of samples from across the state (i.e., all 14 AC pavements and all 6 PCC pavements) to minimize the local bias. Firstly, the null hypothesis was checked for the entire sampling template shown in Table 3.6. The null hypothesis is that the average residual error or the bias is zero for a specified confidence level or level of significance. A 90% confidence level per AASHTO (2008) was used in this study.

$$H_0: \sum_{i=1}^{n}(y_{Measured} - x_{Predicted})_i = 0 \qquad 3.1$$

Table 3.6 shows the bias (e_r, difference between mean measured and mean predicted performance indicators) for each performance indicator for the entire sampling template (i.e., 14 AC and 6 PCC pavement sections). It also shows the standard error of estimates (S_e, standard deviation of the residual errors), S_e / S_y (S_y, standard deviation of the measure values), and coefficients of determination. The null hypothesis is rejected for all indicators except for AC pavement transverse cracking. This means that bias is high for the measured and predicted values except for AC pavement transverse cracking after using the global calibration factors. Therefore, local bias needs to be eliminated by performing local calibration.

The predicted and measured values for each AC performance indicator using the global calibration values are compared to local calibration values of unity in Figure 3.4 to Figure 3.8 for IRI, rut depth, fatigue cracking, and longitudinal cracking, respectively. Figure 3.4(a) shows the measured versus predicted IRI using the global calibration factors and local calibration values as unity; and, indicates that most of the data points are at or near the line of equity. The intercept estimator is significantly different from 0, and the slope estimator is significantly different from 1. The coefficient of determination of 0.11 indicates that for 11% of the total data the model predicts well. Figure 3.4 (b) shows that IRI residual errors are negative (e_r=-4.84) with a low standard error of estimates (S_e=19.83) and the slope of the residual errors versus predicted values (x_i=-0.17) is relatively constant and close to zero. That means the precision of the prediction model is reasonable but the accuracy is poor (i.e., large bias).

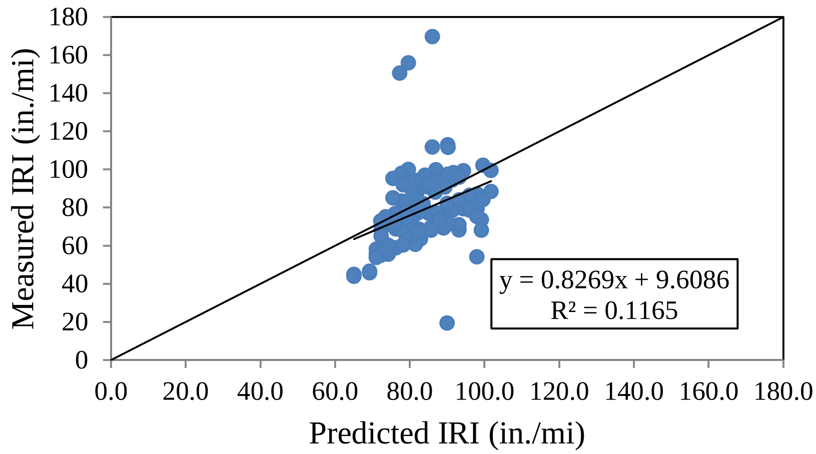
Table 3.6 Summary of Statistical Parameters for Preliminary Analysis with Global Calibration Values

Pavement Type	Performance Indicator	Bias, e_r	Standard Error, S_e	S_e/S_y	R^2	Hypothesis; H_0: y_i-x_i = 0	Comments
AC	Rutting	-0.19	0.13	1.72	Poor	Reject; p = 0	Bias
	Fatigue Cracking	12.6	17.25	0.99	0.11	Reject; p = 0	Bias
	Longitudinal Cracking	-12.87	115.30	5.86	Poor	Accept; p = 0.18	No Bias
	Transverse Cracking	25.42	354.23	1.47	Poor	Accept; p = 0.56	No Bias
	IRI	-4.84	19.83	0.94	0.11	Reject; p = 0.013	Bias
PCC	Transverse Cracking	10.74	9.33	1	1	Reject; p = 0	Bias
	IRI	20.05	19.1	0.95	0.27	Reject; p = 0	Bias

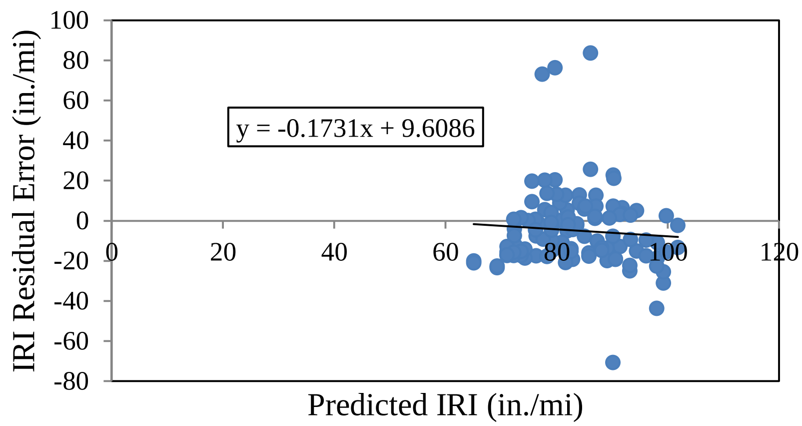
Note: Poor means that the model did not explain variation in the measured data within and between the pavement sections.
Bias, $e_r = y_i - x_i$
y_i = Measured Value; Standard Deviation of the measured values.
x_i = Predicted Values

As shown in Figure 3.5, the bias is large (p = 0) for the measured and predicted rut depth, coefficient of determination is poor ($R^2 = 0.0012$), and MEPDG overestimates the predicted rut depth. In contrast, MEPDG under predicts the total fatigue cracking (Figure 3.6). Also, large dispersion exists (i.e., coefficients of determination are poor) between the measured and predicted transverse cracking, and longitudinal cracking, as shown in Figure 3.7 and Figure 3.8, respectively. As SCDOT reports longitudinal cracking in percentages, to compare with the MEPDG predicted longitudinal cracking (in ft/mi) it was assumed that percent longitudinal cracking is equal to the ft/mi cracking.

In summary, all performance indicators for AC pavement show a similar trend (i.e., bias). Moreover, almost all data points for each distress were found either above or below the line of equity when using the global calibration values. MEPDG global calibration values resulted in bias for IRI, rutting, fatigue cracking and longitudinal cracking as the null hypothesis was rejected ($p < 0.05$). The null hypothesis was accepted only for the transverse cracking ($p > 0.05$).



(a)



(b)

Figure 3.4 Predicted and Measured IRI Using the Global Calibration Factors (AC)

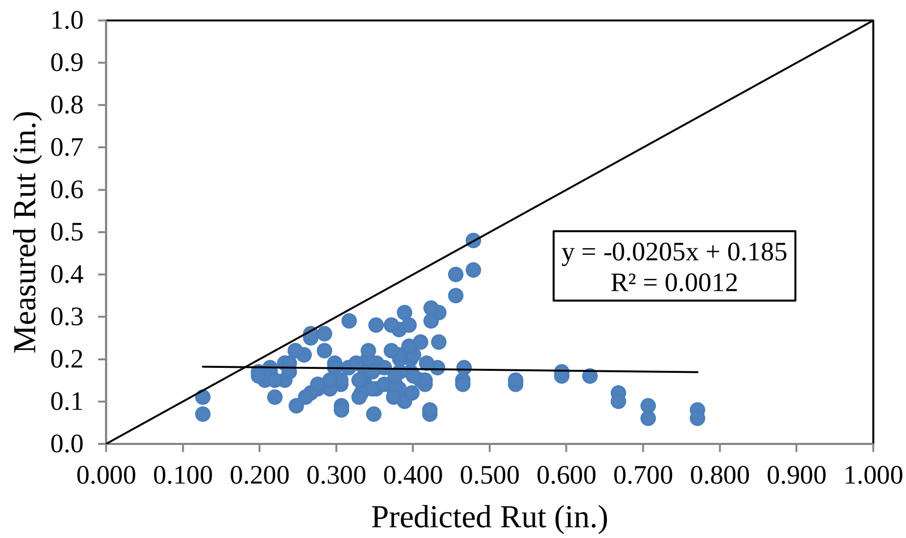


Figure 3.5 Predicted and Measured Rut Depth Using the Global Calibration Factors (AC)

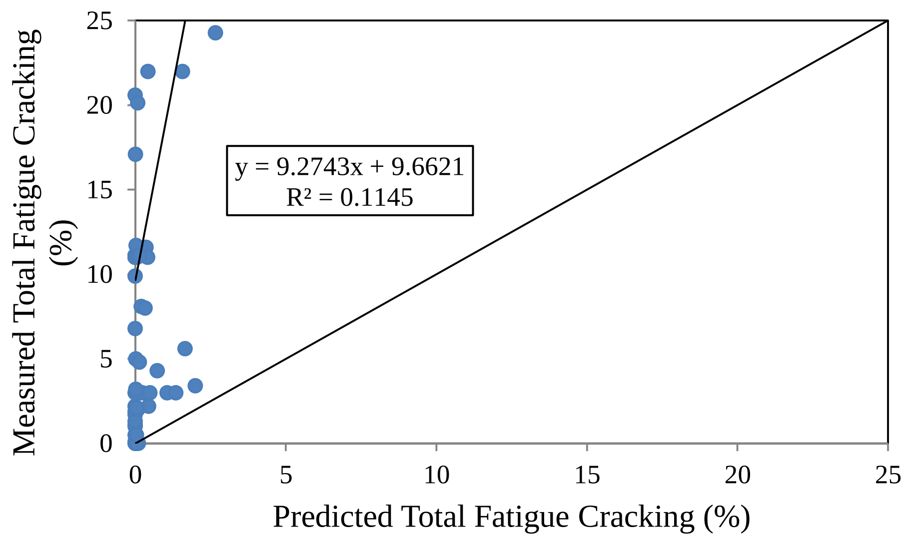


Figure 3.6 Predicted and Measured Fatigue Cracking Using the Global Calibration Factors (AC)

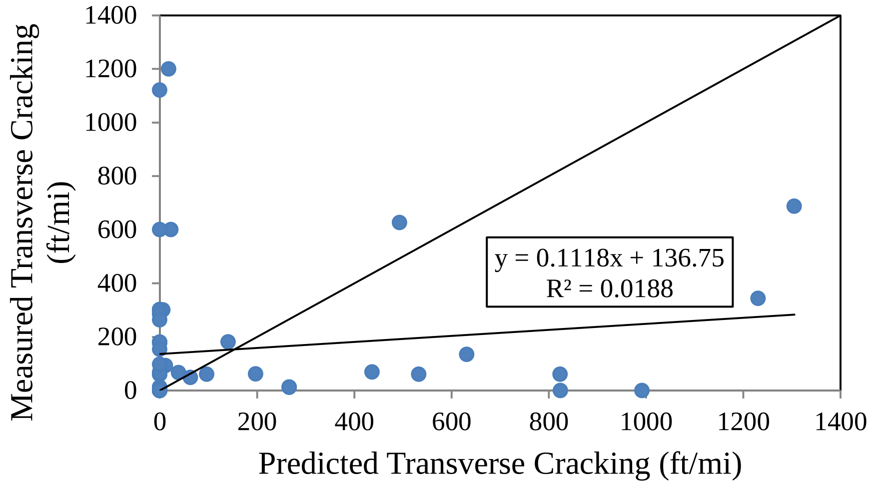


Figure 3.7 Predicted and Measured Transverse Cracking Using Global Calibration Factors (AC)

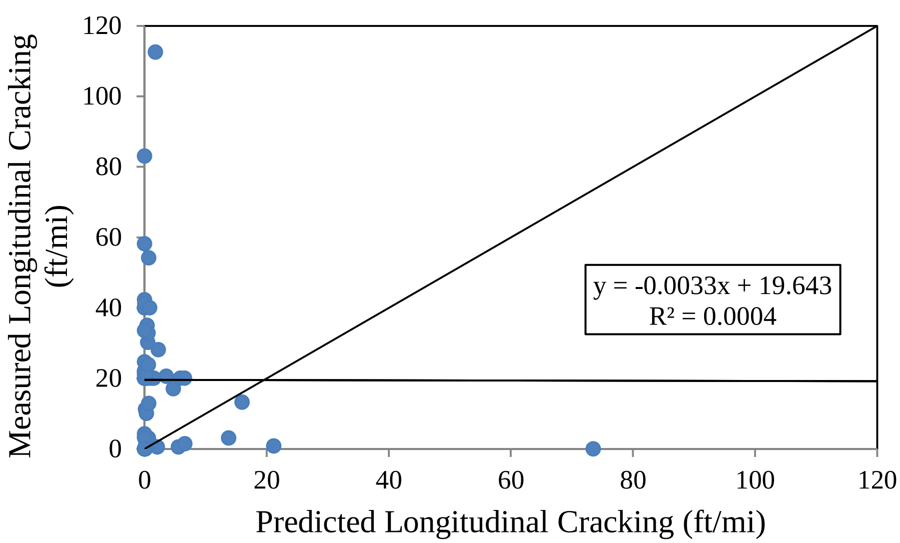
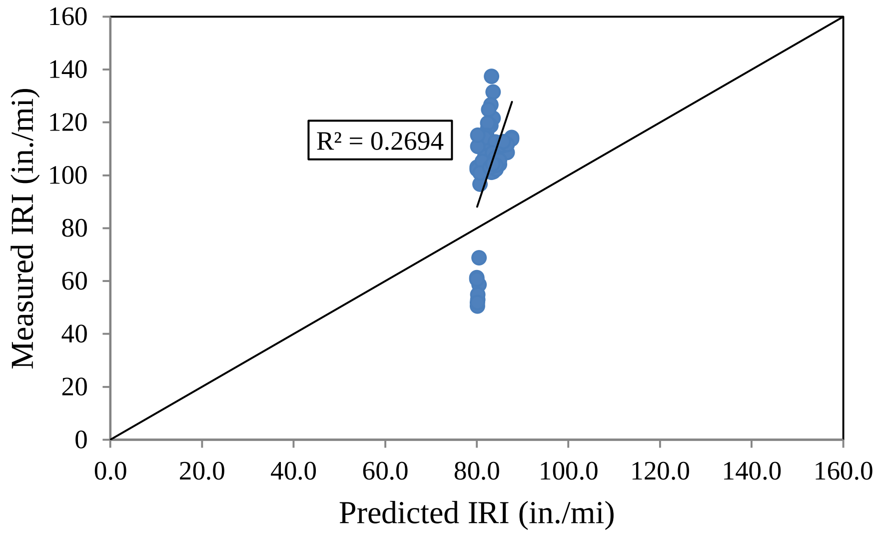
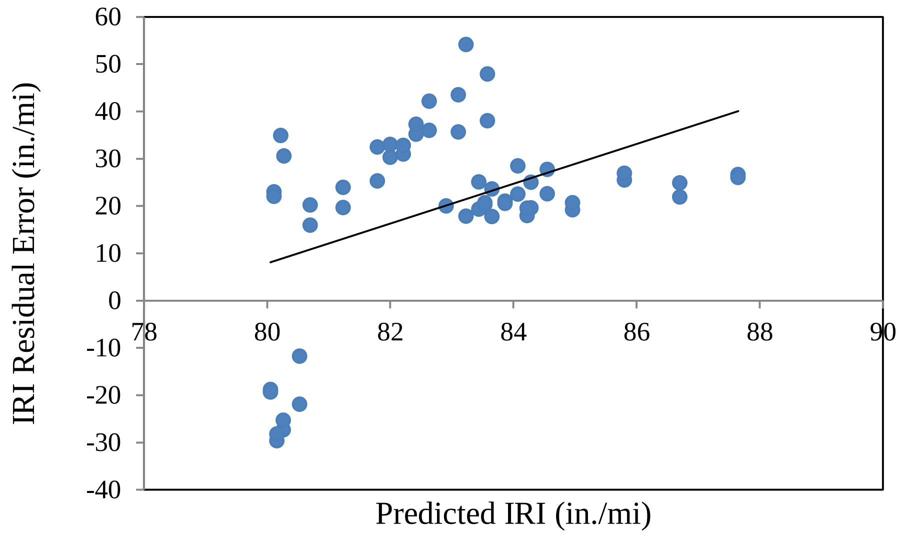


Figure 3.8 Predicted and Measured Longitudinal Cracking Using Global Calibration Factors (AC)

Figure 3.9 and Figure 3.10 compare the predicted and measured values for PCC pavement IRI and transverse cracking, respectively, using the global calibration values and local calibration values of unity. The results for both IRI and transverse cracking show bias for PCC pavements just as bias was shown for AC pavements (also see Table 3.6). No to nil transverse cracking was predicted by MEPDG in Figure 3.10.



(a)



(b)

Figure 3.9 Predicted and Measured IRI Using Global Calibration Factors (PCC)

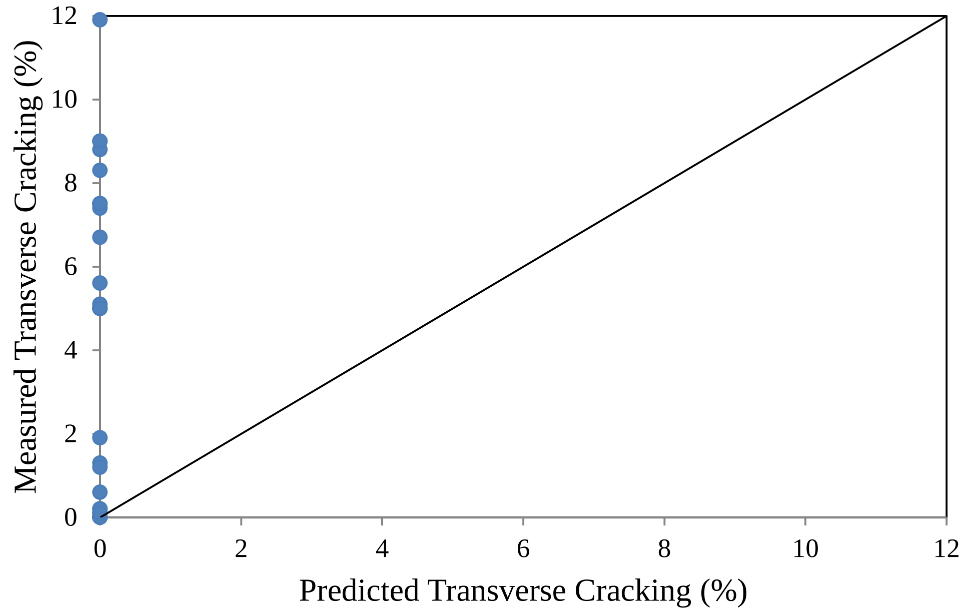


Figure 3.10 Predicted and Measured Transverse Cracking Using Global Calibration Factors (PCC)

3.3 SUMMARY OF PRELIMINARY GLOBAL CALIBRATION

The following conclusions were made based on the preliminary analysis of the 20 pavement sections using global calibration values:

- Bias has been found in all predicted functions except AC pavement transverse cracking.
- MEPDG over predicted the measured rut depths for AC pavements. Poor correlation (R^2= 0.0012) was found between the predicted and measured rut depth.
- MEPDG under predicted the measured fatigue cracking for AC pavements. Most of the data showed nil or no predicted fatigue cracking.

- MEPDG showed the least bias for transverse cracking among all the pavement distresses. However, poor correlation (R^2= 0.0188) was observed between the measured and predicted distress.
- For both the AC pavement total fatigue cracking and IRI the coefficients of determination (R^2) were found to be around 0.11, which means 11% of the total data were well predicted by the models.
- MEPDG predicted IRI for PCC pavements very close to the initial IRI (80 in./mi) and showed extensive bias (p = 0).
- MEPDG predicts no PCC transverse cracking using the global calibration factors.

In summary, bias was found between the measured and predicted distresses for almost all of the distress prediction models. Therefore, bias needs to be eliminated from all distress prediction models using local calibration values for both AC and PCC pavements.

CHAPTER 4

PRELIMINARY STUDY ON THE LOCAL CALIBRATION OF THE MEPDG FOR SOUTH CAROLINA

4.1 INTRODUCTION

The models used in the Mechanistic-Empirical Pavement Design Guide (MEPDG) were originally calibrated using national Long-Term Pavement Performance (LTPP) data from the Strategic Highway Research Program (SHRP). Because the calibration included data from areas that have significant differences in materials, climate, traffic loading, and construction practices from South Carolina (SC), the procedure may not be accurate for SC conditions. Therefore, it is necessary to calibrate the MEPDG distress prediction models for SC local conditions.

In this study, preliminary analysis was performed to locally calibrate the distress models of Asphalt Concrete (AC) pavement rutting and fatigue cracking (i.e., bottom-up and top-down) using the historical data obtained for the 14 AC pavement sections summarized in Table 2.1. Also, preliminary analysis was performed to locally calibrate the distress models of Jointed Plain Concrete Pavement (JPCP) transverse slab cracking using the historical data obtained for the 6 PCC pavement sections also shown in Table 2.1. Preliminary local calibration of the distress models for AC transverse cracking, AC roughness, and JPCP roughness was also initiated. As discussed in Chapter 2, not all of the necessary data was available in the SCDOT files and databases, and the quality of the distress data is uncertain; thus, the local calibration factors presented herein are preliminary, and are not recommended to be used for design. Further work is required in a Phase II study.

4.2 STEP-BY-STEP PROCEDURE OF MEPDG LOCAL CALIBRATION

As described in Section 3.2, there are 11 steps in the MEPDG local calibration procedure per the Local Calibration Guide (AASHTO 2010). Steps 1 to Step 7 were demonstrated in Chapter 3 using the collected data described in Chapter 2 with global calibration factors. Steps 8 to Step 11 are described below.

4.2.1 Step 8: Eliminate Local Bias of Distress Prediction Models

Most of the globally calibrated transfer functions were found to be biased based on the probability study in Chapter 3 (see Table 3.6). Therefore bias for different distress prediction models needs to be eliminated by considering 1 of the 3 following possibilities (AASHTO, 2010):

- If the precision of the prediction model is reasonable (i.e., based on intercept and slope estimator) but the accuracy is poor (i.e., bias is high), it requires the least level of effort and the fewest number of runs or iteration of the MEPDG to reduce the bias.
- If the accuracy of the prediction model is reasonable (i.e., bias is low and relatively constant with time) but the precision is poor (i.e., residual error has wide dispersion from positive to negative values), it requires more runs and a higher level of effort to reduce dispersion of the residual errors.
- Is the precision of the prediction model and the accuracy are both poor, then highest level of effort and many more runs are required to reduce bias and dispersion.

As discussed in Chapter 3, both the accuracy (i.e., large bias) and the precision (i.e., based on intercept and slope estimator) of the prediction models were poor, except for the AC transverse cracking model. Therefore, the highest level of effort and numerous runs is required to reduce the bias and dispersion. In this Phase I study, the primary goal of the local calibration is to reduce the bias. It is recommended to both reduce the bias and increase the precision by performing many more runs and using more Level 1 inputs in Phase II.

4.2.2 Step 9: Assess Standard Error of the Estimate

After the bias is eliminated or reduced for each transfer function, the standard errors of the estimates (S_e) between the global calibration (Table 3.6) and the local calibration need to be compared. It is expected that the S_e values will be lower for the locally calibrated transfer functions than for globally calibrated functions.

4.2.3 Step 10: Reduce Standard Errors of the Estimates

If based on statistical tests it is found that the residual error is dependent on some other parameters or material property for the pavement section, the standard errors of the estimates should be reduced by considering those parameters. If no correlation between the residual error and the other parameters or material property is identified, the S_e from Step 9 should be taken as the final values. For Phase I, no further studies were performed to reduce the standard errors as the S_e values were found close or below the reasonable values recommended by AASHTO (AASHTO, 2008). Reasonable S_e values for IRI, alligator cracking, longitudinal cracking, rutting, and transverse cracking, are 17 in./mi, 7%, 600 ft/mi, 0.10 in, and 250 ft/mi, respectively. However, further work in a Phase II study is recommended to obtain standard errors that are all below the reasonable values.

4.2.4 Step 11: Interpretation of Results and Deciding on Adequacy of Calibration Factors

For Step 11, the adequacy of the calibration factors needs to be assessed in order to make recommendations for implementation. The Phase I study results are assessed herein and recommendations for further study in Phase II are presented in Chapter 5.

4.3 PRELIMINARY MEPDG LOCAL CALIBRATION FOR FLEXIBLE PAVEMENT

Preliminary local calibration was performed for the AC pavement transfer functions: rutting models, and fatigue cracking models (bottom-up/fatigue cracking, top-down/longitudinal cracking) because large bias was found for these models (Table 3.6). Firstly, local bias was eliminated by performing the local calibration analysis and optimizing the calibration factors (Step 8). Optimization runs were made using the AASHTOWare Pavement ME Design software (Version 2.2) or using Microsoft Excel Solver (Microsoft Office, 2010). The optimization method for each of the calibration factors will be described in the subsequent subsections. Secondly, the standard errors of the estimates were determined for these distress models (Step 9). The standard errors of the estimates were not reduced further in this study because they were found to be close or below the reasonable values (Step 10). Finally, results have been interpreted and recommendations are made based on the preliminary local calibration analysis (Step 11). The methodology used to perform the preliminary local calibration for each of the AC distress models is described in the following subsections.

4.3.1 Calibration of Rutting Models

Rutting is the pavement surface depression in the wheel paths and is caused by the permanent deformation of the Hot Mix Asphalt (HMA), unbound layers, and foundation soil. It originates from the lateral movement of pavement material due to cumulative traffic loading. The approach used in the MEPDG to calculate total rut depth is based upon calculating incremental distortion or rutting within each sub-layer. MEPDG uses the following equation to calculate total rutting:

$$RD = \sum_{i=1}^{n} \varepsilon_{p(i)} h_i \qquad 4.1$$

where $RD =$ total rut depth,

i = sub-layer number,

n = total number of sub-layers,

$\varepsilon_{p(i)}$ = plastic strain in sub-layer i, and

h_i = thickness of sub-layer i.

The MEPDG permanent deformation models for HMA layers and for unbound base and subgrade layers are shown in Equation 4.2 and Equation 4.3 respectively.

$$\Delta_{p(HMA)} = \varepsilon_{p(HMA)} h_{HMA} = \beta_{r_1} k_z \varepsilon_{r(HMA)} 10^{k_{r_1}} N^{k_{r_2}\beta_{r_2}} T^{k_{r_3}\beta_{r_3}} \qquad 4.2$$

where $\Delta_{p(HMA)}$ = Accumulated permanent or plastic vertical deformation in the HMA layer, in.,

$\varepsilon_{p(HMA)}$ = Accumulated permanent or plastic axial strain in the HMA layer, in./in.,

$\varepsilon_{r(HMA)}$ = Resilient or elastic strain at the mid-depth of each HMA layer, in./in.,

h_{HMA} = Thickness of HMA layer, in.,

N = Number of axle-load repetitions,

T = Temperature of HMA layer at mid-depth, °F,

$k_{r_1}, k_{r_2}, k_{r_3}$ = Global calibration parameter, $k_{r_1} = -3.35412$, $k_{r_2} = 0.4791$, $k_{r_3} = 1.5606$

$\beta_{r_1}, \beta_{r_2}, \beta_{r_3}$ = Local calibration factors (for global calibration these factors are all set to 1.0),

k_z = Depth confinement factor = $(C_1 + C_2 D)(0.328196)^D$

$C_1 = -0.1039(H_{HMA})^2 + 2.4868 H_{HMA} - 17.342$

$C_2 = 0.0172(H_{HMA})^2 - 1.733 H_{HMA} + 27.428$

where D = Depth below the surface, in., and H_{HMA} = Total HMA thickness, in.

$$\Delta_{p(soil)} = \beta_{s_1} k_{s_1} \varepsilon_v h_{soil} \left(\frac{\varepsilon_0}{\varepsilon_r}\right)\left(e^{-\left(\frac{\rho}{N}\right)^{\beta}}\right) \qquad 4.3$$

where $\Delta_{p(soil)}$ = Permanent or plastic deformation for the unbound layer, in.,

N = Number of axle-load repetitions,

ε_0 = Intercept determined from laboratory repeated load permanents deformation test, in./in.

ε_r = Resilient strain imposed in laboratory test to obtain material properties ε_0, ε, and ρ, in./in.,

ε_v = Average vertical strain, in./in.,

h_{soil} = Thickness of the unbound layer, in.,

k_{s_1} = Global calibration coefficients; $k_{s_1} = 1.673$ for granular materials and 1.35 for fine grained materials, and

β_{s_1} = Local calibration constant for the rutting in the unbound layers; ($\beta_{s_1} = \beta_{GB}$ for unbound granular base; and $\beta_{s_1} = \beta_{SG}$ for subgrade material), the local calibration constant was set to 1.0 for the global calibration effort.

Therefore, the rutting model of the AC layer has 3 local calibration coefficients: β_{r_1}, β_{r_2}, and β_{r_3}, while rutting model for the unbound layers has 2 calibration coefficients: β_{GB} and β_{SG} . In this study, the rutting models for the AC and unbound layers are locally calibrated to determine preliminary values of these calibration factors for SC, comparing the measured and predicted total rut depth. A total number of 14 AC pavement sections were used in the calibration process, which include 109 data points as shown in Appendix F. Data points include measured rutting on the 14 selected pavement sections for the past 10 years. Rutting models were not calibrated for the individual layers because no information was found for individual AC and unbound base and subgrade layer rutting. In Phase II of the project, it is recommended to collect periodic rut data of the individual layers by doing trench studies.

The calibration was performed by varying different coefficients in the rutting models to reduce the sum of squared errors between the predicted and measured total rutting. Firstly, optimization runs were made using the AASHTOWare Pavement ME Design software (Version 2.2) by using different sets of β_{r_2} and β_{r_3} by keeping the other local calibration factors (β_{r_1}, β_{GB} and β_{SG}) constant. MEPDG analysis were run using trial values of different combination of β_{r_2} and β_{r_3} and the values which gave the minimum sum of squared errors between predicted and measured total rutting were taken as an initial estimation of these coefficients. In the next step, the obtained values of β_{r_2} and β_{r_3} were kept constant, and optimization was performed by changing the values of β_{r_1}, β_{GB} and β_{SG} using Microsoft Excel Solver (Microsoft Excel, 2010). The objective was to minimize the sum of squared errors by optimizing these 3 factors. If the local bias of rutting prediction models was eliminated and the standard errors of the estimates were reduced when compared to the global calibration factors, local calibration factors were then obtained. The preliminary local calibration factors obtained are: β_{r_1}= 0.240, β_{r_2}= 1, β_{r_3}= 1, β_{GB} = 2.979, β_{SG} = 0.393. Therefore, the preliminary local calibrated AC rutting model, unbound granular base, and subgrade model for SC are shown in Equation 4.4, Equation 4.5, and Equation 4.6, respectively.

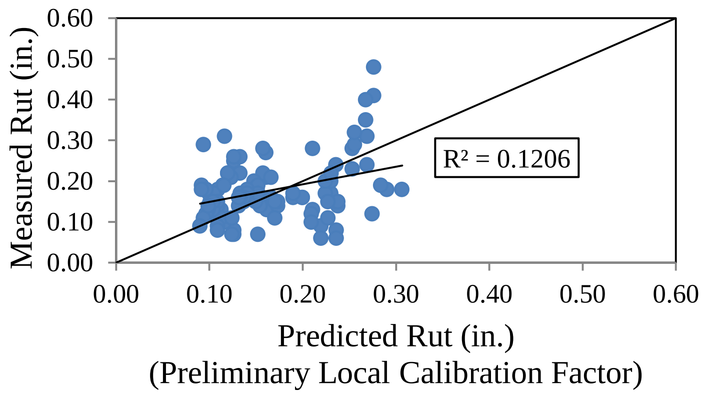
$$\Delta_{p(HMA)} = 0.240 k_z \varepsilon_{r(HMA)} 10^{-3.35412} N^{0.4791} T^{1.5606} \qquad 4.4$$

$$\Delta_{p(soil)} = 4.98387 \varepsilon_v h_{soil} (\frac{\varepsilon_0}{\varepsilon_r})(e^{-(\frac{\rho}{N})^{\beta}}) \qquad 4.5$$

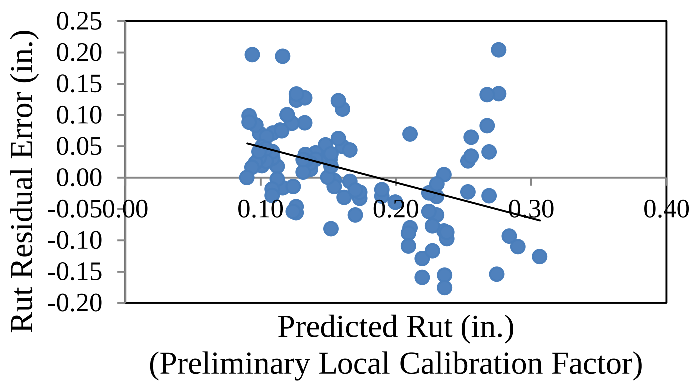
$$\Delta_{p(soil)} = 0.53055 \varepsilon_v h_{soil} (\frac{\varepsilon_0}{\varepsilon_r})(e^{-(\frac{\rho}{N})^{\beta}}) \qquad 4.6$$

As discussed in Chapter 3, the MEPDG analysis was performed using the global calibration values to predict the performance indicators for each selected pavement sections and the null hypothesis (Equation 3.1) was checked for the entire sampling template. In this chapter, the null hypothesis was checked again but this time using the preliminary local calibration factors.

Figure 4.1 compares the predicted and measured values for rutting using the preliminary local calibration values. Figure 4.1(a) compares the predicted and measured rutting while Figure 4.1(b) shows comparison between residual errors (e_r) and predicted values (x_i). As shown, after the preliminary local calibration, bias has been eliminated and the coefficient of determination has been improved for the rutting model. Using the preliminary local calibration factors the model explains 12% of the measured data ($R^2 = 0.12$), while using the global calibration factor the model did not explain the variation in the measured data ($R^2 =$ poor).



(a)



(b)

Figure 4.1 Predicted and Measured Rutting Using the Preliminary Local Calibration Factors

4.3.2 Calibration of Fatigue Cracking Models

Two types of load-related cracks (i.e., fatigue cracking) are predicted by MEPDG: alligator cracking (bottom-up) and longitudinal cracking (top-down). The MEPDG assumes that alligator (or area cracks) initiate at the bottom of the HMA layers and propagate to the surface with continued truck traffic; while longitudinal cracking are assumed to initiate at the surface. In this study, both the alligator cracking transfer function, FC_{Bottom}, and the longitudinal cracking transfer function, FC_{Top}, were preliminarily calibrated for SC.

There are 3 calibration coefficients: β_{f_1}, β_{f_2}, and β_{f_3}, which are used for both the alligator and longitudinal cracking models. In addition, there are 3 calibration coefficients (i.e., C_1,C_2, C_4) for each of the models. Both the alligator and longitudinal cracking models are a function of the cumulative damage index (DI):

$$DI = \sum(\Delta DI)_{j,m,l,p,T} = \sum(\frac{N}{N_{f-HMA}})_{j,m,l,p,T} \qquad 4.7$$

where n = Actual number of axle-load applications within a specific time-period,

j = Axle-load interval,

m = Axle load type (single, tandem, tridem, quad, or special axle configuration,

l = Truck type using the truck classification groups included in the MEPDG,

p = Month,

T = Median temperature for the five temperature intervals, °F, and

N_{f-HMA} = Allowable number of axle load applications for a flexible pavement

The allowable number of axle-load applications ($N_{f\text{-}HMA}$) needed for the incremental damage-index approach is:

$$N_{f-HMA} = 0.00432(C)(k_{f_1})\,\beta_{f_1}(\varepsilon_t)^{k_{f2}\,\beta_{f2}}(E_{HMA})^{k_{f3}\,\beta_{f3}} \qquad 4.8$$

where ε_t = Tensile strain at critical locations, in./in.,

E_{HMA} = Dynamic modulus of the HMA measured in compression, psi,

$k_{f_1}, k_{f_2}, k_{f_3}$ = Global calibration parameter = 0.007566, 3.9492, and 1.281, respectively,

$\beta_{f_1}, \beta_{f_2}, \beta_{f_3} =$ Local calibration factors (for global calibration these factors are all set to 1.0),

$C = 10^{M}$

$M = 4.84(\frac{V_{be}}{V_a+V_{be}} - 0.69)$

where V_{be} = Effective binder content, %

V_a = air void, %

The alligator cracking transfer function, FC_{Bottom}, and longitudinal cracking transfer function, FC_{Top}, are shown in Equation 4.9 and Equation 4.10 respectively.

$$FC_{Bottom} = \left(\frac{C_4}{1+e^{(C_1C_1'+C_2C_2'Log(DI_{Bottom}))}}\right) * (\frac{1}{60}) \qquad 4.9$$

where FC_{Bottom} = Bottom-up or alligator fatigue cracking, % of total lane area,

DI_{Bottom} = Cumulative damage index at the bottom of the HMA layers

$C_2' = -2.40874 - 39.748(1 + H_{HMA})^{-2.856}$

$C_1' = -2C_2'$

C_1, C_2, C_4 = Local calibration coefficients.

$$FC_{Top} = \left(\frac{C_4}{1+e^{(C_1-C_2Log(DI_{Top}))}}\right)(10.56) \qquad 4.10$$

where FC_{Top} = Top-down fatigue cracking or longitudinal cracking, ft/mi.,

DI_{Top} = Cumulative damage index near the top of the HMA surface,

C_1, C_2, C_4 = Local calibration coefficients.

In this study, preliminary calibration studies were performed for both alligator cracking (bottom-up) and longitudinal cracking (top-down) by comparing the measured to the predicted values. The 14 AC pavement sections in Table 2.2 were used in the calibration process, which include 67 data points (see Appendix F). Data points include measurement of distresses on the 14 selected pavements for the past several years. There are less data points available for fatigue cracking compared to the number of data points available for the rutting model because rut data were collected from both sides of the roadway sections; whereas, for cracking, single values were taken as representative of both sides of the roadway. The dissimilarity between rutting and distress data collection were found because of the differences in the data sources; rut and IRI data were collected from SCDOT ITMS; whereas, distress data were collected directly from the SCDOT Division of Pavement Management.

The preliminary calibration was performed by varying different coefficients in the fatigue cracking models in order to reduce the sum of squared errors between the predicted and measured total fatigue cracking (i.e., alligator and longitudinal cracking separately). Preliminary local calibration factors were determined stepwise using previous literature as guide (e.g., Jadoun and Kim, 2012; Souliman et al., 2010). Firstly, optimization runs were made using the AASHTOWare Pavement ME Design software (Version 2.2) by using different sets of β_{f_2}, and β_{f_3} and keeping other local calibration factors (i.e., β_{f_1}, C_1, C_2, C_4 for bottom-up, and C_1, C_2, C_4 for top-down) constant. MEPDG analyses were run using trial values of different combinations of β_{f_2} and β_{f_3}, and the values which gave the minimum sum of squared errors between predicted and measured bottom-up fatigue (i.e., alligator cracking) were taken for an initial estimation of these coefficients. Then, Microsoft Excel Solver (Microsoft Excel, 2010) was used to minimize the sum of squared errors of the measured and predicted alligator cracking by optimizing β_{f_1}, C_1 and C_2 coefficients in the alligator transfer function. As bias was eliminated using these 3 calibration coefficients, C_4 was not used in the optimization.

Next, optimization was performed to determine the preliminary local calibration factors for AC top-down fatigue cracking (i.e., longitudinal cracking). The first try of the optimization was made considering 2 of the 3 local calibration factors for the AC top-down fatigue cracking (i.e., C_1 and C_2). Microsoft Excel Solver was used to minimize the sum of squared errors of the measured and predicted longitudinal cracking by optimizing C_1 and C_2 in the longitudinal transfer function. However, bias was still found after optimizing these 2 factors. Therefore, the optimization process was still continued considering all 3 local calibration coefficients of longitudinal cracking (i.e., C_1, C_2, C_4). Finally, bias was eliminated for the AC top-down fatigue cracking model.

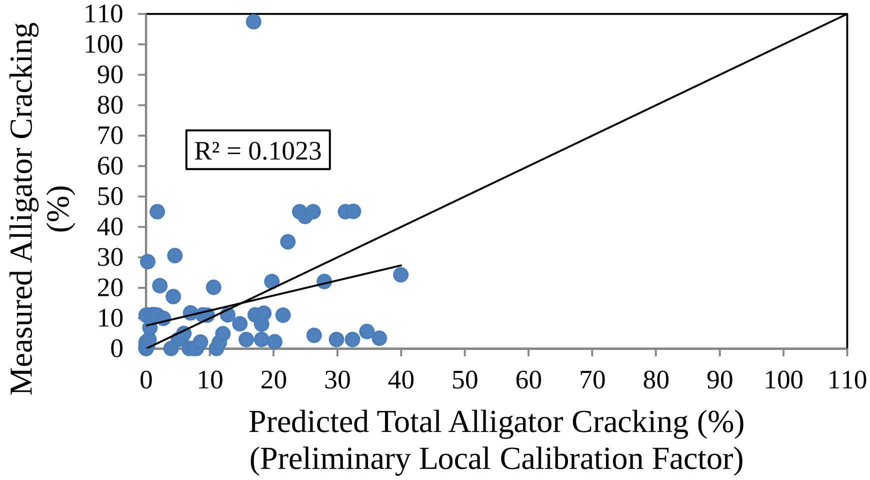
The obtained preliminary local calibration coefficients are β_{f_1}= 5, β_{f_2}= 0.8, and β_{f_3}= 0.8. The preliminary transfer function coefficients for alligator cracking were found as C_1= C_2=0.47 and the preliminary longitudinal transfer functions coefficients were found to be C_1= 0.2, C_2= 0.1, and C_4 = 3.97. The preliminary calibrated fatigue cracking MEPDG model, alligator cracking transfer function and longitudinal cracking transfer function for SC are shown in Equation 4.11, Equation 4.12, and Equation 4.13, respectively.

$$N_{f-HMA} = 0.00432(C).03783(\varepsilon_t)^{3.15936}(E_{HMA})^{1.0248} \tag{4.11}$$

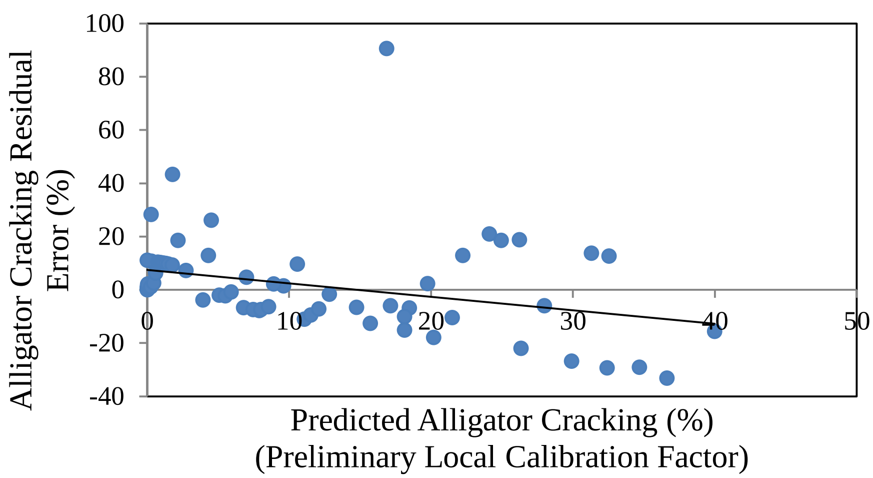
$$FC_{Bottom} = \left(\frac{6000}{1+e^{(0.47C_1'+0.47C_2'Log(DI_{Bottom}))}}\right)\left(\frac{1}{60}\right) \tag{4.12}$$

$$FC_{Top} = \left(\frac{3.97}{1+e^{(0.2-0.1Log(DI_{Top})}}\right)(10.56) \tag{4.13}$$

Figure 4.2 compares the predicted and measured values for alligator cracking using the preliminary local calibration values. Figure 4.2(a) compares the predicted and measured alligator cracking while, Figure 4.2(b) shows comparison between residual errors (e_r) and predicted values (x_i). As shown, after the preliminary local calibration, bias has been eliminated and the coefficient of determination has been improved for the alligator cracking model. Using the preliminary local calibration factors, the model explains 10% of the measured data ($R^2 = 0.10$); while using the global calibration factor, the model did not explain the variation in the measured data ($R^2 =$ poor).

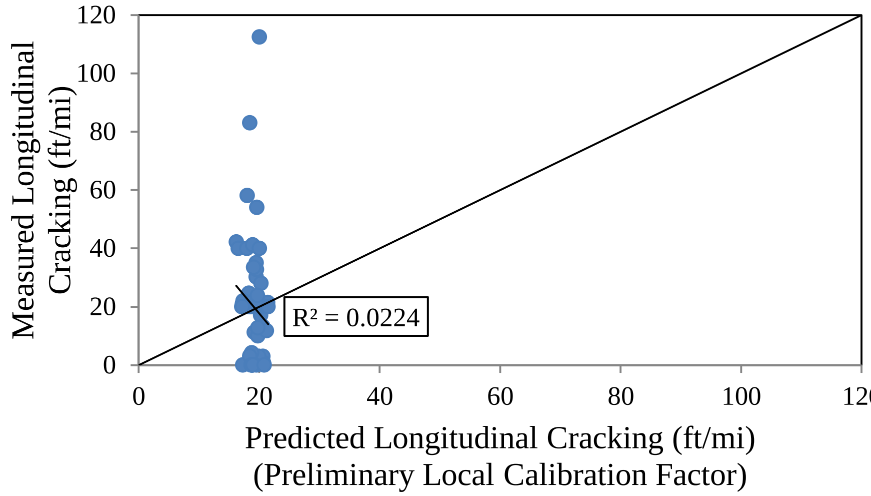


(a)

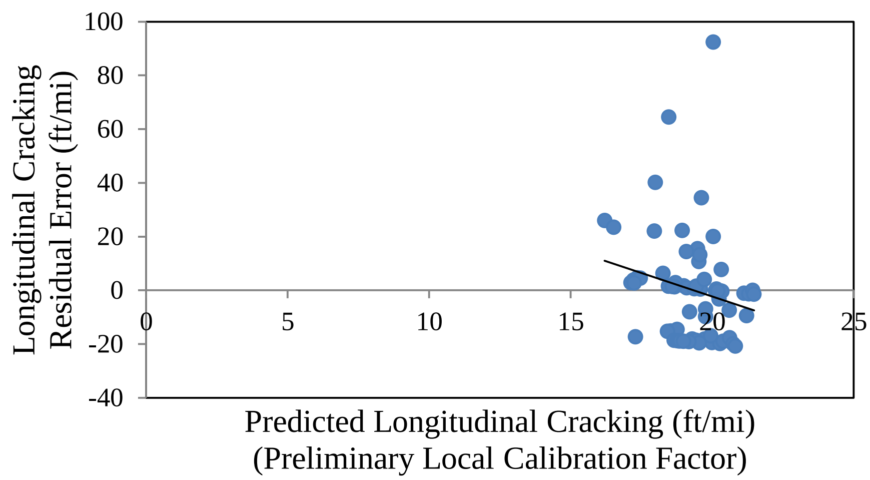


(b)

Figure 4.2 Predicted and Measured Alligator Cracking Using Preliminary Local Calibration Factors



(a)



(b)

Figure 4.3 Predicted and Measured Longitudinal Cracking Using Preliminary Local Calibration Factors

For longitudinal cracking, Figure 4.3 shows that bias has been eliminated but the coefficient of determination is still poor. Therefore, a high amount of variability between predicted and measured values was observed for longitudinal cracking, even after the calibration. There remains a question regarding the usability of longitudinal cracking models, as supported by precious research (Williams and Shaidur, 2013). There are two NCHRP research projects that are closely related to local calibration of MEPDG performance predictions: NCHRP 9-30 (NCHRP, 2003), and NCHRP 1-40B (NCHRP, 2007). Based on the findings from the NCHRP 9-30 study, it was recommended that the longitudinal cracking model be dropped from the local calibration guide developed in NCHRP 1-40B study due to lack of accuracy in the predictions (Muthadi, 2007). NCHRP 1-40B was completed in 2009 and currently, a NCHRP project 01-52 is underway to improve the longitudinal cracking model.

4.3.3 Calibration of Transverse Cracking Model

Calibration of the transverse cracking model was not performed for SC as no bias was found using the global calibration factors (see Figure 3.7 and Table 3.6). Therefore, the preliminary local calibration factors for SC are the same as the nationally used global calibration factor.

4.3.4 Calibration of Roughness Model

Pavement roughness is characterized by the International Roughness Index (IRI) which is the inverse of smoothness. Pavements IRI depends on initial IRI, rut depth, fatigue cracking, transverse cracking and the site factor:

$$IRI = IRI_0 + C_1 * RD + C_2 * FC + C_3 * TC + C_4 * SF \qquad 4.14$$

where, $IRI =$ International Roughness Index, in./mi,

$IRI_0 =$ Initial IRI after construction, in./mi,

RD = Average rut depth, in.,

FC = Area of total fatigue cracking, % of lane area,

TC = Length of transverse cracking, ft/mi,

SF =Site Factor= $Age(0.02003(PI + 1) + 0.007947(Rain + 1) + 0.000636(FI + 1))$

Age = Pavement age, years,

PI = Plasticity index of the soil, %,

FI = Average annual freezing index, °F,

Rain = Average annual rainfall, in.,

C_1, C_2, C_3, C_4 = Local calibration coefficients.

As IRI depends on several types of distresses (i.e., rutting, fatigue cracking, transverse cracking), the IRI model should be calibrated after the other models are calibrated. Figure 4.4 shows the IRI model using global calibration factors before calibration of the other distress models and Figure 4.5 shows the IRI model after preliminary local calibration of the other distress models but not calibrating IRI model. Comparing the figures, it is clear that the coefficient of determination has decreased, and the bias has not been removed; thus the model has not improved satisfactorily. Local calibration of the IRI model is proposed as part of the Phase II study.

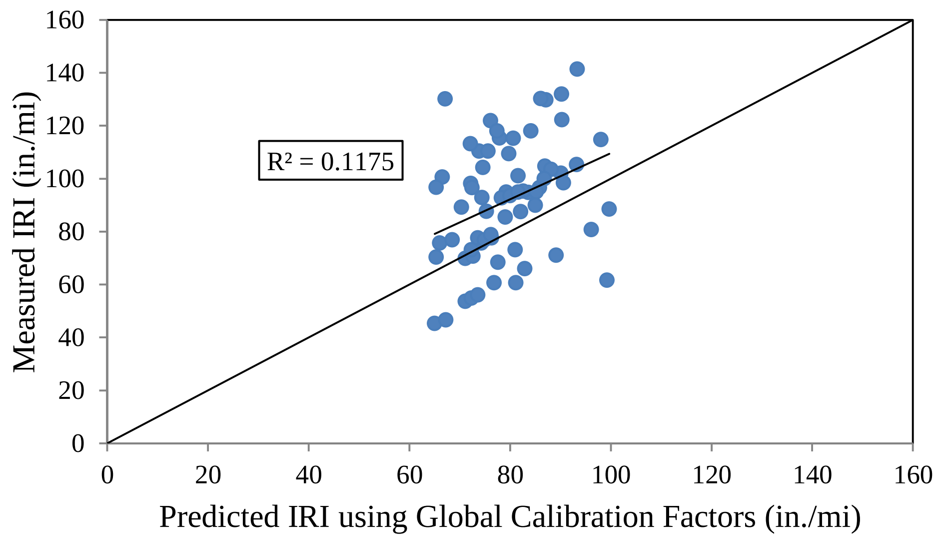


Figure 4.4 Predicted and Measured IRI using Global Calibration Factors

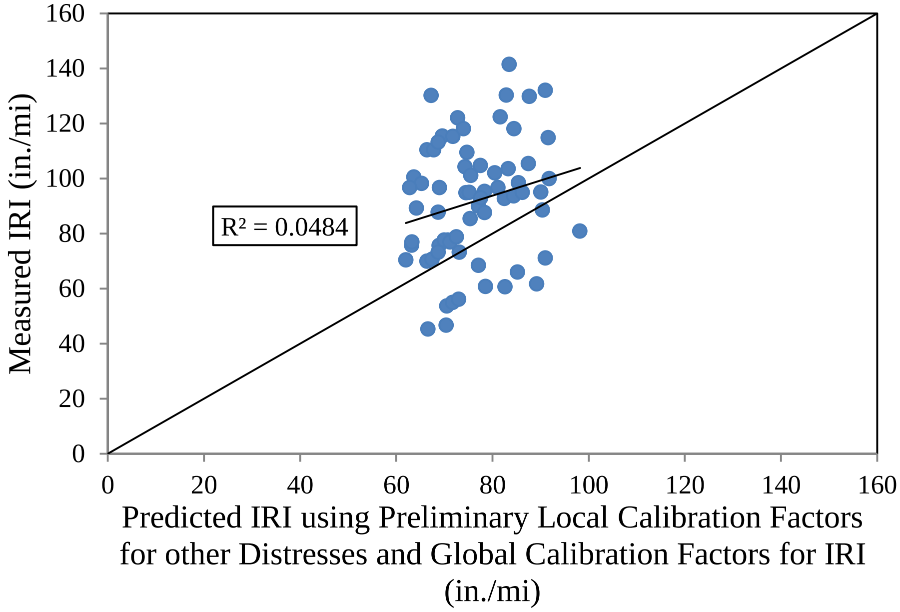


Figure 4.5 Predicted and Measured IRI Using Preliminary Local Calibration Factors for other Distresses

4.4 PRELIMINARY MEPDG LOCAL CALIBRATION FOR RIGID PAVEMENT

A preliminary study on the local calibration of the JPCP models was performed using Steps 8 to 11 in a similar manner as described for flexible pavement in Section 4.3. The optimization method for each of the calibration factors will be described in the subsequent subsections.

4.4.1 Calibration of JPCP Transverse Slab Cracking Model

The percentage of slabs with transverse cracking (i.e., including all severities) in a given traffic lane is used as the measure of transverse cracking and is predicted using the following global equation for both bottom-up and top-down cracking.

$$CRK = \frac{1}{1+(DI_F)^{-1.98}} \quad 4.15$$

where CRK = Predicted amount of bottom-up or top-down cracking (fraction)

DI_F = Total fatigue damage (top-down or bottom-up) = $\sum \frac{n_{i,j,k,l,m,n,o}}{N_{i,j,k,l,m,no}}$

where $n_{i,j,k,l,m,n}$ = Applied number of load applications at condition i, j, k, l, m, n,

$N_{i,j,k,l,m,n}$ = Allowable number of load applications at condition i, j, k, l, m, n,

i, j, k, l, m, n,o = Age, month, axle type, load level, equivalent temperature difference, traffic offset path, and hourly truck traffic fraction, respectively.

The allowable number of load applications is determined using the following equation.

$$\log(N_{i,j,k,l,m,n}) = C_1(\frac{MR_i}{\sigma_{i,j,k,l,m}})^{C_2} \qquad 4.16$$

where $N_{i,j,k,l,m,n}$ = Allowable number of load application at condition i, j, k, l, m, n,

MR_i = PCC modulus of rupture at age i, psi

$\sigma_{i,j,k,l,m}$ = Applied stress at condition i, j, k, l, m, n,

C_1= Calibration constant, 2.0, and

C_2= Calibration constant, 1.22.

Total combined cracking is determined using following equation.

$$TCRACK = \left(CRK_{Bottom-up} + CRK_{Top-down} - CRK_{Bottom-up} * CRK_{Top-down}\right) * 100\% \qquad 4.17$$

where $TCRACK$ = Total transverse cracking (%, all severities),

$CRK_{Bottom-up}$ = Predicted amount of bottom-up transverse cracking (fraction), and

$CRK_{Top-down}$ = Predicted amount of top-down transverse cracking (fraction).

In this study, the transverse slab cracking model was locally calibrated to determine preliminary values of the calibration factors for SC, comparing the measured and predicted transverse cracking. The 6 PCC pavement sections in Table 2.2 were used in the calibration process, which include 56 data points (see Appendix F). Data points include measurement of transverse cracking on the 6 selected pavements for the past several years. Calibration constant C_1 was optimized using different trial values in MEPDG. The minimum sum of squared errors between measured and predicted transverse cracking was found for C_1= 1.25. As no bias was found after using that value, no further optimization was needed for C_2, and it was kept as the global calibration factor (C_2=1.22). Figure 4.5 and Figure 4.6 show the measured and predicted transverse cracking before (i.e., using global calibration factors and local calibration values as unity) and after calibration (i.e., using preliminary local calibration factors), respectively. No to nil transverse cracking was predicted by MEPDG using global calibration factors or before the preliminary local calibration (Figure 4.6). However, the model has been improved by performing the preliminary local calibration as shown in Figure 4.7. Bias has been eliminated as measured and predicted values are almost evenly distributed across the line of equity.

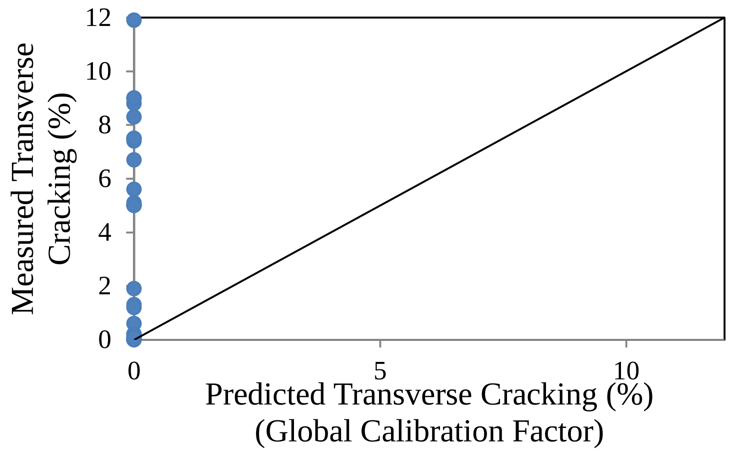


Figure 4.6 Predicted and Measured Transverse Slab Cracking Using the Global Calibration Factors

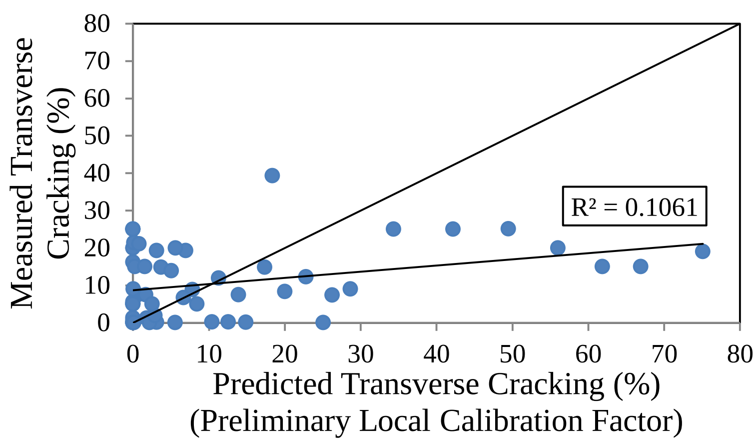


Figure 4.7 Predicted and Measured Transverse Slab Cracking Using Preliminary Local Calibration Factors

4.4.2 Calibration of JPCP Mean Transverse Joint Faulting Model

The mean transverse joint faulting model of JPCP pavement was not calibrated because no measured faulting information was available in the SCDOT historical files.

4.4.3 Calibration of JPCP Roughness Model

The IRI model of JPCP pavements depends on initial IRI, transverse slab cracking, spalling, faulting, and the site factor as stated below.

$$IRI = IRI_0 + C_1 * CRK + C_2 * SPALL + C_3 * TFAULT + 4 * SF \qquad 4.18$$

where IRI = Predicted IRI, in./mi,

IRI_0 = Initial IRI, in./mi,

CRK = Transverse cracking, % slabs,

$SPALL$ = Percentage of joints with spalling (medium and high severities),

$TFAULT$ = Total joint faulting cumulated per mi, in., and

SF = Site factor = $Age(1 + 0.5556 * FI)(1 + P_{200}) * 10^{-6}$

where Age = Pavement age, years,

FI = Freezing index, °F-days, and

P_{200} = Percent subgrade material passing No. 200 sieve,

C_1, C_2, C_3, C_4 =Calibration constants= 0.8203, 0.4417, 0.4929, 25.24 respectively.

As transverse cracking is one of the factors in the IRI model, the IRI model was calibrated after the transverse cracking model local calibration was performed. Figure 4.8 shows the results for the IRI model with global calibration factors and Figure 4.9 shows the results of the calibration after transverse cracking model was locally calibrated. As no bias is shown for the IRI model in Figure 4.9 after the transverse cracking local calibration, additional calibration is not necessary for the IRI model in the Phase I study. The bias will need to be re-examined with additional Level 1 data in a Phase II study.

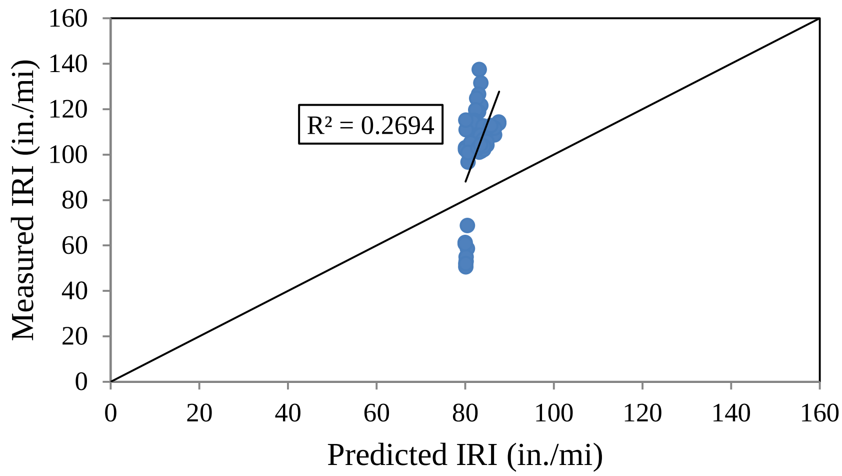


Figure 4.8 Predicted and Measured IRI Using the Global Calibration Factors

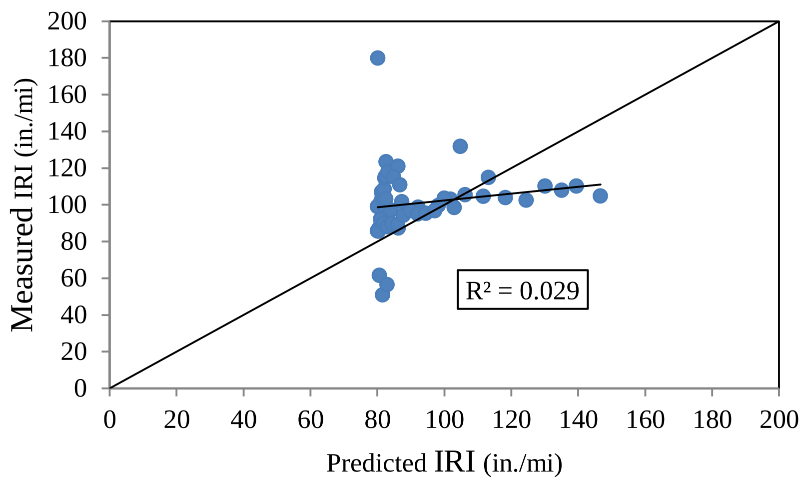


Figure 4.9 Predicted and Measured IRI after Transverse Cracking Model Calibration

Table 4.1 Summary of Statistical Parameter for Preliminary Local Calibration Values

Pavement Type	Performance Indicator	Bias, e_r	Standard Error, S_e	S_e/ S_y	R^2	Hypothesis; H_0: y_i-x_i = 0	Comments
AC	Rutting	0.01	0.08	1.14	0.12	Accept; p = 0.104	No Bias
	Fatigue Cracking	1.91	17.49	1.00	0.10	Accept; p = 0.377	No Bias
	Longitudinal Cracking	0.26	19.88	1.01	Poor	Accept; p = 0.919	No Bias
	Transverse Cracking	25.42	354.23	0.67	Poor	Accept; p = 0.56	No Bias
	IRI	15.5	21.87	0.99	Poor	Reject; p = 0	Bias
PCC	Transverse Cracking	-1.38	17.80	1.91	0.11	Accept; p = 0.568	No Bias
	IRI	8.73	22.03	1.24	Poor	Accept; p = 0.005	No Bias

Note: Poor means that the model did not explain variation in the measured data within and between the pavement segments.
Bias, $e_r = y_i - x_i$
y_i = Measured Value; Standard Deviation of the measured values.
x_i = Predicted Values

4.5 SUMMARY OF PRELIMINARY LOCAL CALIBRATION

Table 4.1 shows a summary of the statistical parameters after the preliminary local calibration. Bias (e_r) is shown for each performance indicator for the entire sampling template for AC and PCC pavements using the preliminary local calibration factors. The null hypothesis (H_0) for rutting was accepted using the preliminary local calibration factors; whereas, it was rejected when global calibration factors were used (refer to Table 3.6). Therefore, local bias for all AC and PCC performance indicators (except AC pavement IRI) was eliminated by performing the preliminary local calibration.

4.6 PRELIMINARY LOCAL CALIBRATION CONCLUSIONS

A summary of the preliminary calibration coefficients found in this study are shown in Table 4.2. The effect of the preliminary local calibration on the pavement distress models prediction in SC is also shown. Note that these preliminary calibration factors are not recommended to be used for design until further study with sufficient quantity and quality of data is obtained in future phases of the research.

In the preliminary study for rutting, use of the global calibration factors in Chapter 3 over-predicted the rutting for SC; whereas, use of the global calibration factors under-predicted the AC fatigue cracking (i.e., both alligator (bottom-up) and longitudinal (top-down)) and JPCP transverse cracking. Therefore, by using the preliminary local calibration factors, the net effect on prediction was decreased for the rutting model, and increased for the AC fatigue cracking (both alligator (bottom-up) and longitudinal (top-down)) and JPCP transverse cracking.

There was no net effect on the AC pavement transverse cracking because the global calibration coefficients were used as the local calibration factors. Even though a high degree of variability was observed between the measured and predicted longitudinal cracking, the preliminary local calibration model shows better predictions with lower bias and standard error than the global calibration model. It is important to stress that all of these results are preliminary and further work is required in a Phase II study.

Table 4.2 Summary and Effect of Preliminary Local Calibration Coefficients
(Not Recommended to be used for Design)

Pavement Type	MEPDG Models	Calibration Coefficients	Global Calibration Coefficients	Preliminary Local Calibration Coefficients	Net Effect on Prediction
AC Pavement	Rutting Model	β_{r_1}	1	0.240	Decreased
		β_{r_2}	1	1	
		β_{r_3}	1	1	
		β_{GB}	1	2.979	
		β_{SG}	1	0.393	
	Bottom-up Fatigue / Alligator Cracking Model	β_{f_1}	1	5	Increased
		β_{f_2}	1	0.8	
		β_{f_3}	1	0.8	
		C_1	1.00	0.47	
		C_2	1.00	0.47	
		C_4	6000	6000	
	Top-down Fatigue / Longitudinal Cracking Model	β_{f_1}	1	5	Increased
		β_{f_2}	1	0.8	
		β_{f_3}	1	0.8	
		C_1	7.00	0.2	
		C_2	3.5	0.1	
		C_4	1000	3.97	
	AC Transverse Cracking Model	$k_t\ (Level\ 1)$	5.0	5.0	No Change
		$k_t\ (Level\ 1)$	1.5	1.5	
		$k_t\ (Level\ 1)$	3.0	3.0	
		β_t	1	1	
	IRI	C_1	40		
		C_2	0.400		
		C_3	0.0080		
		C_4	0.0150		
PCC Pavement (JPCP)	Transverse Cracking Model	C_1	2.0	1.25	Increased
		C_2	1.22	1.22	
	Mean Transverse Joint Faulting	C_1	1.29		
		C_2	1.1		
		C_3	0.001725		
		C_4	0.0008		
		C_5	250		
		C_6	0.4		
		C_7	1.2		
	IRI	C_1	0.8203		
		C_2	0.4417		
		C_3	0.4929		
		C_4	25.24		

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 SUMMARY

Existing historical data within the South Carolina Department of Transportation (SCDOT) were identified and reviewed for use in the local calibration of the Mechanistic Empirical Pavement Design Guide (MEPDG) for South Carolina (SC). The major categories of the compiled data include climate, traffic, pavement structure and materials, and pavement performance. Based on all of the compiled information, 20 in-service pavement sections were identified: 14 asphalt concrete (AC) pavement sections and 6 Portland cement concrete (PCC) pavement sections. For 3 of these sections, field sample collection, Falling Weight Deflectometer (FWD) tests, and geotechnical laboratory tests were also performed to determine project specific material inputs (i.e., soil classification and resilient modulus of subgrade soils). The collected data was used to perform a preliminary MEPDG analysis study for the AC rutting models, AC fatigue cracking models, AC transverse cracking model, and the JPCP transverse cracking model. Additional analysis was performed to eliminate the bias for each pavement distress prediction model and obtain preliminary local calibration factors. Inputs for the analysis were from all 3 hierarchical categories: Level 1 (project specific), Level 2 (region specific), and Level 3 (national or default values). Level 2 and Level 3 inputs were used for many of the material property inputs due to their unavailability in the SCDOT files and databases for the selected 20 pavement sections. The local calibration coefficients obtained from this analysis are preliminary, and should not be used for design until further studies are performed to obtain high priority, high quality data.

5.2 FINDINGS AND RECOMMENDATIONS

5.2.1 Identification and Review of Data

Based on the identification and review of historical data within SCDOT, the key findings and corresponding recommendations are as follows:

- 20 pavement sections were selected from 15 counties in SC to serve as a representative sample—14 AC sections of average length 5.3 miles and 6 PCC sections of average length 5.8 miles. This number of pavement sections was deemed adequate for Phase I of the project; however, it is recommended to follow the AASHTO (2010) recommendations and use at least 30 pavement sections for each of AC and PCC pavements in Phase II.
- The primary source of traffic data was the traffic data collected through traffic counts by ATRs; however, ATRs do not provide axle load spectra. Vehicle class distributions were gathered; however, monthly and hourly traffic distributions were not obtained. It is recommended to obtain this missing traffic data, including Weigh-in-Motion (WIM) data, for the next phase of the project.
- WIM data were collected from the SCDPS State Transport Police for the 2 active stations: 1 is in Townville (Anderson County) on I-85 N, mile marker 9 and the other 1 is in St. George (Dorchester County) on I-95 N, mile marker 74. For these 2 stations, WIM data were collected for the last 5 years for different single, tandem, tridem and quadrem axles and used as Level 2 inputs for the MEPDG analysis. For Phase II, it is recommended to establish portable WIM stations to generate project specific (Level 1) WIM data.

- Dynamic modulus, binder grades, air voids, effective binder content, and mix gradations are required key inputs for AC layers for MEPDG. Asphalt mix design information for various job mixes was obtained from laboratory test reports for the time period from 2012 to 2014. Dynamic modulus data from one test was collected from a project currently funded by the FHWA/SCDOT. None of the collected data represents project specific (Level 1) information, thus extensive testing is required to gather those data.
- The required PCC properties for MEPDG are elastic modulus, Poisson's ratio, flexural strength, unit weight, coefficient of thermal expansion, thermal conductivity, heat capacity, cement type, cementitious material content, water-cement ratio, aggregate type, and ultimate shrinkage. However, none of the required properties for PCC pavements were available for this project. Therefore, studies need to be conducted to obtain these properties, including the coefficient of thermal expansion (CTE), for use in the MEPDG local calibration in Phase II.
- The material properties required for unbound layers for MEPDG are resilient modulus, gradation, Poisson's ratio, coefficient of lateral earth pressure, and other engineering properties. As part of this Phase I study, resilient modulus values of subgrade soils from 3 different soil regions of SC were determined though field and laboratory testing. In Phase II, it is recommended to collect samples of subgrade soils from all the selected sites. Then, perform laboratory tests on the samples to classify the soil and determine the resilient modulus. Moreover, the resilient modulus data of Phase I study was used to represent the annual representative values in MEPDG. In Phase II, monthly representative values are recommended to use in MEPDG by collecting samples for each

month of the year. The effect of seasonal variation of subgrade strength on subgrade rutting and pavement distresses needs to be investigated.

- Different types of materials are used for the base layer in SC (i.e., Graded Aggregate Base (GAB), Stabilized Aggregate Base (SAB), Cement Stabilized Macadam (CSM), Cement Modified Subbase (CMS), and Sand-Clay) which may have large differences in material properties. Due to the lack of information on the material properties for each of the SC base layer types, Level 3 (default) values were used. In future phases, it is recommended to gather Level 1 information (i.e., unit weight, resilient modulus, thermal conductivity, and heat capacity) for each type of base layer used in SC.
- While pavement distress data was available for this project, the confidence in the data is low; thus, detailed distress surveys are recommended. SCDOT measures alligator (bottom-up) and longitudinal (top-down) fatigue cracking by manual survey. However, it is difficult to distinguish between these two distresses without taking cores. Moreover, SCDOT measures rutting on pavement surfaces which is assumed to be the total rut. Trench studies are needed to determine AC rut, base rutting, subgrade rutting, and total rutting.
- In summary, as material properties for the surface layer (i.e., AC, PCC), base layer, and subgrade layer are basic input parameters to the AASHTOWare Pavement ME Design software, field testing, sampling, and laboratory testing of these materials is needed to obtain the missing data and achieve Level 1 inputs for each of the identified pavement sections. Furthermore, trench studies and distress surveys need to be performed to increase the confidence in the pavement performance data. WIM stations need to be installed to obtain site-specific load spectra.

5.2.2 Preliminary MEPDG Analysis

The following summary, conclusions and recommendations were made based on the results obtained from the preliminary MEPDG local calibration analysis for SC conditions:

- Preliminary studies were performed to locally calibrate the AC rutting, AC fatigue cracking (bottom-up and top-down), and JPCP transverse cracking models using the collected data having all 3 hierarchical categories (i.e., Level 1, Level 2, and Level 3). AASHTOWare 2.2.4 was used to analyze the 14 AC and 6 PCC pavement sections with numerous iterations and optimization of different trial values of calibration coefficients to preliminarily calibrate the different transfer functions in the distress prediction models.
- Initially, the calibration process was performed using the global calibration factors and taking all the local calibration coefficients as unity. The results showed extensive dispersion between the measured and predicted distresses for almost all distress prediction models. Therefore, bias was required to be reduced from all distress prediction models using local calibration values for both AC and PCC pavements.
- After performing the preliminary local calibration for all available distresses, the bias was reduced and preliminary local calibration factors for different distress prediction models were determined. Comparisons were made between the results, before and after local calibration of different models. It was found that the rutting model before local calibration was over predicting the rutting for SC; whereas, the AC fatigue cracking and JPCP transverse cracking models were under predicting the cracking before local calibration. No net effect on the AC pavement transverse cracking model was found.
- As rut depth data for each pavement layer is not a measurement collected by the SCDOT, local calibration could not be performed for the rutting models of the individual layers

(i.e., AC, base, and subgrade). However, assuming the collected rut data as the total rut depth, local calibration was performed for the total rut only. Therefore, the contribution from each layer to total rutting was not obtained from this study, and it is recommended to collect rut depth data for the individual pavement layers in Phase II.

- Data for both alligator cracking and longitudinal cracking were available in the SCDOT files and assumed as bottom-up fatigue and top-down fatigue cracking, respectively, for MEPDG analysis. This assumption may affect the calibration results for the AC fatigue cracking model because the SCDOT uses visual inspection methods to perform their distress surveys; and, the difference between alligator (bottom-up) and longitudinal (top-down) cracking is difficult to discern without trench studies. Therefore, pavement coring and trench studies are recommended for Phase II to measure top-down and bottom-up cracking to gain better confidence in the data.
- To improve the performance of the distress prediction models for SC, the number of pavement sections selected should be increased to 30 for both AC and PCC pavements, and more Level 1 and Level 2 data should be obtained through field and laboratory testing. This data is needed to improve the distress prediction models prior to constructing the special pavement sections for validation purposes.
- The SCDOT should adopt distress measurement techniques that are in agreement with the MEPDG distress measurement procedures. For instance, top-down and bottom-up fatigue cracking should be determined by trench studies and rutting should be measured for each individual pavement layer (i.e., AC layer, base layer, and subgrade layer), rather than only measuring the total rutting.

- The JPCP mean transverse joint faulting and IRI models were not calibrated due to the lack of PCC faulting and spalling data within SCDOT. The required distress information (i.e., JPCP fatigue cracking, JPCP faulting, spalling, and IRI) needs to be obtained to perform the local calibration of these distress models for Phase II.
- Initial IRI is an important input parameter to MEPDG. As initial IRI was not available for the 20 selected pavements, the value of initial IRI was back-predicted considering the IRI trend for all pavements of same type (i.e., AC, PCC). That value represents the average initial IRI for all pavements (i.e., 60 for AC, 80 for PCC). In Phase II, it is recommended to use site specific initial IRI by studying the trend of IRI of that section, rather the average IRI.
- NCHRP project 01-52 is underway to improve the longitudinal cracking model for the MEPDG. Therefore, it is recommended to wait and perform the local calibration for longitudinal cracking after the new model has been implemented in AASHTOWare.
- In summary, the SCDOT measures IRI, rutting, fatigue cracking, longitudinal cracking, and transverse cracking for AC pavements; however, the cracking and rutting data cannot be implemented into MEPDG with the highest confidence level. This is because bottom-up and top-down cracking are not clearly distinguished by their visual inspection procedure and only the total rut depth is measured. Therefore, pavement trench studies are recommended for Phase II to measure top-down and bottom-up cracking and also to measure rut depth for each individual pavement layer. For rigid pavements, pavement inspections to obtain pavement cracking data (i.e. faulting, punchouts) are required for MEPDG local calibration.

5.2.3 Use of Results

Use of the preliminary local calibration factors presented herein may result in over- or under-conservative, and potentially uneconomical, designs; thus, should not be used without performing additional studies to collect the missing high priority, site-specific materials data and obtain high quality pavement distress data. A plan to collect this information is presented in Chapter 6.

CHAPTER 6

PLAN FOR PHASE II

6.1 INTRODUCTION

Based on research performed during Phase I of the *Calibration of the AASHTO Pavement Design Guide to South Carolina Conditions*, the following tasks are proposed for Phase II:

Task 1 Identify additional pavement sections

Task 2 Collect distress survey data and perform trench studies

Task 3 Collect high priority materials data

Task 4 Install portable WIM stations

Task 5 Study seasonal variation of subgrade modulus

Task 6 Perform MEPDG analysis using the new data obtained in Tasks 1-5

Task 7 Plan for special pavement sections

The tasks will be finalized based on discussions with the Project Steering and Implementation Committee. Some details regarding each proposed task are summarized in the following subsections.

6.2 MEPDG ANALYSES WITH NEW DATA

The objective of Phase II will be to build upon the studies in Phase I to obtain local calibration factors and improve distress predictions by collecting new data of high priority.

6.2.1 Task 1: Identify Additional Pavement Sections

Additional pavement sections need to be identified for inclusion in the local calibration of the distress models. A total of at least 30 AC sections and at least 30 PCC (JPCP) sections are recommended by AASHTO. This means that an additional 16 AC pavement sections are needed and 24 more JPCP sections are needed. Historical data for each section will need to be collected.

In addition, 5 AC and 5 JPCP sections will need to be selected to validate the models once they are calibrated.

6.2.2 Task 2: Data for Pavement Performance

Because the confidence in the quality of the distress data collected for AC pavements in Phase I was low, there is a need to collect distress survey data and perform trench studies. High quality data for HMA rut, base rut, subgrade rut, total rut, and bottom-up and top-down fatigue cracking needs to be obtained. At a minimum, distress surveys and trench studies should be performed for the pavement sections along US-321 in Orangeburg County, US-521 in Georgetown County, and SC-93 in Pickens County. These sites represent different soil regions above and below the fall line, and tests were performed to obtain both FWD and resilient modulus data for these sites in 2015. The following data is needed:

Rut depth (in.) for each individual pavement layer

Bottom-up fatigue (alligator cracking) (%)

Top-down fatigue (longitudinal cracking) (ft/mi)

Transverse cracking (ft/mi)

IRI (in./mi)

This distress data can then be compared to the distress data obtained in the SCDOT files and an assessment of the overall quality of the data as a whole can be made. If needed, adjustments can be made to the data for each of the identified pavement sections.

For PCC (JPCP) pavements, the following data was missing and needs to be obtained:

Transverse cracking (% slabs cracked)

Transverse joint faulting (in.)

Spalling (% of joints)

IRI (in./mi)

6.2.3 Task 3: Collect High Priority Materials Data

The missing data that was identified in Phase I is summarized in Table 6.1. High priority has been assigned to those properties that have been identified in the literature through sensitivity analyses and other studies as having the greatest impact on pavement design using the MEPDG. For the high priority data, field and laboratory investigations need to be performed to obtain the material properties for the pavement sections along US-321 in Orangeburg County, US-521 in Georgetown County, and SC-93 in Pickens County, at a minimum, and used as Level 1 inputs. For the other pavement sections, high priority data such as CTE and Dynamic Modulus currently being studied in FHWA/SCDOT research projects needs to be obtained and can be used as Level 2 inputs for pavements with similar materials.

Table 6.1 Material Data Needs with Priority to Obtain Level 1 Inputs

Layer	Properties	Priority
Unbound Base & Subgrade	Resilient Modulus, Gradation, Liquid Limit, Plasticity Index, Dry Unit Weight	High
	Hydraulic Conductivity, Specific Gravity, Optimum Moisture Content, Soil Water Relation	Medium
	Poisson's Ratio, Coefficient of Lateral Earth Pressure	Low
HMA	Dynamic Modulus, Unit Weight, Binder Grade, Air Void, Effective Binder Content	High
	Creep Compliance, Indirect Tensile Strength, Fatigue Endurance Limit, Thermal Conductivity, Heat Capacity, Thermal Contraction	Medium
	Poisson's Ratio	Low
PCC	Coefficient of Thermal Expansion, Modulus of Rupture, Elastic Modulus, Compression Strength, Unit Weight	High
	Thermal Conductivity, Heat Capacity	Medium
	Cement type, Aggregate Type, Cementitious Material Content, Water Cement Ratio, Ultimate Shrinkage, Reversible Shrinkage	Low

Future phases should also consider the properties of the unbound base layer in SC i.e., Graded Aggregate Base (GAB), Stabilized Aggregate Base (SAB), Cement Stabilized Macadam (CSM), Cement Modified Subbase (CMS), and Sand-Clay) which may have large differences in material properties. High priority for Level 1 inputs is identified in Table 6.2.

Table 6.2 Material Data Needs for Unbound Base Layer with Priority to Obtain Level 1 Inputs

Layer	Properties	Priority
Unbound Base (Non-stabilized)	Resilient Modulus, Gradation, Liquid Limit, Plasticity Index, Dry Unit Weight	High
	Hydraulic Conductivity, Specific Gravity, Optimum Moisture Content, Soil Water Relation	Medium
	Poisson's Ratio, Coefficient of Lateral Earth Pressure	Low
Unbound Base (Stabilized)	Resilient Modulus, Modulus of Rupture, Unit Weight	High
	Thermal Conductivity, Heat Capacity	Medium
	Poisson's Ratio	Low

6.2.4 Task 4: Install Portable WIM Stations

It is recommended to install portable WIM stations along US-321 in Orangeburg County, US-521 in Georgetown County, and SC-93 in Pickens County. Also, 1 WIM station should be installed on an interstate highway (i.e., there are 6 candidate sites in Table 2.1) to have a total of 3 active WIM stations on interstates, and 1 WIM station should be installed on S 378. Data will need to be continuously collected and monitored.

6.2.5 Task 5: Study Seasonal Variation of Subgrade Modulus

As the average annual resilient modulus was used as MEPDG input for the Phase I study, and because the resilient modulus has been shown to have seasonal variation (Chu, 1972; Ceratti, 2004; Heydinger, 2003; Guan et. al, 1998; Orobio and Zaniewski, 2011; Nassiri and Bayat, 2013; Khouri and Zaman, 2004), there is a need to investigate the effect of moisture content and volume changes on the resilient modulus for SC soils. At a minimum, the seasonal variation of resilient modulus should be studied for the soils along US-321 in Orangeburg County, US-521 in Georgetown County, and SC-93 in Pickens County to represent Group A and Group B soils. For each of the 3 sites, Shelby tube samples will be collected each month for the period of 1 year to perform moisture content and resilient modulus tests. FWD tests will be performed at the subgrade sample locations. Shelby tube and bulk sampling can occur concurrently with the trench studies performed in Task 2.

6.2.6 Task 6: Local Calibration of Distress Models

MEPDG analysis will be performed using the additional high priority data obtained in Tasks 1-5 to eliminate the bias and increase the precision for each of the pavement distress models. Local calibration factors will be obtained. The analysis will begin with the models analyzed in Phase I (i.e., AC rutting, fatigue cracking (bottom-up and top-down, and transverse cracking); and JPCP transverse cracking). Then the local calibration for AC IRI, JPCP mean transverse joint faulting (when data becomes available), and JPCP IRI will be performed. Reasonable trigger values for all distresses will need to be defined for SC conditions. The initial IRI for each pavement type will need to be studied.

As part of this analysis, additional data needs (those assigned medium and low priority in Table 6.1) will be prioritized by performing a sensitivity analysis.

6.2.7 Task 7: Plan for Special Pavement Sections

Knowledge of the stress strain behavior of pavement materials is imperative for efficient modeling using MEPDG for new construction and for rehabilitation of old pavements. Significant changes in pavement layer properties may occur due to seasonal changes under variable traffic loading conditions. Therefore, systematic collection and analysis of pavement response data from properly instrumented pavement test sections is proposed for future phases. Phase II will be focused on selecting the location of the sites (i.e., 1 site for AC pavement and 1 site of JPCP pavement), identifying vendors for sensors and equipment, obtaining cost estimates and developing a monitoring plan. Construction and monitoring of the special pavement sections will be conducted in a separate phase.

Based on a review of the literature (Timm et al., 2004; Newcomb et al., 1990, Loulizi et al., 2001; Nassar, 2001, Shelley et al., 2006; Tarefder and Islam, 2015) (see Appendix G), the special pavement test section shown in Figure 6.1 (cross-section) and Figure 6.2 (plan view) for AC pavement is proposed. The proposed test section is heavily instrumented to collect high priority data and will be used to conduct a detailed examination of the predictive equations and sub-routines included in the AASHTOWare Pavement ME Design Guide, and to determine how well the outcome reflects the actual conditions and pavement responses for South Carolina's local conditions.

The section is proposed to have 12 horizontal asphalt strain gauges (HASGs) at the bottom of the AC and 2 HASGs on the top of the second lift of Hot Mix Asphalt (HMA). 6 asphalt strain gauges (VASGs) will be installed on top of the base. 3 earth pressure cells (EPCs) will be installed on the top of the base, the middle of the base, and 4 in. below the subgrade. 3 moisture probes and 5 temperature sensors will also be installed. In addition, a weather station and an axle sensing strip need to be installed.

A summary of each instrument and the corresponding MEPDG parameter obtained from the instrument is presented in Table 6.3. These data will be monitored continuously by a portable field unit with remote communication abilities. A summary of the field investigations and the laboratory tests needed to obtain the material properties and field performance data are summarized in Tables 6.4 and 6.5. Samples of subgrade soil will be collected in accordance with the sampling plan outlined in Section 2.4.2. It is imperative to collect sufficient data to maximize the number of Level 1 inputs for MEPDG.

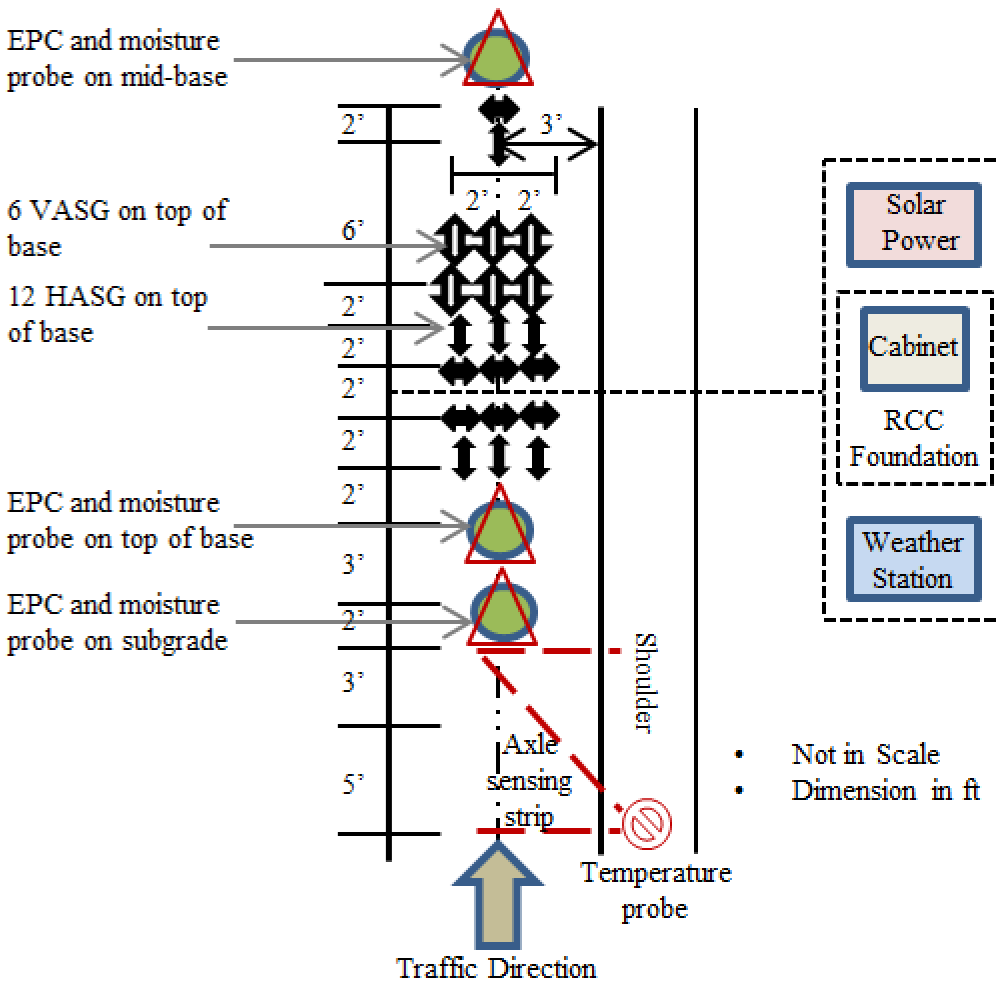


Figure 6.1 Instrumentation - Plan View

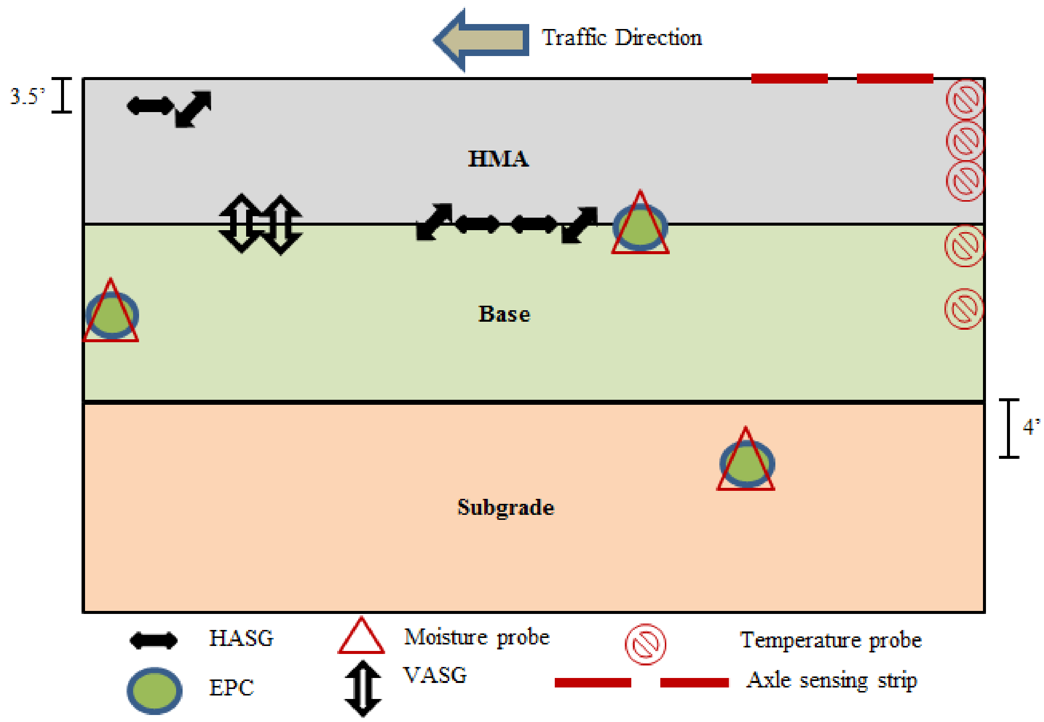


Figure 6.2 Instrumentation - Profile View

Table 6.3 Summary of Instrumentation

Instrument	MEPDG Parameters that will be Obtained	How often data will be taken
Horizontal Asphalt Strain Gage (HASG)	Tensile strain (fatigue)	Continuous
Vertical Asphalt Strain Gage (VASG)	Vertical deflection (rutting)	Continuous
Earth Pressure Cell (EPC)	Vertical pressure due to traffic load	Continuous
Moisture Probe	Moisture content	Continuous
Temperature Sensor	Temperature	Continuous
Weather Station	Air temperature, solar radiation, air speed, humidity, rainfall	Continuous
Axle Sensing Strip	Wheel distribution, Traffic speed	Continuous
WIM Station	Weigh-in-Motion Data	Continuous

Table 6.4 Summary of Field Tests

Field Test	MEPDG Parameters that will be Obtained	How often test will be performed
Falling Weight Deflectometer	Surface, base and subgrade modulus	4 to 12 times per year
Nuclear density	Density of AC layer	4 to 12 times per year
Field performance data	Rutting, IRI, transverse cracking, bottom-up and top-down fatigue cracking, faulting, spalling	1 time per year
Trench study	AC, base, and subgrade rutting, bottom-up and top-down fatigue cracking	1 time per year

Table 6.5 Summary of Laboratory Tests

Layer	Laboratory Test	MEPDG Parameters that will be Obtained
Base and subgrade	Sieve analysis	Soil classification and gradation
	Resilient modulus test	Resilient modulus
	Atterberg limit test	Liquid limit and plasticity index
	Standard Proctor test	Maximum dry unit weight and optimum moisture content
	Cyclic test	Poisson's ratio
	Specific gravity test	Specific gravity
HMA	Component and gradation test	Type and gradation of aggregates
	Dynamic modulus test	Dynamic modulus
	Determination of volumetric properties	Specific gravity, air void, asphalt content
	Binder gradation test	Binder gradation
	Four-point bending test	Fatigue endurance limit
	Indirect tensile strength test	Indirect tensile strength
	Cyclic test	Poisson's ratio
	Bending beam rheometer test	Stiffness of AC
PCC	Coefficient of thermal expansion test	Coefficient of thermal expansion
	Compression strength test	Compression strength
	Modulus of rupture test	Modulus of rupture
	Elastic modulus test	Elastic modulus
	Component and gradation test	Cement type, water-cement ratio, aggregate type
	Cyclic test	Poisson's ratio

REFERENCES

American Association of State Highway and Transportation Officials (AASHTO) (2008). *Mechanistic-Empirical Pavement Design Guide: A Manual of Practice*, Interim edition, Washington, D. C.

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

AASHTO M 145 (2003). “Standard Specification for Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes” Washington, D. C.

AASHTO T 307 (1999). “Determining the Resilient Modulus of Soils and Aggregate Materials”. *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, Washington, D. C.

AASHTO T 311 (2000). “Standard Method of Test for Grain-size Analysis of Granular Soil Materials”. *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, Washington, D. C.

AASHTO T 90 (2015). “Determining the Plastic Limit and Plasticity Index of Soils”. *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, Washington, D. C.

AASHTO T 99 (2015). “Standard Test Method of Test for Moisture-Density Relations of Soils”. *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, Washington, D. C.

AASHTO T 265 (2015). “Laboratory Determination of Moisture Content of Soils”. *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, Washington, D. C.

AASHTO T 208 (2015). “Standard Method of Test for Unconfined Compression Strength of Cohesive Soil”. *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, Washington, D. C.

AASHTOWare (2016). *Pavement ME Design v. 2.2.4*. Software Manual by AASHTO. < http://www.aashtoware.org/Pages/default.aspx>

ASTM D 6913 (2004). “Standard Test Methods for Particle-Size Distribution of Soils Using Sieve Analysis”. *American Society for Testing and Materials*, West Conshohocken, PA.

ASTM D 4318 (2010). “Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils”. *American Society for Testing and Materials*, West Conshohocken, PA.

ASTM D 2216 (2010). “Standard Test Methods for Laboratory Determination of Water Content of Soil and Rock by Mass”. *American Society for Testing and Materials*, West Conshohocken, PA.

ASTM D 698 (2012). “Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort”. *American Society for Testing and Materials*, West Conshohocken, PA.

ASTM D 6166 (2013). “Standard Test Methods for Unconfined Compression Strength of Cohesive Soil”. *American Society for Testing and Materials*, West Conshohocken, PA.

ASTM D 2488 (2009). “Standard Practice for Description and Identification of Soils”. *American Society for Testing and Materials*, West Conshohocken, PA.

Baus, R.L., Stires, and N.R. (2010). “Mechanistic Empirical Pavement Design Guide Implementation”, *Final Project Report*, Submitted to the South Carolina Department of Transportation and the Federal Highway Administration.

Ceratti, J.A., Gehling, W.Y., and Nunez, W.P. (2004). “Seasonal Variations of a Subgrade Soil Resilient Modulus in Southern Brazil”, *Transportation Research Record*, vol. 1874, pp. 165-173.

Chu, T.Y. (1972). “Investigation of Subgrade Moisture Conditions in connection with the Design of Flexible Pavement Structures”, *Final Project Report*, Submitted to the South Carolina State Highway Department and the Federal Highway Administration.

Federal Highway Administration (FHWA) (2003). *Distress Identification Manual for the Long-Term Pavement Performance Program*, FHWA-RD-03-0031. Federal Highway Administration (FHWA) (2004). *Pavement Smoothness Methodologies*, FHWA-HRT-04-061 145-91. <www.fhwa.dot.gov/pavement/smoothness/index.cfm>

Federal Highway Administration (FHWA) (2004). *Pavement Smoothness Methodologies*, FHWA-HRT-04-061 145-91. <www.fhwa.dot.gov/pavement/smoothness/index.cfm>

Guan, Y., Drumm, E.C., and Jackson, N.M. (1998). “Weighting Factor for Seasonal Subgrade Resilient Modulus”, *Transportation Research Record*, vol. 1821, pp. 47-55.

Heydinger, A.G. (2003). “Evaluation of Seasonal Effects on Subgrade Soils”, *Transportation Research Record*, vol. 1619, pp. 94-101.

Jadoun, F., and Kim, Y. (2012). “Calibrating Mechanistic-Empirical Pavement Design Guide for North Carolina, *Transportation Research Record*, vol. 2305, pp. 131-140.

Khoury, N.N., and Zaman, M.M. (2004). “Correlation between Resilient Modulus, Moisture Variation, and Soil Suction for Subgrade Soils”, *Transportation Research Record*, vol. 1874, pp. 99-107.

Loulizi, A., Al-Qadi, I., Lahour, S., and Freeman, T. (2001). “Data Collection and Management of the Instrumented Smart Road Flexible Pavement Sections”, *Journal of Transportation Research Record*, Vol. 1769, pp. 142-151.

Microsoft Office (2010). Software by Microsoft. < https://www.microsoft.com/en-us/>

Muthadi, N.R. (2007). “Local Calibration of the MEPDG for Flexible Pavement Design”, *M.S. Thesis*, North Carolina State University, Raleigh, NC.

Nassar, W. (2001). “Utilization of Instrument Response of SuperPave Mixes at the Virginia Smart Road to Calibrate Laboratory Developed Fatigue Equations”, *PhD Dissertation*, Virginia Polytechnic Institute and State University, Blacksburg, VA.

Nassiri, S., and Bayat, A. (2013). “Evaluation of MEPDG Seasonal Adjustment Factors for the Unbound Layers’ Moduli using Field Moisture and Temperature Data”, *International Journal of Pavement Research and Technology*, vol. 6, n. 1, pp. 45-51.

NCHRP (2003a). *Jackknife Testing- An Experimental Approach to Refine Model Calibration and Validation*, Research Results Digest 283. National Cooperative Highway Research Program 9-30. Washington, D.C. Transportation Research Board, National Research Council.

NCHRP (2003b). *Refining the Calibration and Validation of Hot Mix Asphalt Performance Models: An Experimental Plan and Database*, Research Results Digest 284. National Cooperative Highway Research Program 9-30. Washington, D.C. Transportation Research Board, National Research Council.

NCHRP (2007). *Recommended Practice for Local Calibration of the ME Pavement Design Guide*, National Cooperative Highway Research Program 1-40B Draft. Texas: ARA, Inc.

Newcomb, D.E., Johnson, D.M., Baker, H.B., and Lund, S.M. (1990). “Minnesota Road Research Project: Work Plan for Research Objectives”, *Report No. MN-RC-1990-03*, Minnesota Department of Transportation, MN.

Orobio, A., and Zaniewski, J.P. (2011). “Sampling-Based Sensitivity Analysis of the Mechanistic-Empirical Pavement Design Guide Applied to Material Inputs”, *Transportation Research Record*, vol. 2226, pp. 95-93.

Pierce, C. E., Gassman, S. L., and Ray, R. P. (2011). *Geotechnical Materials Database for Embankment Design and Construction*, Federal Highway Administration, Report FHWA-SC-11-02.

Schwartz, C. (2007). “Implementation of the NCHEP 1-37A Design Guide”, *Report No. SP0077B41*, Maryland State Highway Administration, Lutherville, MD.

Shahin, M. Y. (2005). “Pavement Management for Airports, Roads, and Parking Lots”, *Springer Publication*, New York, USA.

South Carolina Department of Transportation (SCDOT) (2008). *Pavement Design Guidelines*. Office of Materials and Research Pavement Design Unit. < http://www.scdot.org/doing/technicalpdfs/materialsresearch/pavementdesignguide2008.pdf>

South Carolina Department of Transportation (SCDOT) (2010). *Geotechnical Design Manual Version 1.1*. < http://www.scdot.org/doing/structural_geotechnical.aspx >

South Carolina Department of Transportation (SCDOT) (2013). *Guidelines for Asphalt Mixture Selections*.<http://www.scdot.org/doing/technicalPDFs/materialsResearch/Guidelines_Asphalt_Mix_Selection.pdf>

Souliman, M., Mamlouk, M., El-Basyouy, M., and Zapata, C. (2010). “Calibration of the AASHTO MEPDG for Flexible pavement for Arizona conditions”, *Compendium of Papers of the 89th TRB Annual Meeting (CD-ROM), Transportation Research Board*, Washington, DC.

Tanesi, J., Kutay, M., Abbas, A., and Meininger, R. (2007). “Effect of Coefficient of Thermal Expansion Test Variability on Concrete Pavement Performance as Predicted by Mechanistic-Empirical Pavement Design Guide”, *Transportation Research Record*, vol. 2020, pp. 40-44.

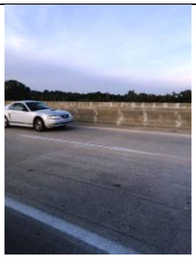
Tarefder, R.A., and Islam, M.R. (2015). “Study and Evaluation of Materials Response in Hot Mix Asphalt Based on Field Instrumentation- Phase I”, *Final Project Report, No. NM-11MSC-03*, Submitted to the New Mexico Department of Transportation and the Federal Highway Administration.

Timm, D.H., Priest, A.L., and McEwen, T.V. (2004). “Design and Instrumentation of the Structural Pavement Experiment at the NCAT Test Track”, *NCAT Report 04-01*, National Center for Asphalt Technology of Auburn University, AL.

Williams, R.C., and Shaidur, R. (2013). “Mechanistic Empirical Pavement Design Guide Calibration For Flexible Rehabilitation”, *Final Project Report*, Submitted to the Oregon Department of Transportation and the Federal Highway Administration.

Appendix A – Identify In-Service Pavement Sections

Table A-1 Site Visits

County	Location	Type	Length (miles)	Let Date	Date of Site Visit	Remarks	OK for Local Calibration?	Photographs
Aiken	I 520	PCC	5.35	7/25/2008	9/11/2014	No Overlay	Yes	
Beaufort	US 278	AC	1.56	3/13/1998	11/4/2014	No Overlay	Yes	
Charleston	SC 461	AC	2.48	5/21/1996	11/4/2014	No Overlay	Yes	
Charleston	I 526	PCC	2.39	6/25/1991	11/4/2014	No Overlay	Yes	

Chester	SC 9	AC	7.12	10/1/1999	11/12/2014	Looks Newer	Maybe Not	
Chesterfield	SC 151	AC	5.36	12/15/1999	11/12/2014	No Overlay	Yes	
Fairfield	I 77	PCC	14.17	10/21/1980	7/25/2014	Some Surfacing	Yes	
Florence	SC 327	AC	5.09	2/25/1992	10/22/2014	No Overlay	Yes	

Florence	US 301	AC	2.38	9/30/2003	10/22/2014	Some Overlay	Maybe	
Georgetown	US 521	AC	4.07	6/1/2003	10/22/2014	No Overlay	Yes	
Greenville	I 385	AC	7.65	8/28/2013	11/13/2014	Some PCC	Yes	
Greenville	I 85	AC	1.00	8/31/2005	11/13/2014	No Overlay	Yes	

Horry	SC 22	AC	24.35	10/12/2001	10/22/2014	No Overlay	Yes	
Horry	SC 31	AC	3.98	1/31/2005	10/22/2014	No Overlay	Yes	
Laurens	SC 72	AC	5.99	3/1/2002	11/13/2014	No Overlay	Yes	
Lexington	S 378	PCC	1.47	11/1/2001	7/25/2014	No Overlay	Yes	

Orangeburg	US 321	AC	6.17	7/1/2004	9/11/2014	No Overlay	Yes	
Pickens	SC 93	AC	1.34	4/10/2001	11/13/2014	No Overlay	Yes	
Spartanburg	SC 80	PCC	3.30	6/1/2000	11/13/2014	No Overlay	Yes	
Spartanburg	I 85	PCC	6.29	6/11/1997	11/13/2014	No Overlay	Yes	

Appendix B – Material Specifications

Table B-1 SCDOT Typical Asphalt Mix Design for Surface Type A

Job Mix	RAP (Y/N)	Source of Aggregate	AC Content	AV	VMA	Binder by Vol (design)	BSG Design	MSG Design	BSG Field (93%)	Binder by Vol (field)
N0139	Y	COLUMBIA	5.20	3.42	15.26	11.84	2.354	2.438	2.267	11.40
N0161	Y	GRAY COURT	5.10	3.49	15.10	11.61	2.354	2.439	2.268	11.19
N0183	N	COLUMBIA	5.20	3.52	15.36	11.84	2.354	2.440	2.269	11.41
N0204	N	LIBERTY/ GRAY COURT	5.10	3.50	15.22	11.72	2.376	2.462	2.290	11.29
N0324	Y	N. COLUMBIA/ COLUMBIA	5.10	3.53	14.50	10.97	2.224	2.305	2.144	10.58
N0325	N	N. COLUMBIA/ COLUMBIA	5.00	3.49	15.02	11.53	2.384	2.471	2.298	11.11
N0374	Y	COLUMBIA	4.80	3.45	14.50	11.05	2.380	2.465	2.293	10.64
N0415	Y	JEFFERSON	4.90	3.40	14.65	11.25	2.374	2.458	2.286	10.83
N0446	Y	SANDY FLATS	4.80	3.51	14.74	11.23	2.419	2.507	2.332	10.82
N0528	Y	SANDY FLATS/ CLINTON	4.90	3.43	14.83	11.40	2.406	2.491	2.317	10.98
N0542	Y	CAMAK	5.10	4.38	15.25	10.87	2.204	2.305	2.143	10.57
N0604	Y	JEFFERSON	5.00	3.51	14.91	11.40	2.358	2.443	2.272	10.99
N0615	N	JEFFERSON	5.10	3.41	15.01	11.60	2.352	2.435	2.264	11.17
P0136	Y	N. COLUMBIA/ COLUMBIA	4.90	3.60	14.90	11.30	2.385	2.474	2.300	10.90
P0231	N	AGUSTA/ CAMAK	4.90	3.54	14.86	11.32	2.389	2.476	2.303	10.91
P0304	Y	ROCK HILL	4.70	3.74	15.20	11.46	2.521	2.619	2.436	11.07
P0409	Y	BLACKSBURG	5.30	3.56	15.70	12.14	2.368	2.456	2.284	11.71
P0456	Y	LYNCHES RIVER	5.10	3.44	14.87	11.43	2.317	2.400	2.232	11.01
P0475	Y	JEFFERSON	4.70	3.86	14.64	10.78	2.372	2.467	2.294	10.43
P0511	N	GARDEN CITY	4.70	3.73	14.52	10.79	2.374	2.466	2.293	10.42
P0579	Y	LYNCHES RIVER	5.10	3.50	15.09	11.59	2.350	2.435	2.265	11.17
P0601	Y	JEFFERSON	4.80	3.56	14.53	10.97	2.363	2.450	2.279	10.58

Table B-1 SCDOT Typical Asphalt Mix Design for Surface Type A (cont.)

Job Mix	RAP (Y/N)	Source of Aggregate	AC Content	AV	VMA	Binder by Vol (design)	BSG Design	MSG Design	BSG Field (93%)	Binder by Vol (field)
A0092	Y	LYNCHES RIVER	4.70	3.47	14.50	11.03	2.427	2.514	2.338	10.63
A0097	Y	HANSON - ANDERSON	4.90	3.93	15.43	11.50	2.427	2.526	2.349	11.13
A0171	Y	LIBERTY	5.20	3.69	15.70	12.01	2.388	2.480	2.306	11.60
A0185	N	LIBERTY	5.30	3.79	16.02	12.23	2.386	2.480	2.306	11.82
A0242	Y	NORTH COLUMBIA	4.90	3.68	15.30	11.62	2.452	2.546	2.368	11.22
A0270	N	BLUEGRASS	5.20	3.82	15.52	11.70	2.327	2.419	2.250	11.31
A0296	Y	GRAY COURT	5.20	3.70	15.61	11.91	2.368	2.459	2.287	11.50
A0297	Y	COLUMBIA	4.70	3.72	14.51	10.79	2.374	2.466	2.293	10.42
A0357	Y	HANSON - ANDERSON	4.90	3.49	14.99	11.50	2.427	2.514	2.338	11.08
A0361	Y	SLOAN - BLACKSBURG	5.30	3.50	15.53	12.03	2.347	2.432	2.262	11.59
A0364	N	VULCAN - BLAIR	4.80	3.77	14.50	10.73	2.311	2.402	2.234	10.37
A0389	Y	ROGERS - HENRIETTA	4.80	3.79	14.93	11.14	2.400	2.494	2.320	10.77
A0400	Y	HANSON-JEFFERSON	5.00	3.51	14.97	11.46	2.370	2.456	2.284	11.05
A0407	N	VULCAN - GRAY COURT	5.20	3.54	15.48	11.94	2.374	2.461	2.289	11.51
A0431	Y	HANSON - JEFFERSON	5.00	3.75	15.18	11.43	2.364	2.456	2.284	11.04
		Average	4.99	3.61	15.05	11.44	2.371	2.460	2.287	11.03

(Source: Office of Materials and Research, SCDOT)

Note:

RAP: Reclaimed Asphalt Pavement
AC: Asphalt Concrete
AV: Air Void
VMA: Voids in Mineral Aggregate
BSG: Bulk Specific Gravity
MSG: Maximum Specific Gravity

Table B-2 SCDOT Typical Asphalt Mix Design for Intermediate B

Job Mix	RAP (Y/N)	Source of Agg.	AC Source	Binder	AV	VMA	Binder by Vol (design)	BSG Design	MSG Design	BSG Field (93%)	Binder by Vol (field)
N0022	Y	LYMAN	AAI	4.6	3.25	14.39	11.14	2.504	2.588	2.407	10.71
N0027	Y	PACOLET/ CAYCE	Nustar	4.2	3.88	13.64	9.76	2.403	2.500	2.325	9.44
N0030	N	NORTH COLUMBIA	Nustar	4.8	3.77	14.88	11.11	2.393	2.487	2.313	10.74
N0077	Y	LYNCHES RIVER	Nustar	4.9	3.62	14.95	11.33	2.391	2.481	2.307	10.93
N0086	Y	PINEVILLE/ ROCKING	Nustar	4.5	3.50	14.67	11.17	2.567	2.660	2.474	10.76
N0106	Y	PINEVILLE/ ROCKING	Nustar	4.5	3.66	14.82	11.16	2.564	2.662	2.475	10.77
N0113	Y	ROCKINGHAM	Nustar	5.1	3.57	15.42	11.85	2.403	2.491	2.317	11.43
N0127	Y	AUGUSTA/ APPLING	Nustar	4.9	3.74	14.99	11.25	2.374	2.466	2.294	10.87
N0156	Y	LOWRYS	AAI	4.5	3.20	13.59	10.39	2.387	2.466	2.294	9.98
N0163	Y	LIBERTY	AAI	4.8	3.99	15.19	11.20	2.413	2.513	2.337	10.85
N0207	Y	LYNCHES RIVER	Nustar	5.1	3.59	15.25	11.66	2.364	2.452	2.280	11.25
N0210	Y	COLUMBIA	Sloan-Union	5.0	3.51	14.94	11.43	2.364	2.450	2.278	11.02
N0270	Y	LIBERTY	AAI	4.5	3.22	13.77	10.55	2.424	2.505	2.329	10.14
N0339	Y	ARR./R. HILL	Nustar	4.7	3.43	15.11	11.68	2.570	2.661	2.475	11.25
N0434	Y	GREENVILLE	AAI	5.0	3.42	15.02	11.60	2.399	2.484	2.310	11.17
N0481	Y	NORTH COLUMBIA	Nustar	4.8	3.38	14.52	11.14	2.400	2.484	2.310	10.72
N0633	N	JEFFERSON	Nustar	4.6	3.48	14.02	10.54	2.369	2.455	2.283	10.16
N0662	Y	JEFFERSON	Nustar	4.3	3.62	13.51	9.89	2.378	2.468	2.295	9.54
P0031	Y	JEFFERSON	Nustar	4.6	3.68	14.27	10.59	2.380	2.471	2.298	10.22
P0092	N	STONEY POINT	AAI	5.0	3.71	15.16	11.45	2.368	2.459	2.287	11.06
P0108	N	AUGUSTA	AAI	5.2	3.93	15.88	11.95	2.376	2.473	2.300	11.57
P0109	Y	ROCK HILL	Nustar	5.2	3.67	16.30	12.63	2.511	2.607	2.425	12.19

Table B-2 SCDOT Typical Asphalt Mix Design for Intermediate B (cont.)

Job Mix	RAP (Y/N)	Source of Agg.	AC Source	Binder	AV	VMA	Binder by Vol (design)	BSG Design	MSG Design	BSG Field (93%)	Binder by Vol (field)
P0132	Y	LOWRYS	Nustar	5.3	3.78	15.84	12.06	2.353	2.445	2.274	11.66
P0135	Y	NORTH COLUMBIA	Nustar	4.9	3.35	14.68	11.33	2.391	2.474	2.301	10.90
P0191	Y	NORTH COLUMBIA	Nustar	4.5	3.26	13.73	10.47	2.406	2.487	2.313	10.07
P0207	Y	PINEVILLE/ ROCKING	AAI	4.9	3.73	15.75	12.02	2.536	2.635	2.450	11.61
P0221	Y	ROCKINGHAM	Nustar	4.8	3.84	15.12	11.28	2.430	2.527	2.350	10.91
P0223	Y	ROCK HILL	AAI	5.5	3.88	17.14	13.26	2.493	2.594	2.412	12.83
P0238	Y	NORTH COLUMBIA	Nustar	5.0	3.67	15.19	11.52	2.382	2.473	2.300	11.12
P0250	Y	NORTH COLUMBIA	Nustar	4.9	3.66	15.01	11.35	2.395	2.486	2.312	10.96
P0311	Y	AUGUSTA/ CAMAK	Nustar	5.1	3.51	15.35	11.84	2.401	2.488	2.314	11.41
P0351	Y	CAMAK	Nustar	4.8	3.29	14.50	11.21	2.415	2.497	2.322	10.78
P0364	Y	JEFFERSON	AAI	4.7	3.79	14.55	10.76	2.367	2.460	2.288	10.40
P0394	N	LIBERTY	AAI	5.0	3.93	15.47	11.54	2.386	2.484	2.310	11.17
P0454	Y	COLUMBIA	Southeast	5.2	3.67	15.49	11.82	2.350	2.440	2.269	11.41
P0459	Y	JEFFERSON	AAI	4.8	3.72	14.73	11.01	2.372	2.463	2.291	10.63
P0491	Y	BLACKSBURG	Southeast	5.3	3.46	15.62	12.16	2.372	2.457	2.285	11.71
P0494	Y	ANDERSON/ LIBERTY	AAI	5.2	3.56	15.85	12.29	2.444	2.534	2.357	11.85
P0550	Y	ARROWOOD	AAI	4.8	3.53	15.41	11.88	2.559	2.653	2.467	11.45
P0577	Y	CONWAY	AAI	5.0	3.44	15.21	11.77	2.434	2.521	2.344	11.34
P0599W	Y	NORTH COLUMBIA	Nustar	5.0	3.55	15.16	11.61	2.401	2.489	2.315	11.19
P0638	Y	JEFFERSON	AAI	4.8	3.62	14.65	11.03	2.376	2.465	2.293	10.64
P0672	Y	LIBERTY	AAI	5.0	3.58	15.17	11.59	2.397	2.486	2.312	11.18
P0666	Y	ROCKINGHAM	Nustar	5.2	3.79	15.94	12.15	2.416	2.511	2.335	11.74
A0018	Y	BLACKSBURG	Southeast	5.3	3.62	15.77	12.15	2.370	2.459	2.287	11.72

Table B-2 SCDOT Typical Asphalt Mix Design for Intermediate B (cont.)

Job Mix	**RAP (Y/N)**	**Source of Agg.**	**AC Source**	**Binder**	**AV**	**VMA**	**Binder by Vol (design)**	**BSG Design**	**MSG Design**	**BSG Field (93%)**	**Binder by Vol (field)**
A0033	Y	LYNCHES RIVER	AAI	4.9	3.48	14.86	11.38	2.401	2.488	2.314	10.96
A0037	Y	CAYCE	AAI	5.1	3.56	15.24	11.68	2.368	2.455	2.284	11.26
A0044	Y	KINGS MTN.	AAI	5.3	3.78	16.25	12.47	2.433	2.528	2.351	12.05
A0075	Y	BLACKSBURG	AAI	5.0	3.71	15.75	12.04	2.490	2.586	2.405	11.63
A0088	Y	COLUMBIA	AAI	5.2	3.54	15.44	11.90	2.366	2.453	2.281	11.47
A0093	Y	STONEY POINT	AAI	5.3	3.54	15.68	12.14	2.368	2.455	2.283	11.70
A0106	Y	BLACKSBURG	Southeast	5.2	3.95	15.87	11.92	2.370	2.468	2.295	11.54
A0120	Y	GRAY COURT	AAI	5.1	3.97	15.75	11.78	2.388	2.487	2.313	11.41
A0128	Y	JEFFERSON	AAI	4.9	3.66	14.93	11.27	2.378	2.469	2.296	10.88
A0144	Y	COLUMBIA	Southeast	4.7	3.82	14.57	10.75	2.365	2.459	2.287	10.39
A0165	N	HANSON LOWRYS	JT Russell	5.1	3.73	15.33	11.60	2.352	2.443	2.272	11.21
A0228	Y	MM/AUGUSTA	Southeast	5.2	3.56	15.42	11.86	2.358	2.445	2.274	11.44
A0230	Y	NOVA SCOTIA	Axeon SP	4.8	3.50	14.58	11.08	2.387	2.473	2.300	10.68
A0310	Y	NORTH COLUMBIA	AAI	4.9	3.61	15.03	11.42	2.410	2.500	2.325	11.02
A0329	Y	GRAY COURT	AAI	5.1	3.64	15.33	11.69	2.370	2.460	2.287	11.28
A0344	N	NORTH COLUMBIA	AAI	5.2	3.67	15.74	12.07	2.400	2.492	2.317	11.65
A0377	Y	ROGERS/ HENRIETTA	AAI	4.7	3.65	14.62	10.97	2.413	2.505	2.329	10.59
A0378	Y	JEFFERSON	AAI	4.8	3.48	14.52	11.04	2.378	2.464	2.291	10.64
A0441	N	JEFFERSON	AAI	5.3	3.71	15.74	12.03	2.347	2.437	2.267	11.62
A0490	Y	LYNCHES RIVER	AAI	4.7	3.73	14.56	10.83	2.383	2.475	2.302	10.46
A0492	Y	LIBERTY/ GRAY COURT	AAI	4.7	3.80	14.92	11.12	2.446	2.543	2.365	10.75
A0500	Y	MM/ROCK HILL	AAI	5.2	3.80	16.38	12.58	2.501	2.600	2.418	12.16
A0507	Y	VULCAN/ PINEVILLE	AAI	4.8	3.77	15.58	11.81	2.544	2.644	2.459	11.41

A0517	N	ROCKINGHAM	AAI	5.4	3.80	16.34	12.54	2.401	2.496	2.321	12.12
A0530	Y	MM/AUGUSTA	Axeon SP	5.2	3.65	15.54	11.89	2.364	2.454	2.282	11.48
A0543	N	VULCAN/BLAIR	AAI	5.1	3.78	15.46	11.68	2.368	2.461	2.289	11.29
			Average	4.94	3.63	15.14	11.51	2.410	2.501	2.326	11.11

(Source: Office of Materials and Research, SCDOT)

Note:

RAP: Reclaimed Asphalt Pavement
AC: Asphalt Concrete
AV: Air Void
VMA: Voids in Mineral Aggregate
BSG: Bulk Specific Gravity
MSG: Maximum Specific Gravity

Appendix C – Soil Test Results

Table C-1 Test Results of SC 93 (Pickens)

Station No.	Depth (m)	Color	AASHTO Classification	(+)60 Sieve	Silt	Clay	LL	PI	Rec. SSV	CBR
1+290	0-0.76	Rd-Brn	A-6 (3)				37	11	2.2	5.4
	0.76-1.52	Rd-Tan								
1+343	0.18-0.61	Brown								
	0.61-1.52	Tn & Brn								
1+480	0-1.22	Dk Tan	A-4(0)					NP		
1+742	0-3.66	Dk Brn	A-2-4(0)					NP	3.5	12.3
1+964	0-1.52	Rd-Brn	A-2-4(0)				35	NP	2.3	6.2
2+516	0-1.52	Rd-Brn	A-4(4)					NP	2.8	8.6
	1.52-3.05	Rd-Ylw	A-4(0)					NP	2.3	6.4
2+643	0-1.52	Lt Tan	A-5(5)				42	7	3.3	11.6
2+808	0.09-3.05	Tn & Brn	A-4(1)				32	10	2.8	8.9
2+970	0.09-1.52	Rd-Brn	A-7-5(10)				57	17	3.2	11
3+056	0.61-3.05	Rd & Tan	A-4(0)					NP	2.7	7.7
3+096	0.61-3.05	Rd-Brn	A-4(0)				38	NP	2.3	6.5
3+230	0.31-3.05	Rd-Brn	A-4(0)				36	NP	3	9.9
3+155	0-1.52	Rd-Brn	A-7-5(4)				44	12	3.6	13
3+219	0.61-1.52	Yl-Org	A-5(0)				41	NP	2.6	8
	3.05-6.10	Tn & Gry	A-6(3)				40	13	2.4	6.6
3+288	1.52-3.05	Lt Tan	A-2-4(0)				29	7	3.5	12.3

(Source: Office of Materials and Research, SCDOT)

*Note: LL = Liquid Limit, PI = Plasticity Index, Rec. SSV = Recommended Soil Support Value, CBR = California Bearing Ratio.

Table C-2 Test Results of US 521 (Georgetown)

Lab No	Station No.	Depth (ft)	Color	AASHTO Classification	(+)60 Sieve	Silt	Clay	LL	PI	Rec. SSV	CBR
5B-07009	2640+00	6"-1.5	Gray	A-7-6(6)	12	11	40	42	19	2.2	5.5
10		1.5-4.0	Rd Tan	A-7-5(13)	9	10	45	57	27	1.4	
11	2645+00	6"-1.5	Gray	A-4(1)	15	7	35	24	9	3.4	
12		1.5-4.0	Gray	A-7-6(9)	14	5	46	48	25	1.9	
13	2650+00	1.0-4.0	Br Gr	A-6(9)	6	9	50	37	20	1.3	
14	2655+00	8"-4.0	Br Gr	A-7-6(13)	5	11	47	48	25	1.9	
15	2660+00	1.0-3.0	Gray	A-2-6(1)	29	5	28	32	14	3.5	9.5
16		3.0-4.0	Tan	A-7-6(6)	32	4	37	51	26	2.2	
17	2665+00	8"-3.0	Gr Tan	A-6(2)	27	9	37	51	26	2.2	
18		3.0-4.0	Gray	A-6(2)	30	7	30	35	18	2.5	
19	2670+00	1.0-4.0	Gr Tan	A-7-6(10)	19	9	38	41	22	2.2	
20	2675+00	8"-4.0	Gr Tan	A-7-6(7)	22	7	41	43	22	2.2	
21	2680+00	6"-2.0	Gray	A-2-6(0)	45	7	26	25	11	3.5	
22		2.0-4.0	Gr Tan	A-7-6(10)	38	4	41	57	12	1.9	
23	2685+00	0-1.5	Lt Tan	A-2-4	20	13	15		NP	3.2	
24		1.5-4.0	Tan	A-7-6(8)	14	9	40	45	26	1.9	
25	2690+00	0-1.5	Lt Tan	A-2-4	14	12	16		NP	3.2	
26		1.5-4.0	Gr Tan	A-6(4)	9	10	37	34	16	2.5	
27	2695+00	6"-2.5	Gray	A-4(2)	17	14	40	24	10	3.4	
28		2.5-4.0	Gr Tan	A-6(2)	25	13	31	29	13	2.5	
29	2700+00	6"-1.0	Gray	A-4(6)	19	15	26		NP	3.4	
30		1.0-4.0	Tan	A-7-6(11)	20	9	45	47	26	1.4	
31	2705+00	0-1.5	Gray	A-2-4	19	15	16		NP	3.2	
32		1.5-4.0	Tan	A-4(1)	16	11	31	24	9	3.2	
33	2710+00	8"-4.0	Gray	A-4(0)	19	10	29	22	7	3.2	
34	2715+00	6"-3.0	Gr Tan	A-7-6(16)	31	5	50	59	34	1.4	
35		3.0-4.0	Gray	A-2-7(3)	60	2	30	57	28	3.5	

Table C-2 Test Results of US 521 (Georgetown) (cont.)

Lab No	Station No.	Depth (ft)	Color	AASHTO Classification	(+)60 Sieve	Silt	Clay	LL	PI	Rec. SSV	CBR
36	2720+00	6"-1.0	Lt Tan	A-2-4	35	10	14		NP	3.7	
37		1.0-3.0	Tan	A-7-6(7)	38	4	39	53	26	1.9	
38		3.0-4.0	Rd Tan	A-7-5(6)	56	3	33	67	36	1.9	
39	2725+00	8"-4.0	Tan	A-6(5)	13	14	40	33	15	2.5	
40	2730+00	6"-2.5	Tan	A-2-4	44	11	10		NP	3.7	
41		2.5-4.0	Tan	A-2-4	40	10	17		NP	3.7	
42	2735+00	1.0-4.0	Tan	A-2-4	45	5	11		NP	3.7	
L33981	2750+00	0-3.0	Gray	A-7-6(7)	51	3	36	52	36	2.2	
982		3.0-4.0	Dk Gr	A-2-6(0)	75	1	16	33	17	3	
983	2755+00	0-2.0	Gray	A-2-4	36	9	19		NP	3.7	
984		2.0-4.0	Gray	A-7-6(10)	42	3	41	57	35	1.9	
985	2760+00	0-1.0	Tan	A-2-4	12	15	18		NP	3.2	
986		1.0-4.0	Rd Tan	A-7-6(31)	4	9	69	65	37	1.4	
987	2765+00	6"-4.0	Gray	A-7-6(16)	6	17	58	43	22	1.4	
988	2770+00	6"-2.0	Tan	A-7-5(55)	4	6	82	88	53	1.4	
989		2.0-4.0	Lt Br	A-7-6(14)	24	6	46	56	35	1.9	
990	2775+00	0-2.5	Lt Tan	A-2-4	30	16	17		NP	3.7	
991		2.5-4.0	Rd Tan	A-7-6(9)	22	11	38	44	26	1.9	
992	2780+00	1.0-4.0	Tan	A-7-6(17)	8	10	49	53	33	1.4	
993	2785+00	1.0-4.0	Brown	A-7-6(15)	18	7	51	54	30	1.4	
994	2790+00	1.0-4.0	Tan	A-6(9)	12	20	46	38	17	1.9	
995	2795+00	0-3.0	Tan	A-6(2)	22	9	38	29	11	2.5	
996		3.0-4.0	Gr Tan	A-4(0)	26	8	31	23	10	3.5	
997											
998	2800+00	1.0-4.0	LT Br	A-7-5(27)	19	9	61	74	35	1.4	
999	2805+00	4"-4.0	Tan	A-7-6(12)	16	11	52	41	24	1.9	
L34000	2810+00	4"-2.0	Br Tan	A-7-6(11)	30	6	45	50	30	1.9	

Table C-2 Test Results of US 521 (Georgetown) (cont.)

Lab No	Station No.	Depth (ft)	Color	AASHTO Classification	(+)60 Sieve	Silt	Clay	LL	PI	Rec. SSV	CBR
1		2.0-4.0	Rd Tan	A-2-7(5)	53	1	33	65	38	2.2	
2	2815+00	0-1.5	Gray	A-2-4	49	9	22	18	2	3.7	
3		1.5-3.0	Tan	A-7-6(11)	39	4	42	62	34	1.9	
4	2820+00	4"-3.0	Tan	A-7-6(28)	7	13	68	55	33	1.4	
5		3.0-4.0	Tan	A-2-6(0)	26	4	27	31	13	3.5	
6	2825+50	6"-3.0	Tan	A-7-6(21)	15	6	57	57	38	1.4	
7		3.0-4.0	Tan	A-6(3)	40	3	33	40	22	2.5	
8	2830+00	1.0-4.0	Brown	A-7-5(8)	53	3	37	69	33	2.2	
9	2835+00	0-4.0	Tn/Dk	A-7-6(5)	54	6	34	50	24	2.2	
10	2840+00	8"-4.0	Brown	A-7-6(14)	48	3	44	70	41	1.9	
11	2845+00	8"-4.0	Brown	A-2-7(0)	62	2	24	58	28	2.2	
12	2850+00	8"-3.0	Gray	A-2-6(0)	60	4	25	33	17	3.5	
13	2850+00	3.0-4.0	Gr Tan	A-2-9	69	3	16	25	8	3.7	
14	2855+00	8"-3.5	Tan	A-7-6(5)	48	4	34	50	29	2.2	
15		3.5-4.0	Brown	A-2-7(2)	64	2	22	44	25	3.5	
16	2860+00	6"-4.0	Tan	A-7-6(16)	18	11	55	50	26	1.9	
17	2865+00	6"-2.5	Brown	A-7-6(25)	5	12	64	60	31	1.4	
18		2.5-4.0	Brown	A-7-6(7)	49	5	38	44	28	2.2	
19	2870+00	0-1.0	Gr Tan	A-2-4	59	6	12		NP	3.7	
20		1.0-4.0	Lt Br	A-7-6(10)	43	4	40	60	35	1.9	
21	2875+00	8"-4.0	Brown	A-7-6(7)	39	6	40	46	22	2.2	
22	2880+00	6"-4.0	Tan	A-7-6(16)	14	13	59	47	23	1.9	
23	2885+00	1.0-4.0	Tan	A-7-6(18)	47	5	39	49	30	1.9	
24	2890+00	1.0-4.0	Brown	A-7-6(12)	28	6	45	57	30	1.4	
25	2895+00	1.0-4.0	Dk Tan	A-7-5(33)	12	8	69	73	39	1.4	
26	2900+00	1.0-4.0	Tan	A-6(6)	14	6	44	36	18	2.5	
27	2905+00	3"-4.0	Gr Tan	A-7-6(53)	4	8	83	75	52	1.4	

Table C-2 Test Results of US 521 (Georgetown) (cont.)

Lab No	Station No.	Depth (ft)	Color	AASHTO Classification	(+)60 Sieve	Silt	Clay	LL	PI	Rec. SSV	CBR
28	2910+06	8"-4.0	Tan	A-7-6(14)	13	11	42	55	33	1.9	
29	2915+00	8"-4.0	Tan	A-2-7(1)	51	3	31	46	16	2.2	
30	2919+50	8"-3.0	Tan	A-7-5(3)	46	3	33	51	21	2.2	
31		3.0-4.0	Rd Tan	A-2-6(0)	46	2	26	38	16	3.2	
32	2925+00	3"-4.0	Tn Gr	A-7-6(20)	7	16	65	45	24	1.4	
33	2930+00	8"-3.0	Tan	A-7-6(35)	16	8	64	72	50	1.4	
34		3.0-4.0	Tan	A-2-4	70	1	13		NP	3.7	
35	2935+00	8"-4.0	Tan	A-6(6)	14	8	41	39	18	2.5	
36	2940+00	8"-4.0	Tan	A-6(8)	13	9	46	39	19	2.5	
37	2945+00	1.0-3.0	Tan	A-6(6)	19	8	38	37	19	2.5	
38		3.0-4.0	Tan	A-2-4	32	4	17		NP	3.7	
39	2950+00	6"-1.0	Lt Tan Gr	A-2-4	12	8	17		NP	2.9	
40		1.0-4.0	Tan	A-7-6(9)	9	7	48	45	21	2.2	
41	2955+00	3"-4.0	Rd Tan	A-7-6(14)	12	7	50	54	28	1.9	
42	2960+00	4"-4.0	Dk Tan	A-7-6(34)	2	12	72	65	36	1.4	
43	2965+00	8"-4.0	Gr Tan	A-7-6(14)	1	17	58	41	20	1.9	
44	2970+00	0-4.0	Gr	A-7-6(39)	1	10	81	66	38	1.4	
45	2975+00	0-4.0	Tan Gr	A-7-6(42)	2	20	70	65	42	1.4	
46	2880+00	0-1.0	Br Tan	A-4(0)	5	11	29	26	9	2.9	
47		1.0-4.0	Tan	A-7-6(20)	1	20	58	47	25	1.9	
48	2985+00	6"-3.0	Tan	A-7-6(21)	8	12	59	55	30	1.9	
49		3.0-4.0	Gr Tan	A-2-6(0)	27	4	29	35	13	3.5	
50	2991+00	6"-4.0	Rd Gr	A-7-5(34)	2	15	75	65	31	1.4	
51		0-6"	Tan	A-4(2)	4	27	50	26	4	2.7	
52	2995+00	6"-4.0	Tan	A-7-5(41)	1	18	76	69	37	1.4	
53	3000+00	0-4.0	Gr Tan	A-7-5(41)	1	17	75	75	35	1.4	
54	3005+00	6"-4.0	Tan	A-7-5(29)	3	11	79	60	27	1.4	
55	3010+00	6"-4.0	Rd GR	A-7-6(32)	5	16	72	61	32	1.4	

Table C-2 Test Results of US 521 (Georgetown) (cont.)

Lab No	**Station No.**	**Depth (ft)**	**Color**	**AASHTO Classification**	**(+)60 Sieve**	**Silt**	**Clay**	**LL**	**PI**	**Rec. SSV**	**CBR**
70618	3015+00	4"-4.0	Gr Tan	A-7-6(23)	4	15	70	52	25	1.9	
619	3020+00	0-4.0	Gr Red	A-7-6(23)	4	6	77	50	26	1.9	
620	3025+00	0-4.0	Gr Tan	A-7-5(29)	5	13	77	58	27	1.4	

(Source: Office of Materials and Research, SCDOT)

*Note: LL = Liquid Limit, PI = Plasticity Index, Rec. SSV = Recommended Soil Support Value, CBR = California Bearing Ratio.

Table C-3 Test Results of S 378 (Lexington)

Location	Depth (ft)	AASHTO	SPT(N)	Natural Moisture (%)	% Finer Than #200	LL	PL	PI	CBR	Standard Proctor Moisture (%) /Density (pcf)
B-95+00	3.5-5.0			4.7	7.8					
B-125+00	8.5-1.0	A-1-B		18	7.7	22	18	4		
B-893+00	1.0-2.5			8						
B-893+00	6.0-7.5	A-2-7		14.3	14.2	54	25	29		
B-898+00	1.0-2.5			13.4						
B-898+00	6.0-7.5	A-7-6		21.8	56.2	53	27	26		
B-903+00	13.5-15.0	A-7-5		23.7	90.4	58	32	26		
B-903+00	18.5-20.0			25.4	98.6					
B-908+00	6.0-7.5			15.3	41.8					
B-912+60	8.5-10.0			12.4	29.9					
B-335+00R1	1.0-2.5			5	6.5					
B-335+00R1	3.5-5.0			7.8	15.4					
B-335+00R1	6.0-7.5			10.9	8.7					
B-335+00R1	8.5-10.0			20.7		44	24	20		
B-335+00R1	13.5-15.0			1.9	1.8					
B-335+00R1	18.5-20.0			4.3	5.6					
B-335+00R1	23.5-25.0			3.9	3.4					
B-335+00R1	28.5-30.0			10	11.1					
B-335+00R1	33.5-35.0			15.8	19.8					
B-335+00R1	38.5-40.0			13.9		29	20	9		

Table C-3 Test Results of S 378 (Lexington) (cont.)

Location	Depth (ft)	AASHTO	SPT(N)	Natural Moisture (%)	% Finer Than #200	LL	PL	PI	CBR	Standard Proctor Moisture (%) /Density (pcf)
B-315+00R4	3.5-5.0			28.9		64	34	30		
B-315+00R4	6.0-7.5			20.7		43	23	20		
B-315+00R4	8.5-10.0			10.6	25.4					
B-315+00R4	13.5-15.0			28.7		61	31	30		
B-320+00R4	1.0-2.5			23.7		59	31	28		
B-320+00R4	3.5-5.0					70	32	38		
B-320+00R4	6.0-7.5					81	38	43		
B-320+00R4	8.5-10.0			12.5	38.1					
B-320+00R4	13.5-15.0			12.5	26.1					
B-320+00R4	18.5-20.0			10.3	20.6					
Bulk 1 B-883+00	0-5.0	A-2-4		13	21	24	17	7	8	10.5/122.4
Bulk 2 B-100+00	0-5.0	A-2-4		6.1	10.3	16	15	1	14	10.0/117.9
Bulk 3 B-115+00	0-5.0	A-2-4		9.7	16.3	19	17	2	11	11.8/18.6
Bulk 4 B-20+00	0-5.0	A-1-B		6.2	9.9	17	17	0	11	10.8/115.3
Bulk 5 B-310+00W	0-5.0	A-2-4		6.1	19.6	23	16	7	13	10.1/121.4
Bulk 6 B-342+50E	0-5.0	A-2-4		8.5	13.4	20	19	1	12	11.2/116.7
Bulk 7 B-365+00W	0-5.0	A-2-4		9	18.5	19	15	4	13	104/121.5
Bulk 8 B-330+00R1	5.0-10.0	A-2-4		9.4	25.6	22	16	6	8	10.3/125.0

(Source: Office of Materials and Research, SCDOT)

*Note: SPT = Standard Penetration Test, LL = Liquid Limit, PL = Plastic Limit, PI = Plasticity Index, CBR = California Bearing Ratio.

Appendix D – Pavement Performance Data

Table D-1 Pavement Performance Data of Selected Pavement Sections

County	Section	Direction	Type	Let date	Survey Year	Age (years)	BMP	EMP	Length (miles)	PSI	PDI	PQI	IRI (in./mile)	Rut (in.)
Aiken	I 520	E	PCC	2008	2014	6	18.05	22.968	4.918	3.81	4.93	4.39	68.74	0.05
					2012	4	18.1	23.61	5.51	4.05	4.94	4.45	52.89	0.04
					2011	3	18.02	23.42	5.4	4.09	5	4.5	50.48	0.09
					2010	2	18.02	23.42	5.4	3.92	5	4.47	61.27	0.08
Aiken	I 520	W	PCC	2008	2014	6	22.148	22.948	0.8	3.96	4.94	4.43	58.58	0.06
					2012	4	18	23.61	5.61	4.02	4.94	4.45	54.93	0.04
					2011	3	18.22	23.61	5.39	4.06	5	4.5	51.91	0.07
					2010	2	18.12	23.42	5.3	3.92	5	4.47	60.7	0.06
Beaufort	US 278	E	AC	1996	2013	17	0	20.74	20.74	3.42	3.54	3.3	98.21	0.14
					2012	16	0	20.74	20.74	3.23	3.71	3.4	112.85	0.15
					2010	14	0	20.74	20.74	3.41	3.47	3.26	99.72	0.17
					2008	12	0	20.74	20.74	3.44	3.76	3.49	96.87	0.17
					2005	9	0	20.74	20.74	3.39	3.49	3.29	100	0.28
Beaufort	US 278	W	AC	1996	2013	17	0	19.76	19.76	3.45	3.49	3.26	95.39	0.15
					2012	16	0	19.76	19.76	3.43	2.97	2.9	97.37	0.24
					2010	14	0	19.76	19.76	3.46	3.3	3.15	94.34	0.2
					2008	12	0	19.76	19.76	3.48	3.26	3.12	92.82	0.2
					2005	9	19.43	19.76	0.19	2.73	3.89	3.42	155.91	0.13
Charleston	SC 461	N	AC	1996	2013	17	0	3.71	3.71	3.65	3.8	3.56	79.46	0.14
					2010	14	2.64	3.71	1.07	3.26	2.77	2.74	111.48	0.29
Charleston	SC 461	S	AC	1996	2013	17	0	3.25	3.25	3.55	3.9	3.62	87.06	0.17
Charleston	I 526	E	PCC	1991	2014	23	15.87	27.5	11.63	3.09	4.38	3.83	121.57	0.05
					2012	21	15.9	27.578	11.678	3.13	4.5	3.93	118.75	0.07

(Source: Integrated Transportation Management System, SCDOT)

*Note: BMP=Beginning Mile Post, EMP=End Mile Post, PSI=Present Serviceability Index, PDI=Pavement Distress Index, PQI=Pavement Quality Index, IRI=International Roughness Index.

Table D-1 Pavement Performance Data of Selected Pavement Sections (cont.)

County	**Section**	**Direction**	**Type**	**Let date**	**Survey Year**	**Age (years)**	**BMP**	**EMP**	**Length (miles)**	**PSI**	**PDI**	**PQI**	**IRI (in./mile)**	**Rut (in.)**
					2010	19	15.87	27.5	11.63	3.14	4.22	3.74	118.63	0.14
					2009	18	15.9	27.5	11.6	3.15	4.1	3.65	117.59	0.11
					2008	17	15.9	27.5	11.6	3.21	4.52	3.97	113.16	0.13
					2007	16	15.9	27.5	11.6	3.22	4.73	4.11	112.31	0.12
					2006	15	16	27.5	11.5	3.28	4.65	4.08	107.05	0.15
Charleston	I 526	W	PCC	1991	2014	23	15.858	27.54	11.682	2.97	4.63	3.98	131.45	0.05
					2012	21	15.82	27.54	11.72	3.03	4.57	3.95	126.6	0.07
					2010	19	15.9	27.54	11.64	3.06	4.29	3.77	124.79	0.13
					2009	18	15.9	27.54	11.64	3.11	4.36	3.83	119.71	0.1
					2008	17	15.9	27.54	11.64	3.17	4.42	3.88	114.97	0.11
					2007	16	15.9	27.54	11.64	3.17	4.76	4.12	114.96	0.12
					2006	15	15.9	27.54	11.64	3.18	4.6	4.02	114.22	0.14
Chester	SC 9	N	AC	1999	2014	15	15.58	28.36	10.15	3.59	2.88	2.87	84.12	0.16
					2012	13	15.58	28.36	10.17	3.65	3.32	3.22	79.3	0.21
					2011	12	15.58	28.36	10.17	3.66	3.48	3.34	78.5	0.22
					2009	10	15.58	28.36	10.01	3.68	3.93	3.67	77.21	0.18
					2008	9	15.58	28.36	10.17	3.69	3.99	3.7	76.88	0.18
Chester	SC 9	S	AC	1999	2014	15	17.42	28.228	10.92	3.37	3.49	3.28	102.17	0.21
					2012	13	17.42	28.409	10.99	3.41	3.24	3.11	99.37	0.27
					2011	12	17.42	28.3	10.88	3.46	3.36	3.19	94.5	0.28
					2009	10	17.42	28.3	10.78	3.5	3.66	3.42	90.81	0.19
					2008	9	17.42	28.39	10.87	3.55	3.78	3.52	87.22	0.2
Chesterfield	SC 151	N	AC	1999	2013	14	16.323	22.026	5.7	3.93	1.85	1.96	60.69	0.2
					2009	10	16.4	22	5.6	4.04	3.98	3.78	53.93	0.19
Chesterfield	SC 151	S	AC	1999	2013	14	16.383	21.992	5.61	3.69	1.92	2.06	77.07	0.3
					2009	10	16.4	22	5.6	3.86	3.66	3.51	64.98	0.26
Fairfield	I 77	N	PCC	1980	2012	32	33.8	48.19	14.39	3.21	4.46	3.92	112.23	0.06

Table D-1 Pavement Performance Data of Selected Pavement Sections (cont.)

County	Section	Direction	Type	Let date	Survey Year	Age (years)	BMP	EMP	Length (miles)	PSI	PDI	PQI	IRI (in./mile)	Rut (in.)
					2011	31	33.8	48.16	14.36	3.25	4.46	3.93	109.32	0.06
					2010	30	33.78	48.1	14.32	3.2	4.54	3.98	112.53	0.11
					2009	29	33.792	48.18	14.09	3.3	4.57	4.02	104.84	0.1
					2008	28	33.8	48.2	14.2	3.27	4.54	4	107.23	0.08
					2007	27	33.9	48.18	13.98	3.25	4.6	4.03	108.52	0.1
					2006	26	33.8	48.2	14.1	2.95	4.58	3.93	137.37	0.1
Fairfield	I 77	S	PCC	1980	2012	32	33.7	48.1	14.4	3.28	4.44	3.93	107.11	0.06
					2011	31	33.772	48.1	14.34	3.32	4.5	3.98	103.84	0.06
					2010	30	33.7	48.1	14.38	3.28	4.53	3.99	106.54	0.11
					2009	29	33.7	48.1	14.1	3.3	4.56	4.02	104.39	0.09
					2008	28	33.7	48.1	14.08	3.34	4.6	4.05	101.39	0.08
					2007	27	33.7	48.1	14.02	3.32	4.62	4.07	102.77	0.09
					2006	26	33.7	48.1	14.02	3.35	4.63	4.08	101.04	0.1
Florence	SC 327	N	AC	1992	2013	21	17.41	22.11	4.7	3.38	0.54	0.63	99.38	0.41
					2010	18	17.41	22.1	4.69	3.55	2.32	2.43	86.44	0.35
					2006	14	17.41	22.1	4.59	3.71	2.78	2.81	75.34	0.29
Florence	SC 327	S	AC	1992	2013	21	17.41	22.085	4.68	3.54	1.03	1.23	88.44	0.48
					2010	18	17.41	22.1	4.69	3.68	1.87	2.07	78.57	0.4
					2006	14	17.41	22.1	4.59	3.81	2.81	2.86	69.19	0.32
Florence	US 301	N	AC	2003	2013	10	27.7	29.79	2.09	3.68	3.69	3.5	77.12	0.13
					2011	8	27.7	29.7	2	3.78	4.34	3.98	70.1	0.07
Georgetown	US 521	N	AC	2003	2010	7	16	19.8	3.8	3.7	4.29	3.93	76.18	0.14
					2008	5	16.1	19.1	3	3.79	4.45	4.06	70.25	0.11
					2007	4	16.1	19.1	3	3.81	4.43	4.05	68.65	0.12
Georgetown	US 521	S	AC	2003	2010	7	16	19.7	3.7	3.75	4.25	3.91	72.07	0.16
					2008	5	16	20	3	3.86	4.41	4.05	65.29	0.13
Greenville	I 385	N	AC	2003	2014	11	22.62	42.104	15.59	3.84	3.36	3.27	68.06	0.07

Table D-1 Pavement Performance Data of Selected Pavement Sections (cont.)

County	Section	Direction	Type	Let date	Survey Year	Age (years)	BMP	EMP	Length (miles)	PSI	PDI	PQI	IRI (in./mile)	Rut (in.)
					2012	9	22.62	42.16	19.54	3.48	3.48	3.27	96	0.31
					2011	8	22.62	41.9	19.28	3.65	3.63	3.44	82.24	0.13
					2010	7	22.62	42.1	19.48	3.56	3.6	3.4	88.05	0.14
					2009	6	22.62	42.16	18.24	3.65	3.83	3.58	81.22	0.13
					2008	5	22.62	42.16	19.14	3.57	3.9	3.62	89.44	0.12
					2007	4	22.62	42.16	19.14	3.43	3.69	3.43	98.01	0.18
					2006	3	22.62	42.16	19.24	3.46	3.79	3.52	95.28	0.19
Greenville	I 385	S	AC	2003	2014	11	22.62	42.05	15.58	3.75	3.45	3.31	73.62	0.08
					2012	9	22.62	42.16	19.54	3.61	3.43	3.25	83.96	0.1
					2011	8	22.62	42.16	19.54	4.66	3.27	3.31	19.41	0.11
					2010	7	22.62	42.16	19.54	3.54	3.22	3.12	89.03	0.18
					2009	6	22.62	42.16	18.14	3.63	3.56	3.38	81.99	0.17
					2008	5	22.62	42.16	18.94	3.65	3.74	3.52	80.6	0.15
					2007	4	22.62	42.16	19.24	3.61	3.63	3.43	83.27	0.18
					2006	3	22.62	42.16	19.14	3.58	3.92	3.64	85.08	0.18
Greenville	I 85	N	AC	2005	2014	9	40.6	55.89	15.29	3.74	3.97	3.7	75.36	0.08
					2012	7	40.6	55.89	15.19	3.83	3.99	3.73	68.16	0.09
					2011	6	40.6	55.89	15.3	3.78	3.54	3.38	71.35	0.12
					2010	5	42.6	55.89	13.09	3.73	3.44	3.29	74.49	0.16
					2009	4	40.7	55.89	14.89	3.79	3.53	3.38	69.65	0.16
					2008	3	40.7	55.89	14.89	3.81	3.68	3.49	68.59	0.14
					2007	2	40.7	55.89	14.89	3.8	3.78	3.56	69.1	0.15
					2006	1	40.7	55.89	14.79	3.81	3.87	3.63	68.28	0.14
Greenville	I 85	S	AC	2005	2014	9	40.6	55.89	15.29	4.4	4.13	3.88	54.18	0.06
					2012	7	40.6	55.89	15.19	3.81	4.37	4	70.88	0.06
					2011	6	40.6	55.89	15.29	3.78	3.72	3.52	71.45	0.1
					2010	5	40.6	55.89	15.29	3.74	3.26	3.18	73.26	0.16

Table D-1 Pavement Performance Data of Selected Pavement Sections (cont.)

County	Section	Direction	Type	Let date	Survey Year	Age (years)	BMP	EMP	Length (miles)	PSI	PDI	PQI	IRI (in./mile)	Rut (in.)
					2009	4	40.7	55.89	14.99	3.82	3.09	3.06	68.1	0.17
					2008	3	40.7	55.89	15.09	3.83	3.41	3.29	67.28	0.15
					2007	2	40.6	55.89	14.89	3.87	3.61	3.44	64.39	0.14
					2006	1	40.6	55.89	14.99	3.86	3.67	3.49	64.97	0.15
Horry	SC 22	E	AC	2001	2013	12	0	29.39	29.39	3.7	2.17	2.25	76.74	0.19
					2012	11	4.54	28.5	23.96	3.82	3.13	3.11	68.41	0.15
					2009	8	4.54	27.7	19.56	4	3.94	3.74	56.16	0.18
					2008	7	4.54	27.8	20.16	4.02	4.18	3.92	54.95	0.15
					2007	6	0	27.5	24.6	4.04	4.19	3.93	53.72	0.17
Horry	SC 22	W	AC	2001	2013	12	0	28.856	28.86	3.74	2.67	2.7	74.3	0.17
					2009	8	4.5	27.6	20.2	3.97	4	3.78	58.2	0.17
					2008	7	4.441	27.7	20.36	4	4.18	3.91	56.47	0.16
					2007	6	0.12	27.7	24.58	4.03	4.23	3.96	54.03	0.16
Horry	SC 31	N	AC	2005	2013	8	0	24.33	24.33	3.94	2.95	2.95	60.51	0.15
					2007	2	0	24.33	23.83	4.17	4.35	4.06	45.78	0.11
					2005	0	0	24.33	23.73	4.18	4.68	4.3	44.91	0.07
Horry	SC 31	S	AC	2005	2013	8	0	24.33	24.31	3.94	3.56	3.43	60.36	0.11
					2007	2	0	24.2	24	4.16	4.55	4.2	46.55	0.15
					2005	0	0	24.33	23.22	4.2	4.55	4.21	43.97	0.11
Laurens	SC 72	E	AC	2002	2014	12	1.77	12.788	11.02	3.59	3.16	3.09	83.91	0.09
					2012	10	1.77	12.8	11.03	3.6	3.71	3.5	82.98	0.13
					2010	8	1.77	12.8	11.03	3.68	4.32	3.94	76.88	0.13
					2009	7	1.77	12.8	10.83	3.71	4.42	4.02	74.88	0.12
					2008	6	1.77	12.8	11.03	3.71	4.47	4.06	75.09	0.11
					2007	5	1.77	12.8	10.73	3.74	4.59	4.15	72.9	0.09
Laurens	SC 72	W	AC	2002	2014	12	1.77	12.564	9.54	3.64	3.77	3.54	80.08	0.08
					2012	10	1.9	12.71	8.8	3.67	3.8	3.57	78.01	0.15

Table D-1 Pavement Performance Data of Selected Pavement Sections (cont.)

County	Section	Direction	Type	Let date	Survey Year	Age (years)	BMP	EMP	Length (miles)	PSI	PDI	PQI	IRI (in./mile)	Rut (in.)
					2010	8	1.9	12.72	8.71	3.76	4.17	3.85	71.56	0.14
Lexington	S 378	E	PCC	2001	2014	13	0	0.7	0.7	3.22	4.66	4.07	110.86	0.07
					2012	11	0	0.7	0.7	3.17	4.84	4.17	115.14	0.11
					2009	8	0	1.28	1.28	3.32	4.92	4.26	103.05	0.08
					2008	7	0	1.28	1.28	3.33	4.92	4.27	102.16	0.08
Orangeburg	US 321	N	AC	2004	2012	8	9.3	15.9	6.6	3.48	3.84	3.54	92.45	0.18
					2010	6	9.3	15.9	6.63	3.46	4.07	3.71	94.18	0.18
					2009	5	9.3	15.9	6.5	3.48	4.06	3.71	93.16	0.19
					2008	4	9.3	15.9	6.5	3.49	4.24	3.84	91.91	0.12
Pickens	SC 93	N	AC	2001	2014	13	0	1.3	1.3	2.6	2.21	2.2	169.6	0.14
					2008	7	0	1.3	1.3	2.82	3.96	3.49	150.43	0.19
Pickens	SC 93	S	AC	2001	2014	13	0.06	0.516	0.46	3.21	2.16	2.21	111.75	0.16
Spartanburg	SC 80	E	PCC	2000	2014	14	0	4.98	4.98	3.36	4.98	4.32	100.86	0.14
					2010	10	0	4.98	4.98	3.41	4.92	4.29	96.66	0.08
Spartanburg	SC 80	W	PCC	2000	2014	14	0.08	4.98	4.9	3.31	4.99	4.3	105.17	0.12
					2010	10	2	4.98	4.68	3.36	4.93	4.28	100.89	0.09
Spartanburg	I 85	N	PCC	1997	2012	15	69.01	77.28	8.23	3.18	4.64	4.04	114.29	0.08
					2011	14	69.1	77.28	8.16	3.25	4.66	4.07	108.61	0.08
					2010	13	69.01	77.28	8.24	3.22	4.68	4.08	111.27	0.12
					2009	12	69.01	77.28	8.04	3.31	4.7	4.11	104.11	0.08
					2008	11	69.1	77.2	7.8	3.33	4.63	4.08	102.18	0.07
					2007	10	69.1	77.28	7.85	3.31	4.66	4.09	103.87	0.08
					2006	9	69.1	77.2	7.7	3.32	4.77	4.16	102.91	0.09
Spartanburg	I 85	S	PCC	1997	2012	15	69.01	77.28	8.24	3.19	4.55	3.99	113.68	0.08
					2011	14	69.01	77.28	8.27	3.22	4.6	4.03	111.58	0.08
					2010	13	69.01	77.28	8.18	3.2	4.56	3.99	112.73	0.12
					2009	12	69.01	77.28	7.93	3.29	4.71	4.12	105.65	0.08

Table D-1 Pavement Performance Data of Selected Pavement Sections (cont.)

County	Section	Direction	Type	Let date	Survey Year	Age (years)	BMP	EMP	Length (miles)	PSI	PDI	PQI	IRI (in./mile)	Rut (in.)
					2008	11	69.01	77.28	7.93	3.31	4.65	4.08	103.72	0.08
					2007	10	69.01	77.28	7.88	3.31	4.67	4.1	104.21	0.1

Appendix E – In-Situ Soil Data and UCS Test Results

Table E-1 Soil Sample Data and Unconfined Compression Strength (Orangeburg)

Sample	H (mm)	D (mm)	H/D	V (ft3)	Moist_W (lb)	Dry_W (lb)	Moist_UW (lb/ft3)	Dry_UW (lb/ft3)	MC (%)	UCS (kPa)
111	142	74	1.92	0.0216	2.587	2.297	120	106	13	26
223	157	74	2.13	0.0235	3.043	2.716	129	115	12	44
221	152	74	2.07	0.0229	2.650	2.411	116	105	10	66
211	159	73	2.18	0.0235	2.699	2.464	115	105	10	46
121	154	77	1.99	0.0255	2.668	2.555	104	100	4	56
513	139	75	1.85	0.0218	2.650	2.426	121	111	9	
512	148	75	1.96	0.0233	2.609	2.417	112	104	8	62
511	150	73	2.04	0.0224	2.794	2.533	125	113	10	90
412	156	74	2.10	0.0238	2.858	2.684	120	113	6	80
411	164	74	2.22	0.0247	3.040	2.792	123	113	9	100
313	164	73	2.23	0.0245	3.224	2.957	132	121	9	160
312	135	74	1.83	0.0203	2.670	2.422	131	119	10	150
613	161	75	2.16	0.0250	3.076	2.876	123	115	7	48
612	147	73	2.01	0.0219	2.672	2.497	122	114	7	40
713	156	76	2.06	0.0249	2.708	2.545	109	102	6	47
712	150	74	2.03	0.0227	2.657	2.503	117	110	6	75
711	148	73	2.02	0.0220	2.369	2.169	108	98	9	70
822	149	74	2.01	0.0227	2.714	2.497	120	110	9	40
821	135	74	1.82	0.0207	2.376	2.152	115	104	10	40
912	154	75	2.06	0.0240	2.930	2.661	122	111	10	80
911	162	75	2.16	0.0253	3.083	2.763	122	109	12	54
811	154	74	2.08	0.0233	2.674	2.497	115	107	7	65

Note: H, D, V = height, diameter, and volume of the specimen respectively, MC = moisture content, W = weight, UW = unit weight. UCS = unconfined compression strength.

Table E-1 Soil Sample Data and Unconfined Compression Strength (Orangeburg) (cont.)

Sample	**H (mm)**	**D (mm)**	**H/D**	**V (ft3)**	**Moist_W (lb)**	**Dry_W (lb)**	**Moist_UW (lb/ft3)**	**Dry_UW (lb/ft3)**	**MC (%)**	**UCS (kPa)**
812	145	76	1.92	0.0230	2.542	2.391	110	104	6	58
1014	170	73	2.32	0.0253	3.073	2.754	122	109	12	85
1013	159	73	2.17	0.0236	2.846	2.542	120	108	12	110
1012	157	73	2.14	0.0236	2.995	2.696	127	114	11	180
1011	166	73	2.27	0.0246	3.159	2.894	128	117	9	60
1123	157	73	2.16	0.0232	2.706	2.494	117	107	9	50
1122	151	72	2.09	0.0220	2.736	2.454	125	112	11	105
1121	142	75	1.89	0.0223	2.734	2.415	122	108	13	
1111	164	72	2.29	0.0233	2.626	2.408	113	103	9	55
1221	128	76	1.68	0.0205	2.324	2.145	113	104	8	83
1212	129	75	1.71	0.0203	2.389	2.208	118	109	8	60
1211	139	75	1.86	0.0214	2.570	2.360	120	110	9	65
1312	135	75	1.80	0.0211	2.397	2.271	114	108	6	80
1321	147	74	1.99	0.0222	2.578	2.418	116	109	7	80
1311	145	75	1.94	0.0226	2.588	2.438	114	108	6	80

Note: H, D, V = height, diameter, and volume of the specimen respectively, MC = moisture content, W = weight, UW = unit weight. UCS = unconfined compression strength.

Table E-2 Soil Sample Data and Unconfined Compression Strength (Georgetown)

Sample	H (mm)	D (mm)	H/D	V (ft3)	Moist_W (lb)	Dry_W (lb)	Moist_UW (lb/ft3)	Dry_UW (lb/ft3)	MC (%)	UCS (kPa)
114	164	73	2.24	0.0245	3.244	2.887	132	118	12	230
113	156	73	2.12	0.0233	2.983	2.649	128	114	13	180
112	145	76	1.92	0.0230	2.703	2.442	117	106	11	70
111	142	74	1.92	0.0214	2.580	2.337	120	109	10	130
223	150	74	2.02	0.0230	2.858	2.574	124	112	11	94
222	155	74	2.10	0.0234	2.926	2.686	125	115	9	130
221	150	73	2.04	0.0223	2.774	2.508	124	113	11	130
211	151	73	2.07	0.0225	2.925	2.579	130	114	13	130
322	132	73	1.81	0.0196	2.247	2.091	115	107	7	60
321	138	73	1.88	0.0206	2.608	2.378	127	116	10	160
311	158	73	2.15	0.0237	3.019	2.753	127	116	10	160
421	143	76	1.88	0.0228	2.698	2.210	118	97	22	
411	141	74	1.89	0.0217	2.452	2.264	113	104	8	50
513	145	77	1.89	0.0237	2.619	2.404	111	101	9	70
512	137	77	1.78	0.0224	2.474	2.283	110	102	8	58
511	142	76	1.86	0.0230	2.546	2.368	111	103	8	72
613	143	76	1.88	0.0230	2.542	2.282	110	99	11	30
611	131	75	1.74	0.0207	2.243	2.091	109	101	7	26
722	149	74	2.03	0.0223	2.511	2.278	112	102	10	26
721	150	73	2.04	0.0224	2.609	2.378	116	106	10	45

Note: H, D, V = height, diameter, and volume of the specimen respectively, MC = moisture content, W = weight, UW = unit weight. UCS = unconfined compression strength.

Table E-3 Soil Sample Data and Unconfined Compression Strength (Pickens)

Sample	H (mm)	D (mm)	H/D	V (ft3)	Moist_W (lb)	Dry_W (lb)	Moist_UW (lb/ft3)	Dry_UW (lb/ft3)	MC (%)	UCS (kPa)
311	163	73	2.23	0.0243	2.713	2.309	112	95	17	95
312	161	73	2.20	0.0240	2.819	2.357	117	98	20	140
313	167	73	2.28	0.0249	2.720	2.306	109	93	18	85
314	169	73	2.31	0.0250	2.634	2.283	105	91	15	95
215	164	73	2.24	0.0243	2.588	2.236	107	92	16	75
214	154	74	2.07	0.0236	2.330	1.887	99	80	23	45
213	164	73	2.25	0.0241	2.618	2.241	109	93	17	120
212	161	73	2.20	0.0239	2.477	2.083	104	87	19	100
211	164	73	2.24	0.0244	2.587	2.200	106	90	18	70
115	165	73	2.26	0.0245	3.024	2.600	124	106	16	140
114	162	73	2.21	0.0241	2.759	2.271	115	94	21	90
113	165	73	2.25	0.0244	2.546	2.056	105	84	24	80
112	158	73	2.16	0.0235	2.658	2.212	113	94	20	135
111	163	74	2.21	0.0245	2.575	2.227	105	91	16	75
512	135	73	1.84	0.0202	2.579	2.232	128	110	16	240
511	152	73	2.07	0.0228	2.869	2.473	126	108	16	170
525	161	73	2.20	0.0240	2.785	2.210	116	92	26	140
524	163	73	2.22	0.0242	2.959	2.503	122	103	18	170
523	163	73	2.25	0.0240	2.975	2.526	124	105	18	110
522	160	73	2.18	0.0239	3.057	2.618	128	110	17	180
521	151	73	2.05	0.0225	2.794	2.334	124	104	20	150
415	166	73	2.28	0.0245	3.246	2.832	133	116	15	350
414	159	73	2.17	0.0237	2.961	2.569	125	108	15	320

Note: H, D, V = height, diameter, and volume of the specimen respectively, MC = moisture content, W = weight, UW = unit weight. UCS = unconfined compression strength.

Table E-3 Soil Sample Data and Unconfined Compression Strength (Pickens) (cont.)

Sample	H (mm)	D (mm)	H/D	V (ft3)	Moist_W (lb)	Dry_W (lb)	Moist_UW (lb/ft3)	Dry_UW (lb/ft3)	MC (%)	UCS (kPa)
413	162	74	2.20	0.0243	2.894	2.477	119	102	17	140
412	160	73	2.19	0.0238	2.956	2.503	124	105	18	220
411	169	73	2.31	0.0252	3.255	2.792	129	111	17	400

Note: H, D, V = height, diameter, and volume of the specimen respectively, MC = moisture content, W = weight, UW = unit weight. UCS = unconfined compression strength.

Appendix F – Distress and Roughness Data

Table F-1 Measured IRI and Rut Data for AC Pavements

County	Section	Direction	Type	Let date	Survey Year	Age	Measured IRI (in./mi)	Measure Rut (in.)
Beaufort	US 278	E	AC	1996	2013	17	98.21	0.14
					2012	16	112.85	0.15
					2010	14	99.72	0.17
					2008	12	96.87	0.17
					2005	9	100.00	0.28
Beaufort	US 278	W	AC	1996	2013	17	95.39	0.15
					2012	16	97.37	0.24
					2010	14	94.34	0.20
					2008	12	92.82	0.20
					2005	9	155.91	0.13
Charleston	SC 461	N	AC	1996	2013	17	79.46	0.14
					2010	14	111.48	0.29
Charleston	SC 461	S	AC	1996	2013	17	87.06	0.17
Chester	SC 9	N	AC	1999	2014	15	84.12	0.16
					2012	13	79.30	0.21
					2011	12	78.50	0.22
					2009	10	77.21	0.18
					2008	9	76.88	0.18
Chester	SC 9	S	AC	1999	2014	15	102.17	0.21
					2012	13	99.37	0.27
					2011	12	94.50	0.28
					2009	10	90.81	0.19
					2008	9	87.22	0.20
Chesterfield	SC 151	N	AC	1999	2013	14	71.09	0.24
					2009	10	63.51	0.23
Chesterfield	SC 151	S	AC	1999	2013	14	90.77	0.31
					2009	10	78.37	0.28
					2005	6	75.43	0.22
Florence	SC 327	N	AC	1992	2013	21	99.38	0.41
					2010	18	86.44	0.35
					2006	14	75.34	0.29
Florence	SC 327	S	AC	1992	2013	21	88.44	0.48
					2010	18	78.57	0.40
					2006	14	69.19	0.32
Florence	US 301	N	AC	2003	2013	10	77.12	0.13
					2011	8	70.10	0.07

Table F-1 Measured IRI and Rut Data for AC Pavements (cont.)

County	Section	Direction	Type	Let date	Survey Year	Age	Measured IRI (in./mi)	Measure Rut (in.)
Georgetown	US 521	N	AC	2003	2012	9	60.68	0.22
					2010	7	58.79	0.25
					2009	6	55.56	0.21
					2008	5	57.01	0.22
					2007	4	55.51	0.19
Georgetown	US 521	S	AC	2003	2012	9	65.97	0.26
					2010	7	68.71	0.26
					2009	6	60.12	0.21
					2008	5	60.13	0.22
					2007	4	58.20	0.19
Greenville	I 385	N	AC	2000	2014	14	68.06	0.07
					2012	12	96.00	0.31
					2011	11	82.24	0.13
					2010	10	88.05	0.14
					2009	9	81.22	0.13
					2008	8	89.44	0.12
					2007	7	98.01	0.18
					2006	6	95.28	0.19
Greenville	I 385	S	AC	2000	2014	14	73.62	0.08
					2012	12	83.96	0.10
					2011	11	19.41	0.11
					2010	10	89.03	0.18
					2009	9	81.99	0.17
					2008	8	80.60	0.15
					2007	7	83.27	0.18
					2006	6	85.08	0.18
Greenville	I 85	N	AC	2005	2014	9	75.36	0.08
					2012	7	68.16	0.09
					2011	6	71.35	0.12
					2010	5	74.49	0.16
					2009	4	69.65	0.16
					2008	3	68.59	0.14
					2007	2	69.10	0.15
					2006	1	68.28	0.14
Greenville	I 85	S	AC	2005	2014	9	54.18	0.06
					2012	7	70.88	0.06
					2011	6	71.45	0.10
					2010	5	73.26	0.16

Table F-1 Measured IRI and Rut Data for AC Pavements (cont.)

County	Section	Direction	Type	Let date	Survey Year	Age	Measured IRI (in./mi)	Measure Rut (in.)
					2009	4	68.10	0.17
					2008	3	67.28	0.15
					2007	2	64.39	0.14
					2006	1	64.97	0.15
Horry	SC 22	E	AC	2001	2013	12	76.74	0.19
					2012	11	68.41	0.15
					2009	8	56.16	0.18
					2008	7	54.95	0.15
					2007	6	53.72	0.17
Horry	SC 22	W	AC	2001	2013	12	74.30	0.17
					2009	8	58.20	0.17
					2008	7	56.47	0.16
					2007	6	54.03	0.16
Horry	SC 31	N	AC	2005	2013	8	60.51	0.15
					2007	2	45.78	0.11
					2006	1	44.91	0.07
Horry	SC 31	S	AC	2005	2013	8	60.36	0.11
					2007	2	46.55	0.15
					2006	1	43.97	0.11
Laurens	SC 72	E	AC	2002	2014	12	83.91	0.09
					2012	10	82.98	0.13
					2010	8	76.88	0.13
					2009	7	74.88	0.12
					2008	6	75.09	0.11
					2007	5	72.90	0.09
Laurens	SC 72	W	AC	2002	2014	12	80.08	0.08
					2012	10	78.01	0.15
					2010	8	71.56	0.14
Orangeburg	US 321	N	AC	2004	2012	8	92.45	0.18
					2010	6	94.18	0.18
					2009	5	93.16	0.19
					2008	4	91.91	0.12
Pickens	SC 93	N	AC	2001	2014	13	169.60	0.14
					2008	7	150.43	0.19
Pickens	SC 93	S	AC	2001	2014	13	111.75	0.16

(Source: Division of Traffic Engineering and the Pavement Management Group, SCDOT)

Table F-2 Measured Distress Data for AC Pavements

County	Section	Let Date	Survey	Age	Fatigue (%)	Transverse (%)	Longitudinal (%)	Transverse (ft)
Beaufort	US 278	1996	2001	5	0.00	5.00	1.60	4.80
			2005	9	8.10	12.30	0.50	1.50
			2008	12	11.10	5.40	20.60	64.80
			2010	14	11.60	5.50	17.00	66.00
			2012	16	22.00	1.00	20.00	12.00
Charleston	SC 461	1996	2010	14	11.00	9.80	0.50	0.00
Chester	SC 9	1999	2000	1	0.00	23.86	42.21	286.26
			2005	6	1.10	100.00	58.12	1200.00
			2009	10	1.70	52.22	24.62	626.60
			2014	15	28.49	57.25	83.01	686.98
Chesterfield	SC 151	1999	2000	1	17.10	25.10	20.60	301.20
			2005	6	3.00	25.10	20.00	301.20
			2009	10	2.20	15.00	13.20	180.00
Florence	SC 327	1992	2001	9	8.00	15.00	1.40	180.00
			2006	14	35.10	1.00	3.00	3.00
			2010	18	43.40	1.50	0.80	11.70
Florence	US 301	2003	2011	8	2.20	0.90	20.00	10.80
Georgetown	US 521	2003	2004	1	11.00	5.00	40.00	60.00
			2007	4	0.00	7.70	20.00	92.40
			2008	5	0.10	5.50	21.00	66.00
			2009	6	0.50	5.00	0.00	60.00
			2010	7	1.00	5.10	22.00	61.20
			2012	9	1.90	5.00	22.00	60.00
Greenville	I 385	2000	2001	1	1.30	25.00	40.00	180.00
			2002	2	3.00	15.00	21.60	60.00
			2003	3	6.80	6.30	41.20	75.60
			2004	4	11.00	25.00	20.00	300.00
			2005	5	11.10	50.00	11.20	600.00
			2006	6	11.10	0.00	1.20	48.00
			2007	7	11.10	20.00	20.00	181.20
			2008	8	11.10	10.80	0.80	12.00
			2009	9	11.10	10.90	35.00	68.40
			2010	10	11.00	11.20	30.20	134.40
			2011	11	20.60	5.00	32.80	60.00

Table F-2 Measured Distress Data for AC Pavements (cont.)

County	Section	Let Date	Survey	Age	Fatigue (%)	Transverse (%)	Longitudinal (%)	Transverse (ft)
			2012	12	9.90	0.00	20.00	0.00
			2014	14	30.58	28.64	54.09	343.78
Greenville	I 85	2005	2006	1	2.00	5.00	20.00	60.00
			2007	2	11.00	0.20	0.00	2.40
			2008	3	4.30	0.20	10.00	2.40
			2009	4	3.00	0.10	3.00	1.20
			2010	5	3.00	14.80	40.00	177.60
			2011	6	5.60	8.30	20.00	60.00
			2012	7	3.40	12.30	20.00	63.60
			2014	9	24.27	9.00	28.07	97.61
Horry	SC 22	2001	2007	6	3.00	25.00	20.00	300.00
			2008	7	3.20	25.00	3.40	300.00
			2009	8	5.00	15.00	20.00	180.00
			2012	11	11.70	23.70	4.20	284.40
Horry	SC 31	2005	2006	1	0.00	4.00	3.10	10.40
			2007	2	0.00	0.00	0.10	4.00
Laurence	SC 72	2002	2003	1	45.00	0.00	20.00	0.00
			2007	5	0.00	0.30	0.00	3.60
			2008	6	0.00	0.00	0.00	0.00
			2009	7	0.50	0.00	0.00	0.00
			2010	8	2.10	0.00	0.00	0.00
			2012	10	11.00	0.00	0.00	0.00
			2014	12	20.13	21.95	33.54	263.41
Orangeburg	US 321	2004	2006	2	3.00	0.00	0.00	0.00
			2008	4	45.00	12.80	20.00	153.60
			2009	5	45.00	0.30	11.80	3.60
			2010	6	22.00	0.10	20.00	1.20
			2012	8	45.00	0.20	21.40	2.40
			2013	9	45.10	0.10	20.00	1.20
Pickens	SC 93	2001	2004	3	11.10	0.60	21.00	7.20
			2007	6	4.80	0.00	23.80	0.00
			2008	7	11.10	50.00	12.80	600.00
			2014	13	107.34	93.42	112.45	1121.06

(Source: Division of Traffic Engineering and the Pavement Management Group, SCDOT)

Table F-3 Measured Distress Data for PCC Pavements

County	Section	Let Date	Survey	IRI (Mea)	Age	Transverse Cracking (%)
Aiken	I 520	2008	2009	85.65	1	0.00
			2010	61.44	2	7.50
			2011	50.79	3	7.50
			2012	56.35	4	0.10
			2014	87.33	6	19.24
Charleston	I 526	1991	1999	87.28	8	5.10
			2000	87.93	9	5.60
			2001	100.01	10	5.00
			2002	101.75	11	5.00
			2003	106.86	12	0.60
			2004	104.77	13	16.20
			2005	106.17	14	25.00
			2006	105.81	15	1.30
			2007	108.44	16	0.00
			2008	114.36	17	0.00
			2009	115.37	18	25.00
			2010	123.32	19	20.00
			2012	117.97	21	9.00
			2014	120.83	23	19.30
Fairfield	I 77	1980	1982	98.85	2	0.00
			2000	98.51	20	25.00
			2001	98.51	21	21.40
			2002	97.06	22	1.20
			2003	101.59	23	20.00
			2004	98.58	24	11.90
			2005	99.496	25	39.30
			2006	131.61	26	7.40
			2007	104.43	27	25.00
			2008	103.77	28	25.00
			2009	102.32	29	25.10
			2010	110.04	30	20.00
			2011	107.89	31	15.00
			2012	110.02	32	15.00
			2014	104.67	34	19.03
Lexington	S 378	2001	2003	179.79	2	0.00
			2008	102.12	7	0.00
			2009	103.03	8	1.90
			2012	115.14	11	0.00
			2014	110.86	13	8.80

Table F-3 Measured Distress Data for PCC Pavements (cont.)

County	Section	Let Date	Survey	IRI (Mea)	Age	Transverse Cracking (%)
Spartanburg	SC 80	2000	2010	96.88	10	7.50
			2014	102.98	14	0.00
Spartanburg	I 85	1997	1999	85.50	2	15.00
			2000	92.05	3	21.10
			2001	89.99	4	15.00
			2002	87.91	5	5.00
			2003	89.68	6	14.80
			2004	88.91	7	13.90
			2005	94.25	8	6.70
			2006	96.61	9	5.00
			2007	94.94	10	0.20
			2008	95.32	11	0.20
			2009	96.63	12	0.10
			2010	103.51	13	14.80
			2011	98.38	14	8.30
			2012	105.34	15	12.30
			2014	114.79	17	9

(Source: Division of Traffic Engineering and the Pavement Management Group, SCDOT)

Appendix G – Literature Review of Instrumentation

The USC research team conducted a literature review on instrumentation of special pavement test sections to build a solid base of knowledge on current practice and research. The key findings from different literatures are described below.

NCAT Structural Test Truck

National Center for Asphalt Technology (NCAT) Pavement Test Track constructed 46 test sections with various asphalt mixtures from 2000 to 2003. They constructed several new sections and eight sections are reconstructed to simulate actual pavement section found on the highway systems (Timm et al., 2004). The test sections were equipped with different gauges to measure asphalt strain, base and subgrade pressures, and pavement temperature. Their research objective was to calibrate the subroutines used in AASHTOWare Pavement ME Design Guide for better pavement response predictions.

MnROAD Research Project

Minnesota Road (MnROAD) was the first test track since the AASHTO Road Test of the 1960s. Over 4500 sensors are installed in a 500 ft test section. The main focus of the research was to develop mechanistic model to design pavement (Newcomb et al., 1990). In addition, they evaluated the performance of different pavement materials under different pavement conditions.

Virginia Smart Road

In Virginia Smart Road project, 12 flexible pavements were instrumented to evaluate the performance of different mixes, calibrate the pavement responses to FWD testing and feasibility of using Ground Penetration Radar (GPR) as a pavement assessing tool (Loulizi et al., 2001; Nassar 2001). A complex array of sensors was embedded during construction located beneath the roadway. The environmental sensors used includes thermocouples for temperature measurements, time domain reflectometry probes to measure moisture content in the base layers, and resistivity probes to measure stresses and strains, respectively. Environmental sensors are induced at different layers from truck loading which collect data daily every 15 min for temperature, every hour for moisture, and every 6 hour for frost penetration. Truck testing was performed every week with different loading configurations. The loading variables included three load levels, three wheel inflation pressures, and four different speeds.

Pennsylvania Instrumentation Project

The Pennsylvania Department of Transportation (PennDOT) constructed eight instrumented test sections to calibrate and validate the M-E performance models during 2001-2006. Firstly, the material properties used in different sections were determined by laboratory testing and FWD tests. Then, traffic and climate data were collected. Finally, field performance data was compared with M-E predictions. It was concluded from that study, the AASHTOWare Pavement ME Design software made reasonable predictions only for rutting. Finite element analysis of test sections was also conducted. It was reported that ABAQUS under predicts vertical stresses by about 22% and horizontal strain by about 35% compared to the KENLAYER. However, ABAQUS predicted horizontal strain was up to 70% lower and vertical stress was up to 64% lower than the measured values in an instrumentation section.

New Mexico Instrumentation Project

The New Mexico Department of Transportation (NMDOT) constructed an instrumented section on Interstate 40 (I-40) with 32 sensors including strain gauges, pressure plates, moisture probes, temperature probes, axle sensing strips, weather station and Weigh-in-Motion (WIM) station (Tarefder and Islam, 2015). The sensor data were collected continuously. Pavement materials were collected during the construction and were tested in the laboratory. Routine field testing including Falling Weight Deflectometer (FWD) and density tests, and field survey were conducted to monitor the in-situ properties and performances of the pavement section. As the pavement ME software was not able to generate stress-strain file in the past, a KENLAYER software was used to predict stress-strain. It is found that measured and vertical stress is less than predicted stress and measured horizontal strain differs vey slightly from predicted strain. A goal of that project was to compare the ME predicted distresses with measured distresses. However, the measured performance data from this section are very small, which was expected from a newly constructed pavement. Therefore, project data collection and monitoring were recommended for continuation.

New Hampshire MEPDG Project

The New Hampshire Department of Transportation (NHDOT) constructed instrumented test sections for MEPDG project in 2009. Pavement sensors, including earth-pressure sensors, temperature, and moisture probes, and asphalt strain gages were installed throughout the construction of the pavement. Connectivity and data acquisition were established with an on-site instrumentation cabinet. A weather station was installed within the vicinity during the same time period. An array of axle sensor strips was installed following the completion of the surface course. Following the installation of most of the sensors and the completion of the binder course, a calibration truck run was performed. Full-size falling weight deflectometer (FWD) testing was performed after the completion of the surface course. The data collected throughout the project was used to provide inputs for the modeling of the pavement section and the prediction of the distress performance using AASHTOWare 1.100.