Calibration of the AASHTO Pavement Design Guide to South Carolina Conditions - Phase II

FINAL REPORT

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14. ABSTRACT
This report presents the findings from a Phase II study undertaken to calibrate the distress and performance models of the AASHTO Mechanistic-Empirical Pavement Design Guide (MEPDG) with full consideration to South Carolina conditions. Research included the collection of high-quality and high-priority materials data for asphalt and portland cement concrete pavement sections across South Carolina; the collection of pavement distress and performance data for these pavements; the installation of Weigh-In-Motion stations to collect site-specific traffic data; the development of an asphalt concrete pavement thickness design catalog for high volume roads; and the local calibration of the MEPDG distress and performance models for new flexible and rigid pavements. Local calibration coefficients were obtained for the "Flexible New AC" and "Rigid New JPCP" distress and performance models in the MEPDG using the input data compiled for South Carolina pavements in this study with the AASHTOWare Online Calibration Assistance Tool. Distresses were predicted with the Pavement ME Design software. The results show that the optimized calibration coefficients reduce error in the MEPDG distress and performance predictions and better assist the design of flexible and rigid pavements using MEPDG in South Carolina.

15. SUBJECCT TERMS
Mechanistic-Empirical Pavement Design Guide, Pavement Design, Flexible Pavement, Rigid Pavement

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Executive Summary

This report presents the findings from a Phase II study undertaken to calibrate the distress and performance models of the AASHTO Mechanistic-Empirical Pavement Design Guide (MEPDG) to South Carolina conditions. This study builds upon the Phase I study that was conducted to identify existing historical data (i.e., climate, traffic, pavement structure, material properties, and pavement performance) from high-traffic primary and interstate routes in South Carolina that could be used in the calibration efforts. Historical records for new asphalt concrete (AC) and portland cement concrete (PCC) pavement sections were compiled, revealing limited site-specific data available for calibration. Thus, the efforts in this Phase II study included the collection of high-quality and high-priority materials data for AC and PCC pavement sections across South Carolina, the collection of pavement distress and performance data for these pavements, the installation of Weigh-In-Motion stations to collect site-specific traffic data, the development of an asphalt concrete pavement thickness design catalog for high volume roads, and the local calibration of the MEPDG distress and performance models for new flexible and rigid pavements.

To support the local calibration of the distress and performance models of the MEPDG, extensive laboratory and field studies were performed to collect high-priority, high-quality materials data for flexible and rigid pavements in South Carolina. A review of historic pavement design files produced 11 AC pavement sections and 11 PCC pavement sections for this Phase II study. These sections were further subdivided into smaller segments based on FWD testing and manual distress surveys, resulting in a total of 76 AC pavement segments (66 for “Flexible New AC” models and 10 for “Flexible New Semi-Rigid” models) and 24 PCC pavement segments (for “Rigid New JPCP” models) available for calibration. Materials data was collected for each

segment and used to establish Level 1, Level 2, or Level 3 inputs. For AC pavements, a number of asphalt mix types were sampled and characterized from multiple asphalt plants across the state. For PCC pavements, concrete mix was sampled from three new pavements at the time of construction. The resilient modulus of the subgrade soil was obtained from repeated load triaxial testing on tube samples collected from beneath each of the pavement segments. Relations between the laboratory-derived resilient modulus and Falling Weight Deflectometer (FWD) data, CBR tests, and SSV, were developed in this study to aid the SCDOT in future pavement design.

Pavement distress and performance data was collected for each pavement segment. Two sources of data were available: manual distress surveys performed in this study and historical data retrieved from the SCDOT Pavement Management System (PMS). Manual distress surveys were performed coincident with FWD test locations for 76 AC pavement segments and 24 PCC pavement segments. For asphalt pavements, measurements of fatigue cracking, transverse cracking, and rutting were collected. For PCC pavements, mid-slab cracking and joint faulting measurements were collected. In addition to collecting visual surface distress data, two 4 in. diameter pavement cores were taken from each segment. Tube and bulk samples of subgrade soil were collected from beneath the pavement core. The SCDOT PMS provided historic IRI, rutting, and cracking data, for comparison with the visual distress data. Weigh-in-Motion (WIM) stations were installed at 19 pavement sites across the state of South Carolina. Each WIM station provides site-specific (Level 1) traffic data required for the MEPDG: the hourly distribution factor (HDF), the monthly adjustment factor (MAF), the vehicle class distribution (VCD), and the axle load factor (ALF) for each axle type (single, tandem, tridem and quad). The data is reported in a format that can be directly imported into the AASHTOWare Pavement ME Design software file for each pavement section.

An AC pavement design catalog for high-volume roads in South Carolina was developed by conducting a sensitivity analysis to determine a design asphalt thickness for a combination of variables (AADTT, subgrade type, aggregate base thickness, asphalt mix type, and climate station) using the bottom-up fatigue cracking results in Pavement ME Design software with global calibration factors. It was found that when using the MEPDG to design pavements, it is sufficient to have a single generic input each for surface, intermediate, and base asphalt mixes. Also, the addition of an 8 in. thick layer of a graded aggregate base resulted in an approximately 1-2 in. thinner asphalt layer when all other factors remained the same.

Local calibration coefficients were obtained for the "Flexible New AC" and "Rigid New JPCP" models in the MEPDG using the input data compiled for South Carolina pavements in this study with the AASHTOWare Online Calibration Assistance Tool. In this process, the distresses predicted by the Pavement ME Design software using the nationally calibrated coefficients were first compared with measured distresses for selected pavement sections. After local calibration of the “Flexible New AC” models, the bias and SEE were lower than the values for the global models for Bottom-up Fatigue Cracking, Rutting, and IRI, but not for Top-down Fatigue Cracking. Furthermore, the bias and SEE for the Rutting and IRI models were considered tolerable, whereas the values for Bottom-up Fatigue Cracking and Top-down Fatigue Cracking were not. For Bottom-up Fatigue Cracking, this is a situation where one or more hypothesis tests show a failing result, but the calibration can be accepted because there is a close to zero bias with a SEE lower than the global model (AASHTO 2020). Thus, the local model coefficients for Bottom-up Fatigue Cracking, Rutting, and IRI for "Flexible New AC" were accepted for South Carolina conditions. The local model coefficients for the Top-down Fatigue Cracking were not. For "Rigid New JPCP" the local coefficients for the IRI model were accepted. The local coefficients for the Transverse Cracking and Mean Transverse Joint Faulting models were not.

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List of Acronyms

AASHTO = American Association of State Highway and Transportation Officials
AC = Asphalt Concrete
ALF = Axle Load Factor
AMPT = Asphalt Mixture Performance Tester
ASTM = American Society for Testing and Materials
BMP = Beginning Milepost
CBR = California Bearing Ratio
CBR_U = California Bearing Ratio (unsoaked)
CBR_S = California Bearing Ratio (soaked)
CMS = Cement Modified Subbase
CS = Non-stabilized base: crushed stone
CSCS = Chemically stabilized: cement stabilized
CRA = Non-stabilized base: Cold recycled asphalt – RAP pulverized in place
EMP = Ending Milepost
FHWA = Federal Highway Administration
FWD = Falling Weight Deflectometer
GAB = Graded Aggregate Base
HDF = Hourly Adjustment Factor
HMA = Hot Mix Asphalt
HPMA= Highway Pavement Management Application
JPCP = Joint Plain Cement Concrete
MAF = Monthly Adjustment Factor
ME = Mechanistic-Empirical
MEPDG = Mechanistic-Empirical Pavement Design Guide
M_R = Resilient Modulus
$M_{R(Lab)}$ = Laboratory-measured Resilient Modulus
$M_{R(FWD)}$ = Resilient Modulus Backcalculated from FWD
MOR = Modulus of Rupture
NA = Not Available
OCAT = Online Calibration Assistance Tool
n = number of forecasted years
PCC = Portland Cement Concrete
PMS = Pavement Management System
SCDOT = South Carolina Department of Transportation
SSV = Soil Support Value
USC = University of South Carolina
VCD = Vehicle Class Distribution
WIM = Weigh-In-Motion

Introduction

The pavement design methodology currently used by the South Carolina Department of Transportation (SCDOT) is based on updates to the original 1961 procedure that was developed from the American Association of State Highway Officials (AASHO) Road Test and South Carolina (SC)-specific local calibration studies conducted at the University of South Carolina (USC) and Clemson University from approximately 1964 to 1973. The most recent version of the SCDOT pavement design guidelines was published in 2008; however, these procedures were . 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 SPR 708: Calibration of the AASHTO Pavement Design Guide to South Carolina Conditions-Phase I by Gassman and Rahman (2016), the primary objective 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 MEPDG for South Carolina. 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 three of these sections (i.e., 1 in the Piedmont Region and two in the Coastal Plain), field sample collection, Falling Weight Deflectometer (FWD) 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 three 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, all of the cracking and rutting data cannot be implemented into the 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, not the AC rutting. Because not all of the necessary data was available in the SCDOT files and databases, and the quality of the distress data was uncertain in the Phase I study, Gassman and Rahman (2016) deemed the local calibration factors to be preliminary and not recommended to be used for design until further research is performed in a Phase II study to obtain high quality, high priority data.

Research Objective

The overarching goal of this multi-phase research effort is to reduce design bias and increase the precision of the model predictions used in the MEPDG with full consideration of South Carolina local conditions. The objective of Phase II was to build upon the studies in Phase I to obtain local calibration factors and improve distress predictions by collecting new data of high priority.

Research Tasks

To meet the research objective, the project was divided into 9 work tasks. The first 8 tasks were part of the original proposed work and the last task was added after the project began.

Task 1: Identify Additional Pavement Sections (Lead: USC)

Task 2: AC Mixture Catalog for Pavement Design (Lead: Clemson)

Task 3: Collect distress survey data and perform trench studies (Lead: Clemson)

Task 4: Collect high-priority materials data (USC: PCC and subgrade; Clemson: AC)

Task 5: Determination of In-Place Asphalt |E*| and Subgrade M_R of Calibration Sections (Lead: Clemson/USC)

Task 6: Plan for Special Pavement Validation Sections (Lead: Clemson/USC)

Task 7: Local Calibration of Distress Models (Lead: USC)

Task 8: Develop a plan for WIM clusters (Lead: USC)

Task 9: AC Pavement Design Catalog (Lead: Clemson)

Project Work Plan

The research team was composed of researchers from the University of South Carolina (Dr. Sarah Gassman) and Clemson University (Dr. Brad Putman) who collaborated in the completion of this comprehensive study. Dr. Gassman was responsible for the overall direction and successful completion of the project by coordinating all research activities. Additionally, she led the data collection, materials characterization, and calibration activities in Tasks 1, 4, 7, and 8. Dr. Gassman was assisted by two graduate research assistants. Dr. Putman lead the asphalt material catalog development in Tasks 2 and 4b, the distress survey activities in Tasks 3, the investigation of the dynamic modulus in Task 5, the special test section development and analysis in Task 6, and the AC pavement design catalog development in Task 9. Dr. Putman was assisted by a research technician and four graduate research assistants. At both institutions, the graduate assistants were assisted by hourly research assistants throughout the duration of the project. Dr. Gassman coordinated the submission of the progress and final reports.

Organization of Report

This final report begins with the Introduction, Research Objectives, Project Work Plan, and Tasks. Next, a summary of the work performed for Tasks 1 to 9 is presented including the key findings for each task. Finally, the Conclusions, Recommendations, and Implementation Plan are presented. Detailed methodology and comprehensive findings from this study are presented in the following Supplemental Reports:

- Selection of Pavement ME Input Parameters for PCC Model Calibration
- Selection of Pavement ME Input Parameters for AC Model Calibration
- Pavement ME Input Parameters for Subgrade Material
- Local Calibration of PCC Distress Models using South Carolina Input Parameters
- Local Calibration of AC Distress Models using South Carolina Input Parameters
- Development of an Asphalt Mixture Catalog to Support the MEPDG in South Carolina
- Pavement Distress Evaluation to Support Local Calibration of the MEPDG in South Carolina
- Development of an Asphalt Pavement Design Catalog for High-Volume Roads in South Carolina
- Evaluation of Asphalt Dynamic Modulus Measured with Small-Scale Specimens

Summary of Work Performed and Findings by Task

A brief discussion of the findings and conclusions related to each task is summarized below:

Task 1: Identify Additional Pavement Sections

In the Phase I study, existing historical data within the SCDOT were compiled for the 20 pavement sections listed in Table 1. Fourteen of these pavements are AC, and six are PCC. The minimum number of recommended pavement sections for the local calibration and validation of the distress models in MEPDG per AASHTO (2010) is:

- Distortion (Total Rutting or Faulting) - 20 roadway segments
- Load Related Cracking - 30 roadway segments
- Non-Load-Related Cracking - 26 roadway segments

At the start of this Phase II study, four of the pavement sections noted in Table 1 had been rehabilitated or reconstructed, leaving 11 AC pavement sections and five PCC pavement sections that were available for evaluation. Thus, there was an inadequate number of pavement sections available for local calibration of the performance indicators in the list.

To identify additional pavement sections, the research team initially used the same approach from Phase I by consulting with the SCDOT Pavement Design Group and reviewing the historic pavement design files available at the SCDOT Office of Materials and Research to see if there were any additional suitable pavement design files. This method produced six additional PCC sections, as summarized in Table 2. However, this method did not produce any additional AC pavement sections. In total, there were 11 AC pavement sections, as located in Figure 1, and 11 PCC pavement sections, as located in Figure 2, that were available for this study.

Table 1 Pavement Sections Identified in Phase I

County	Location	Type	Length (miles)	Let date	Design File No.
Beaufort	US-278	AC	1.56	3/13/1998	7.558
Charleston	SC-461	AC	2.48	5/21/1996	10.195A
Chester	SC-9	AC	7.12	10/1/1999	12.606
Chesterfield	SC-151	AC	5.36	12/15/1999	13.585
Florence	SC-327	AC	5.09	2/25/1992	21.873
Florence[1]	US-301	AC	2.38	9/30/2003	21.147A
Georgetown	US-521	AC	4.07	6/1/2003	22.619
Greenville[1]	I-385	AC	7.65	8/28/2000	23.038621
Greenville[1]	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
Orangeburg	US-321	AC	6.17	7/1/2004	38.157A
Pickens	SC-93	AC	1.34	4/10/2001	39.730
Aiken	I-520	PCC	5.35	7/25/2008	2.140B
Charleston	I-526	PCC	2.39	6/25/1991	810.482
Fairfield	I-77	PCC	14.17	10/21/1980	20.437
Lexington	S-378	PCC	1.47	11/1/2001	32.128A
Spartanburg	SC-80	PCC	3.30	6/1/2000	42.108B
Spartanburg[1]	I-85_1	PCC	6.29	6/11/1997	42.146A.1

[1]Not available for distress survey evaluation in Phase II due to rehabilitation or reconstruction.

Table 2 Pavement Sections Identified in Phase II

County	Location	Type	Length (miles)	Let date	Design File No.
Berkeley	I-526	PCC	6.17	3/19/1987	810.482
Charleston	S-97	PCC	0.45	11/01/2001	10.439A
Cherokee	I-85	PCC	21	9/29/1917	P027114
Lexington	I-20	PCC	11	12/21/2018	P027003
Spartanburg[a]	I-85_2	PCC	8	01/10/2017	P029074

[a]Spartanburg/I-85_2 is a rebuild of Spartanburg/I-85_1

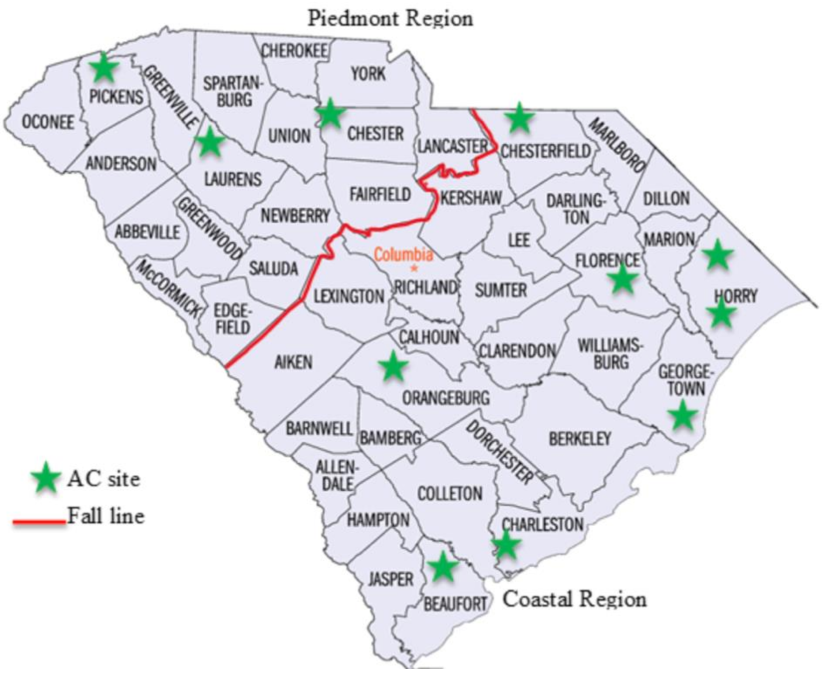


Figure 1 Map Showing Locations of AC Pavement Sections

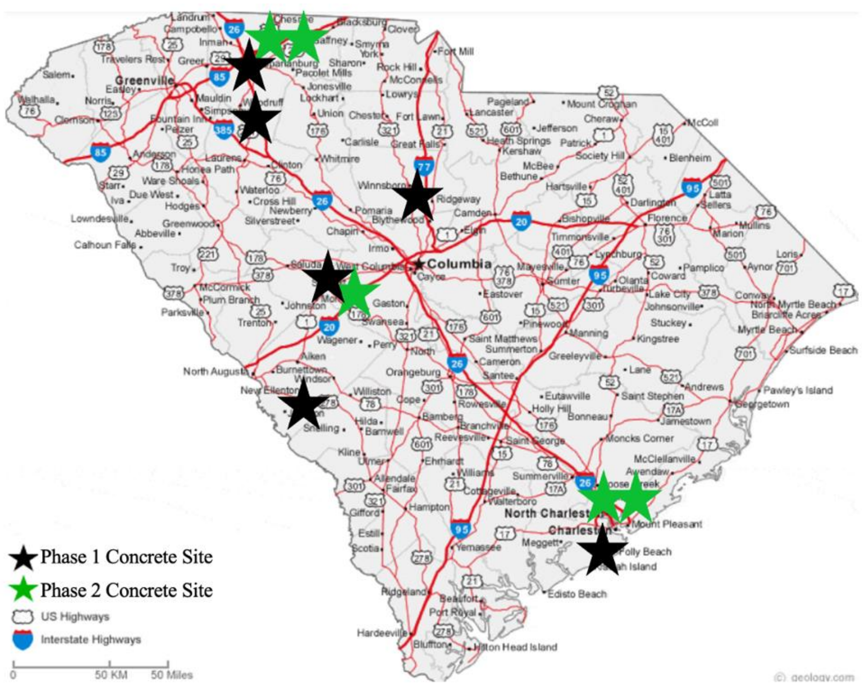


Figure 2 Map Showing Locations of PCC Pavement Sections

To maximize the number of roadway segments available for calibration, each of the 11 AC sections and four of the PCC pavement sections were divided into shorter segments (see Figure 3), along which pavement distress surveys were performed. Details of the segmentation procedure are presented in Appendix D of *Supplemental Report: Pavement ME Input Parameter for Subgrade Material*. In total, 78 segments were obtained from the 11 AC pavement sections and 12 from the four PCC pavement sections, as summarized in Table 3 for the AC pavements and Table 4 for the PCC pavements. Each segment was used as a separate segment for calibration.

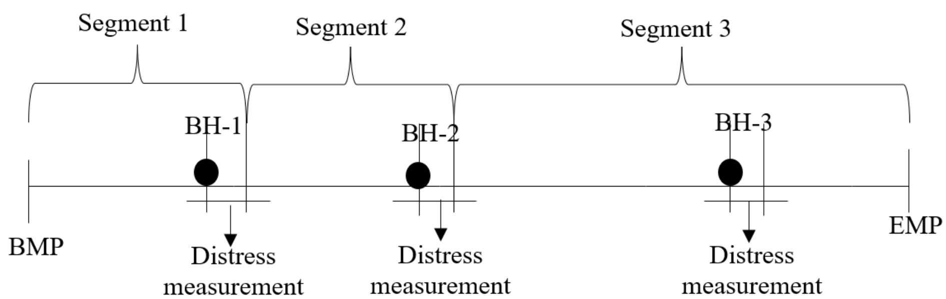


Figure 3 Schematic Showing Selection of Roadway Segment Length (BH=bore hole)

Table 3 Summary of Segment Length for AC Pavement Sections

Pavement Section	Segment	BMP	EMP	Length (mile)	Segment Id
Beaufort /US-278	1	19.1	19.3	0.2	AC_B278_S1
	2	19.3	19.5	0.2	AC_B278_S2
	3	19.5	20.0	0.5	AC_B278_S3
Charleston /SC-461	1	2.5	2.1	0.4	AC_C461_S1
	2	2.1	1.2	0.9	AC_C461_S2
	3	1.2	0.0	1.2	AC_C461_S3
Chester /SC-9	1	33.0	33.5	0.5	AC_C9_S1
	2	33.5	34.4	0.9	AC_C9_S2
	3	34.4	34.9	0.5	AC_C9_S3
Chesterfield /SC-151	1	16.0	16.2	0.2	AC_C151_S1
	2	16.2	16.4	0.2	AC_C151_S2
	3	16.4	16.7	0.3	AC_C151_S3
	4	16.7	17.1	0.4	AC_C151_S4
	5	17.1	17.6	0.5	AC_C151_S5
	6	17.6	18.0	0.4	AC_C151_S6
	7	18.0	18.3	0.3	AC_C151_S7
	8	18.3	18.5	0.2	AC_C151_S8
	9	18.5	19.0	0.5	AC_C151_S9
	10	19.0	20.0	1.0	AC_C151_S10
	11	20.0	21.4	1.4	AC_C151_S11
Florence /SC-327	1	17.5	17.7	0.2	AC_F327_S1
	2	17.7	18.0	0.3	AC_F327_S2
	3	18.0	18.2	0.2	AC_F327_S3
	4	18.2	18.5	0.3	AC_F327_S4
	5	18.5	19.0	0.5	AC_F327_S5
	6	19.0	20.0	1.0	AC_F327_S6
	7	20.0	21.0	1.0	AC_F327_S7
	8	21.0	22.4	1.4	AC_F327_S8
Georgetown /US-521	1	19.7	19.5	0.2	AC_G521_S1
	2	19.5	19.3	0.2	AC_G521_S2
	3	19.3	18.8	0.5	AC_G521_S3
	4	18.8	18.5	0.3	AC_G521_S4
	5	18.5	18.1	0.4	AC_G521_S5
	6	18.1	17.8	0.3	AC_G521_S6
	7	17.8	16.7	1.1	AC_G521_S7
Horry /SC-22	1	0.0	0.4	0.4	AC_H22_S1
	2	0.4	0.8	0.4	AC_H22_S2
	3	0.8	1.0	0.2	AC_H22_S3

Table 3 (cont.) Summary of Segment Length for AC Pavement Sections

Pavement Section	Segment	BMP	EMP	Length (mile)	Segment Id
Horry /SC-31	1	4.4	3.9	0.5	AC_H31_S1
	2	3.9	3.6	0.3	AC_H31_S2
	3	3.6	3.3	0.3	AC_H31_S3
	4	3.3	3.1	0.2	AC_H31_S4
	5	3.1	2.9	0.2	AC_H31_S5
	6	2.9	2.5	0.4	AC_H31_S6
	7	2.5	2.1	0.4	AC_H31_S7
	8	2.1	1.8	0.3	AC_H31_S8
	9	1.8	1.4	0.4	AC_H31_S9
	10	1.4	1.0	0.4	AC_H31_S10
	11	1.0	0.4	0.6	AC_H31_S11
Orangeburg /US-321	1	15.4	15.2	0.2	AC_O321_S1
	2	15.2	15.0	0.2	AC_O321_S2
	3	15.0	14.7	0.3	AC_O321_S3
	4	14.7	14.5	0.2	AC_O321_S4
	5	14.5	14.2	0.3	AC_O321_S5
	6	14.2	13.9	0.3	AC_O321_S6
	7	13.9	13.3	0.6	AC_O321_S7
	8	13.3	12.7	0.6	AC_O321_S8
	9	12.7	12.2	0.5	AC_O321_S9
	10	12.2	11.6	0.6	AC_O321_S10
	11	11.6	11.0	0.6	AC_O321_S11
	12	11.0	10.5	0.5	AC_O321_S12
	13	10.5	9.3	1.2	AC_O321_S13
Laurens /SC-72	1	9.4	9.8	0.4	AC_L72_S1
	2	9.8	11.8	2.0	AC_L72_S2
	3	11.8	12.3	0.5	AC_L72_S3
	4	12.3	12.5	0.2	AC_L72_S4
	5	12.5	12.9	0.4	AC_L72_S5
	6	12.9	13.1	0.2	AC_L72_S6
	7	13.1	13.3	0.2	AC_L72_S7
	8	13.3	13.8	0.5	AC_L72_S8
	9	13.8	14.7	0.9	AC_L72_S9
	10	14.7	14.9	0.2	AC_L72_S10
	11	14.9	15.1	0.2	AC_L72_S11
	12	15.1	15.3	0.2	AC_L72_S12
	13	15.3	15.5	0.2	AC_L72_S13
Pickens /SC-93	1	1.4	1.1	0.3	AC_P93_S1
	2	1.1	0.9	0.2	AC_P93_S2
	3	0.7	0.5	0.2	AC_P93_S3
	4	0.5	0.3	0.2	AC_P93_S4
	5	0.3	0.1	0.2	AC_P93_S5

Table 4 Summary of Segment Length for PCC Pavement Sections

Pavement Section	Segment	BMP	EMP	Length (mile)	Segment Id
Aiken /I-520	1	17.5	18.6	1.1	2_PCC_I520_1
	2	18.6	19.3	0.7	2_PCC_I520_2
	3	19.3	23.0	3.7	2_PCC_I520_3
Charleston /I-526	1	23.5	26.7	3.2	2_PCC_I526_1
	2	26.7	27.1	0.4	2_PCC_I526_2
	3	27.1	27.5	0.4	2_PCC_I526_3
Lexington /S-378	1	1.5	1.3	0.2	2_PCC_S378_1
	2	1.3	0.6	0.7	2_PCC_S378_2
	3	0.6	0.0	0.6	2_PCC_S378_3
Spartanburg /SC-80	1	1.5	1.7	0.2	2_PCC_S80_1
	2	1.7	2.9	1.2	2_PCC_S80_2
	3	2.9	4.9	2.0	2_PCC_S80_3

Task 2: AC Mixture Catalog for Pavement Design

The primary objective of this task was to measure the dynamic modulus of select asphalt mixes from across the state of South Carolina, and then to characterize the variability of the dynamic modulus. This information was then used to develop a catalog of the typical asphalt mixture input values for the design of asphalt pavements using the MEPDG in South Carolina.

The dynamic modulus of Hot Mix Asphalt (HMA) is the most significant material-related input parameter in the structural design of an asphalt pavement using the MEPDG. The dynamic modulus is used to determine the stress/strain responses needed by the performance models to predict the pavement performance with the MEPDG. These values have a direct influence on the fatigue (bottom-up and top-down) cracking and rutting (Yu and Shen 2012). Therefore, a thorough characterization of asphalt materials and appropriate input values is crucial to design pavement using the MEPDG.

For a Level 1 input, the dynamic modulus is determined in the laboratory in accordance with the procedure outlined in AASHTO T 378-17: *Standard Method of Test for Determining the Dynamic Modulus and Flow Number for Asphalt Mixtures Using the Asphalt Mixture Performance Tester (AMPT)*. Cyclic loads are applied to the specimen across a range of temperatures and frequencies, and a master curve is generated using time-temperature superposition. The master curve is then used as the input in the MEPDG. For Level 2 inputs, instead of measuring the dynamic modulus value in the lab, it is estimated using predictive models and the aggregate gradation, mixture volumetrics, and asphalt binder properties (Witczak and Fonseca 1996). Typical models used to predict the dynamic modulus are the Witczak model (Bari and Witczak 2006) and the Hirsch model (Christensen et al. 2003).

An important parameter that affects the dynamic modulus other than a source of aggregate is the aggregate gradation. Flintsch et al. (2008) concluded that the dynamic modulus is sensitive to the mix constituents, including aggregate type, asphalt content, reclaimed asphalt pavement (RAP) content, etc. Additionally, studies by Ali et al. (2016) showed that the main factors that contribute to the dynamic modulus are temperature, frequency, and the nominal maximum aggregate size (NMAS) of the aggregate in the mix.

Characterization of the dynamic modulus of asphalt is directly dependent on the source of aggregates since aggregates are the major component of HMA, and other factors like asphalt binder grade, binder content, and volumetrics of the mix. Therefore, this study was designed to measure the dynamic modulus data from six different asphalt mix types from five different asphalt plants across the state of South Carolina. Table 5 shows the contractors and respective plant locations selected to sample asphalt mixes for this study. Each location was selected from a different SCDOT district.

Table 5 Plant Locations Sampled with SCDOT Districts and Mix Types Sampled from Each

Contractor	Location	SCDOT District	Mix Types
Lane	Columbia, SC	1	Surface A, B, C Intermediate B, C Base A
King	Liberty, SC	3	Surface B, C Intermediate B, C Base A
Sloan	Duncan, SC	3	Surface A
CR Jackson	Jefferson, SC	4	Surface B, C Intermediate B, C Base A
Satterfield	Eureka, SC	7	Surface B, C Base A

Six different mix types that are more commonly used for higher volume roadways were considered for this study, each with a different gradation or binder grade (SCDOT 2018): Surface Type A, Surface Type B, Surface Type C, Intermediate B, Intermediate C, Base A. The mixes were collected during the 2017 and 2018 paving seasons in accordance with the SC-M-402 Supplemental Technical Specifications: *Materials Properties for Asphalt Mixtures* (SCDOT 2022). Each mix was sampled three times per day for three days at each plant. It should be noted that each plant did not produce all mixes during the study period.

The dynamic modulus of each mix was measured at an air void content of 7 ± 1% using an AMPT at test frequencies of 0.1, 1, 2, 5, 10, 25 Hz and at temperatures of 40, 70, 100 and 130°F (14, 21, 37, 54°C) in accordance with AASHTO T 378-17.

The purpose of this portion of the study was to assess the variability of dynamic modulus with respect to the mix type and contractor producing the mix. From this information, the development of typical MEPDG input values were developed for representative asphalt mixtures in South Carolina.

The results of this evaluation and comparison of the dynamic modulus of asphalt mixtures used in South Carolina informed the development of the mix design catalog and the following findings that are detailed in *Supplemental Report: Development of an Asphalt Mixture Catalog to Support the MEPDG in South Carolina.* An example of the data provided in the mix design catalog is included in Figure 4.

- The dynamic modulus of a mix from a particular production plant was consistent from production day to production day and from specimen to specimen. Therefore, it would be sufficient for a mix to be sampled from a single day of production on a regular basis (e.g., annually, when there are changes to the mix design, etc.). Additionally, using two test specimens as per AASHTO T378 is sufficient.
- At all temperatures, the differences between surface mixes for a given contractor were insignificant indicating that dynamic modulus input for a single "typical" surface mix from a given contractor can be used in design instead of separate input values for individual surface mix types. The same was generally true for the intermediate mixes. Only one asphalt base course mix type was evaluated.

Level 1

Asphalt Mix: Dynamic Modulus Table						
Temperature (°F)	Mixture |E*| (psi)					
	0.1 Hz	0.5 Hz	1 Hz	5 Hz	10 Hz	25 Hz
14	2383054	2578090	2648169	2782508	2829586	2883288
40	1569486	1893931	2022138	2286425	2385079	2501608
70	611723	908226	1050058	1393800	1541761	1731712
100	149460	270083	341996	560399	675964	845747
130	33888	63307	83292	156073	202392	280962

Asphalt Binder: Superpave Binder Test Data		
Temperature (°F)	Angular freq. =	
	G* (Pa)	Delta (degree)
136.4	3.46	85.4
147.2	1.51	86.9
158	0.708	88.0

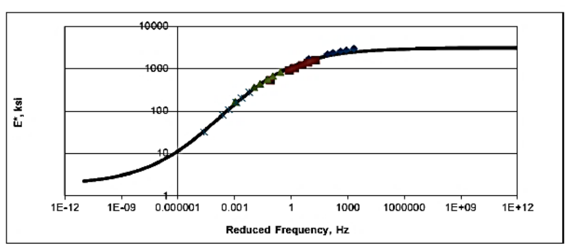
Asphalt General: Volumetric Properties as Built	
Effective Binder (%)	9.36
Air Voids (%)	6.97
Total unit weight	146.5

Level 2

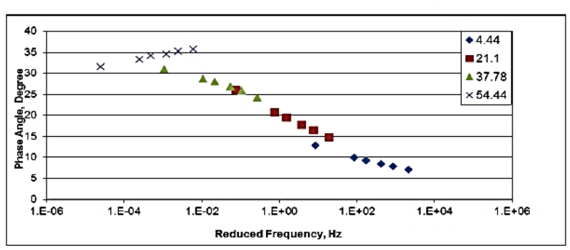
Asphalt Mix: Aggregate Gradation	
Cumulative % Retained ¾" sieve	3
Cumulative % Retained ⅜" sieve	35
Cumulative % Retained #4 sieve	49
% Passing #200 sieve	5.56

Level 3

Asphalt Binder: Superpave Binder Grading	PG 64-22



Master dynamic modulus curves for Day 2 mix from King Base A



Master phase angle curves for Day 2 mix from King Base A

Figure 4 Sample MEPDG Input Data for a Typical Asphalt Mixture

Task 3: Collect Distress Survey Data

The main objective of this task was to collect and assess pavement distress data to support the overall calibration of the MEPDG in South Carolina. The results of the pavement distress evaluation were compiled and used in the calibration process documented in Task 7. There were two sources of pavement distress data available for this task: manual distress survey data collected by Clemson University in this study and data from the SCDOT Pavement Management System (PMS) retrieved by the USC team. Both data sources were utilized in the Phase II study, whereas only the SCDOT PMS was utilized in the Phase I study.

Task 3a: Collect Manual Distress Survey Data

This task involved developing a protocol to appropriately sample the calibration sections, conduct surface distress evaluations, and obtain pavement cores and subgrade specimens. Details of these procedures are presented in *Supplemental Report: Pavement Distress Evaluation to Support Local Calibration of the MEPDG in South Carolina*.

There were a total of eleven asphalt and four concrete pavement sections included in this effort as noted in Table 6. The surface distress evaluation was conducted visually using the process outlined in ASTM D6433: *Standard Practice for Roads and Parking Lots Pavement Condition Index Surveys* as a guide. Due to the length of the sections and the manual nature of the evaluation, each pavement section was divided into 500 ft long segments that were randomly sampled per ASTM D6433. The number of segments evaluated was dependent on the length of the section.

The surface evaluation procedure was based on the LTPP Distress Identification Manual (Miller and Bellinger, 2014) to ensure consistency in the evaluation. As much as possible, the person conducting the distress evaluation was kept consistent throughout, which also helped with consistency. The pavement distress data collected for asphalt and concrete pavements included the distresses included in the pavement performance prediction in the MEPDG, as listed in Table 7.

Table 6 Asphalt and Concrete Pavement Sections where Distress Data was Obtained

Pavement Type	County/Route	Construction Finish Year	BMP	EMP	Length of Section (miles)
AC	Beaufort/US-278	1998	19.1	20.7	1.6
	Charleston/SC-461	1996	2.5	0.0	2.5
	Chester/SC-9	1999	33.0	34.9	1.9
	Chesterfield/SC-151	1999	16.0	21.4	5.4
	Florence/SC-327	1992	17.5	22.4	4.9
	Georgetown/US-521	2003	19.7	16.7	3.0
	Horry/SC-22	2001	0.0	1.0	1.0
	Horry/SC-31	2005	4.4	0.4	4.0
	Orangeburg/US-321	2004	15.4	9.3	6.1
	Laurens/SC-72	2002	9.4	15.5	6.1
	Pickens/SC-93	2001	1.4	0.2	1.2
PCC	Aiken/I-520	2009	17.4	22.9	5.3
	Charleston/I-526	1991	22.8	26.2	2.3
	Lexington/S-378	2001	0.1	1.5	1.5
	Spartanburg/SC-80	2000	1.5	4.9	3.3

Table 7 Pavement Distress Data Collected During Surface Evaluations

Pavement Type	Distress	Severity Level	Unit
AC	Bottom-up Fatigue Cracking	Low, Moderate, High	ft^{2*}
	Top-down Fatigue Cracking	Low, Moderate, High	ft^{2*}
	Transverse Cracking	Low, Moderate, High	ft/mile
	Rutting	N/A	Avg. rut depth
PCC	Mid-slab Cracking	N/A	Number of slabs
	Joint Faulting	N/A	Number of slabs

*converted to (% lane area) for Pavement ME Design software

In addition to collecting visual surface distress data, two 4 in. diameter cores were taken from each segment. One core was taken from a location exhibiting cracking (typically a wheel path) and the other from a non-distressed location within 10 ft of the distressed core (Figure 5). The cores were evaluated to gather information about the as-built pavement cross-section and layer thicknesses as well as to identify whether fatigue cracking was top-down or bottom-up. Figure 6 shows an example of the cores collected. Photographs of all cores are presented in Appendix B of *Supplemental Report: Selection of Pavement ME Input Parameters for AC Model Calibration*.

After removing the core from each location, subgrade samples were collected for the analysis completed as part of Task 4c. In core locations where an aggregate base layer was present, the aggregate was removed prior to sampling the subgrade soil. A bulk soil sample was collected from the distressed core hole using a 4 in. diameter hand auger, and two augers worth of soil was collected and stored in bags for transport. A moisture-content specimen was collected from each location and stored in an air-tight glass container. Undisturbed soil specimens were collected from the non-distressed core hole using a Shelby tube after removing the first 6 in. of soil. Two Shelby tube specimens were collected from each of these locations, then soil was hand augured and collected to a depth of 5 ft from the top of the subgrade.

Figure 5 Photo of Coring and Collecting Samples

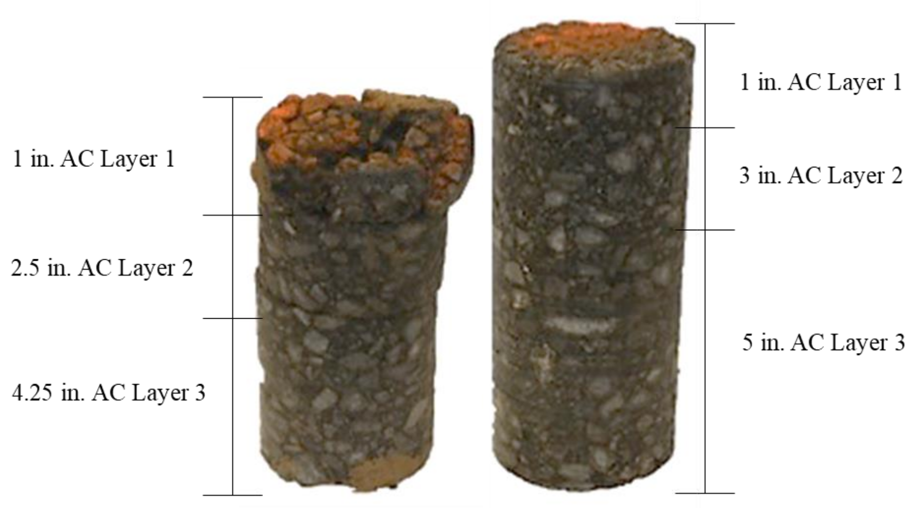


Figure 6 Photos of Cores Collected from a Distressed Location of a Pavement (left) and Non-Distressed Location (right) with Dimensions of each Asphalt Layer

Table 8 provides an example of manual distress survey data collected in the task that was used in the calibration process. Data for all segments are presented in Table 2.36 of *Supplemental Report: Selection of Pavement ME Input Parameters for AC Model Calibration*.

Table 8 Example of Pavement Distress Data Collected for Calibration

Letting Year	Evaluation Year	Pavement Age (months)	Segment Number	Bottom-up Cracking (% lane area)	Top-down Cracking (% lane area)	Transverse Cracking (ft/mile)	Rutting (in)
1992	2019	324	1	0	13.9	6758	0.55
			2	0	15.5	6093	0.55
			3	0	16.8	8015	0.51
			4	0	15.4	13823	0.44
			5	0	16.6	7033	0.87
			6	1	16.6	7033	0.64
			7	0	16.2	8839	0.66
			8	0	16.0	10212	0.49

Task 3b: Collect Distress Survey Data from PMS

As an example of the distress data collected from the SCDOT PMS, Table 9 presents the pavement survey year, age (month) of distress collection, pavement distresses, and IRI data for the three segments (S1, S2 and S3) of Beaufort/US-278 (see Table 3 for the BMP, EMP and length of each segment). Data for all segments are presented in Appendix F of *Supplemental Report: Selection of Pavement ME Input Parameters for AC Model Calibration*. Note that the Pavement ME Design software requires a separate row for each age and distress entry (see Table 2.38 in the Supplemental Report).

Table 9 Pavement Distresses and Roughness Data for Beaufort/US-278 from SCDOT PMS

Survey Year	Age (Month)	Fatigue Cracking (% lane area)	Longitudinal Cracking (% lane area)	Transverse Cracking (ft/mile)	Rut (in)	IRI (in/mile)
Segment Id: AC_B278_S1						
2001	36	0*	0*	0*	0.12	120
2002	48	0*	0*	0*	0.13	134
2005	84	8	0*	0*	0.14	113
2008	120	6	2	108	0.13	129
2009	132	0*	0*	0*	0.17	156
2010	144	2	0*	0*	0.13	154
2012	168	10	0*	0*	0.12	158
2014	192	0*	0*	0*	0.15	162
2015	204	0*	0*	0*	0.15	174
2016	216	26	5	230	0.18	175
2017	228	0*	0*	0*	0.12	176
2019	252	36	17	544	0.17	165
2021	264	0*	1	0*	0.13	110
Segment Id: AC_B278_S2						
2001	36	0*	0*	0*	0.14	113
2002	48	0*	0*	0*	0.20	117
2005	84	1	0*	0*	0.18	121
2008	120	4	5	264	0.12	126
2009	132	0*	0*	0*	0.16	130
2010	144	6	1	33	0.14	135
2012	168	13	0*	0*	0.12	120
2013	180	60	2	108	0.14	138
2014	192	0*	0*	0*	0.20	147
2015	204	0*	0*	0*	0.17	158
2016	216	21	7	384	0.24	144
2017	228	0*	0*	0*	0.19	146
2019	252	23	16	465	0.16	148
2021	264	0*	1	0*	0.14	64
Segment Id: AC_B278_S3						
2001	36	0*	0*	0*	0.17	94
2002	48	0*	0*	0*	0.17	93
2005	84	1	1	0*	0.20	102
2008	120	6	13	181	0.22	104
2009	132	6	3	0*	0.20	113
2010	144	0*	0*	0*	0.19	113
2012	168	11	19	0*	0.19	108
2013	180	37	1	8	0.19	135
2014	192	0*	0*	0*	0.17	109
2015	204	0*	0*	0*	0.17	116
2016	216	18	6	284	0.23	109
2017	228	0*	0*	0*	0.21	122
2019	252	24	8	392	0.17	139
2021	264	8	1	67	0.14	64

*Not measured data (i.e., cracking data is assumed to be non-zero given let date of 1998)

Task 4: Collect High-Priority Materials Data

The need for materials data that was identified in the Phase I study is summarized in Table 10. High priority was assigned to properties identified in the literature through sensitivity analyses and other studies as having the greatest impact on pavement design using the MEPDG. In this study, the high-priority data listed in Table 10 were obtained and compiled for the representative materials of the pavement sections in Tables 1 and 2 through extensive field and laboratory investigations. The data was used to establish Level 1, Level 2, or Level 3 inputs for local calibration of MEPDG distress models. The USC team led the efforts to collect the PCC (Task 4a) and subgrade (Task 4c) material inputs. The Clemson team led the efforts to collect the AC material inputs (Task 4b).

Table 10 Material Data Needs with Priority to Obtain Level 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, Compressive Strength, Unit Weight	High
	Thermal Conductivity, Heat Capacity	Medium
	Cement type, Aggregate Type, Cementitious Material Content, Water Cement Ratio, Ultimate Shrinkage, Reversible Shrinkage	Low

Task 4a: PCC Pavement

The required PCC properties to calibrate the jointed plain concrete pavement (JPCP) distress models in the Pavement ME Design software 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. None of this data was available in the SCDOT historical data files for the Phase I study and was not found through additional searches (i.e., f'_c from QC/QA reports) in this Phase II study; thus, extensive field and laboratory studies were conducted to obtain the high priority material parameters listed in Table 11 for the PCC sites.

Samples of pavement concrete were obtained at the time of construction for three newly constructed PCC pavements: Cherokee/I-85, Lexington/I-20, and Spartanburg/I-85. The design strength for the concrete pavement at Cherokee/I-85 and Spartanburg/I-85 was 4500 psi, and Lexington/I-20 was 5500 psi. Specimens were prepared in the field, and strength tests were performed in the Structures Laboratory at the University of South Carolina. At each site, 4x8 cylinders, 6x12 cylinders, and 6x6 beam specimens were fabricated from freshly prepared batch plant mixtures. The specimens were tested at 7, 14, 28, 90, and 365 days, as well as after 3 years (3+ yrs), for each site. The 3+ year data was used to establish the long-term strength gain of the concrete mixtures.

A list of the tests performed, ASTM standards, and parameters obtained is shown in Table 11. During casting, all mixtures were tested for the concrete slump, concrete temperature, wet concrete density, and ambient temperature. In the laboratory, tests were performed to obtain compressive strength (f'_c), modulus of elasticity (E), Poisson's ratio (v), modulus of rupture (MOR), and shrinkage, in general accordance with ASTM standards. The results for each specimen are summarized in Appendix A of *Supplemental Report: Selection of Pavement ME Input Parameters for PCC Model Calibration*. In addition to the material parameters obtained at the University of South Carolina, coefficient of thermal expansion (CTE) testing was performed per AASHTO T336 and ASTM C 138 at Clemson University on 4x8 cylinders that were cast for each of the three sites.

Table 11 Concrete Tests Performed and ASTM Standards

Test Performed	ASTM Standard	Parameter Obtained
Standard Test Method for Slump of Hydraulic-Cement Concrete	C143	Slump
Standard Test Method for Temperature of Freshly Mixed Hydraulic-Cement Concrete	C1064	PCC zero-stress temperature (deg F)
Standard Test Method for Density (Unit Weight), Yield, and Air Content (Gravimetric) of Concrete	C138	Unit Weight & Air Content
Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens	C39	Compressive Strength, f'_c
Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression	C469	Modulus of Elasticity, E, Poisson's Ratio, v
Standard Test Method for Flexural Strength of Concrete (Using Simple Beam and Third Point Loading)	C78	Modulus of Rupture, MOR
Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete	C157	Ultimate Shrinkage, Time to Develop 50% of Ultimate Shrinkage
Standard Method of Test for Coefficient of Thermal Expansion of Hydraulic Cement Concrete	AASHTO T336	Coefficient of Thermal Expansion, CTE

Task 4b: Asphalt Concrete

Dynamic modulus, binder grades, air voids, effective binder content, and mix gradations are key inputs for AC layers for MEPDG. In the Phase I study, 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 project SPR 720, "Characterization of Asphalt Concrete Dynamic Modulus in South Carolina." None of the collected data represents project-specific (Level 1) information; thus, extensive testing was performed in Phase II to gather the required input data.

To support the local calibration efforts and obtain a broader database of material inputs that includes existing pavement materials that are not currently produced in South Carolina, the research team characterized current representative mixtures utilized in the construction of a limited number of asphalt pavements included in Table 1. The testing protocol used in Task 2 was used in this task as well, and the data is presented in a catalog format as in Task 2, which was utilized for the calibration process in Task 7. The results of this task are summarized in Section 2.4 and Appendix C of the *Supplemental Report: Selection of Pavement ME Input Parameters for AC Model Calibration*. Table 12 lists the tests performed, standards, and parameters obtained.

Table 12 Asphalt Tests Performed and Standards

Test Performed	Standard	Parameter Obtained
Standard Test Method of Test for Determining the Dynamic Modulus and Flow Number of Asphalt Mixtures Using the Asphalt Mixture Performance Tester (AMPT)	AASHTO T378-17	Dynamic Modulus
Standard Method of Test for Determination of Volumetric Properties in Asphalt Laboratory Compacted Specimens	SC-T-68	% Air Voids, % Voids in Mineral Aggregate (VMA), % Binder by Volume, Bulk Specific Gravity, Unit Weight
Standard Method of Test for Determination of Asphalt Binder Content of Asphalt Mixtures using the Ignition Oven	SC-T-75*	% Binder by Weight
Standard Method of Test for Determination of Maximum Theoretical Specific Gravity	SC-T-83	Maximum Specific Gravity
Standard Method of Test for Determination of Dry Aggregate Gradation of Hot Mix Asphalt Extracted Aggregates	SC-T-102*	Aggregate Gradation

**Notes tests conducted by the asphalt mix producer*

Task 4c: Subgrade

The research team from USC led the effort to collect the high-priority material properties required as input for the subgrade. These include resilient modulus, gradation, liquid limit, plasticity index, and dry unit weight, as listed in Table 10. Specific gravity and optimum moisture content (medium-level inputs) were also obtained. In Phase I, these data were obtained for three sites: US-321 in Orangeburg County, US-521 in Georgetown County, and SC-93 in Pickens County using the subgrade sampling and testing plan that was developed as part of the project. In Phase II, extensive testing was performed to gather the required input data for the remaining pavement sections investigated in this study. The number of pavement cores and Shelby tube samples for each pavement section are summarized in Tables 13 and 14.

Table 13 ***Summary of Samples Collected for the AC Sections***

County/Route	Total Length of Pavement Sections (miles)	Total Length of Pavement Sections (ft)	No. of Segments Investigated = No. of BH	No. of Pavement Cores	No. of Shelby Tubes
Beaufort/US-278	1.6	8448	3	6	6
Charleston/SC-461	2.5	13200	3	6	6
Chester/SC-9	1.9	10032	3	6	6
Chesterfield/SC-151	5.4	28512	11	20	20
Florence/SC-327	4.9	25872	8	12	12
Georgetown/US-521	3	15840	7	7*	19*
Horry/SC-22	1	5280	3	6	6
Horry/SC-31	4	21120	11	22	18
Laurens/SC-72	6.1	32208	11	22	22
Orangeburg/US-321	6.1	32208	13	13*	13*
Pickens/SC-93	1.2	6336	5	8	8*

*Samples obtained from SPR 708 Phase I.

Table 14 Summary of Samples Collected for the PCC Sections

County/Route	Total Length of Pavement Sections		No. of Segments Investigated = No. of BH	No. of Pavement Cores	No. of Shelby Tubes
	(miles)	(ft)			
Aiken/I-520	2.3	12144	3	3	3
Charleston/I-526	1.5	7920	3	3	3
Lexington/S-378	3.3	17424	3	3	3
Spartanburg/SC-80	5.3	27984	3	3	3

Soil samples were collected in conjunction with the coring studies performed in Task 3 for the distress surveys. Figure 7 shows photographs of the soil sampling process. The detailed procedures used to collect the thin-walled Shelby tubes and bulk soil samples are documented in Appendix B of the *Supplemental Report: Pavement ME Input Parameters for AC Subgrade Material*. The depths where the thin-walled Shelby tube samples and bulk samples were collected at Hole A and Hole B are shown in Figure 8. Bulk soil samples and samples for moisture content testing were obtained from Hole A. Two Shelby tube samples and a bulk sample of the subgrade soil were obtained from Hole B. The subgrade soil samples were collected to a depth ranging from 36 to 76 inches, depending on site conditions (see Table 2.2 of the *Supplemental Report: Pavement ME Input Parameters for AC Subgrade Material*).

At each borehole location in Tables 3 and 4, 3 ft long and 3 in. diameter Shelby tubes were used to collect high-quality soil samples from the same holes where distress measurements were obtained. These samples were used to obtain the resilient modulus by a repeated load triaxial test per AASHTO T307. Bulk soil samples were collected from an adjacent hole and used to conduct laboratory tests listed in Table 15. Soils were classified according to USCS (ASTM D2488) and AASHTO (AASHTO M145).

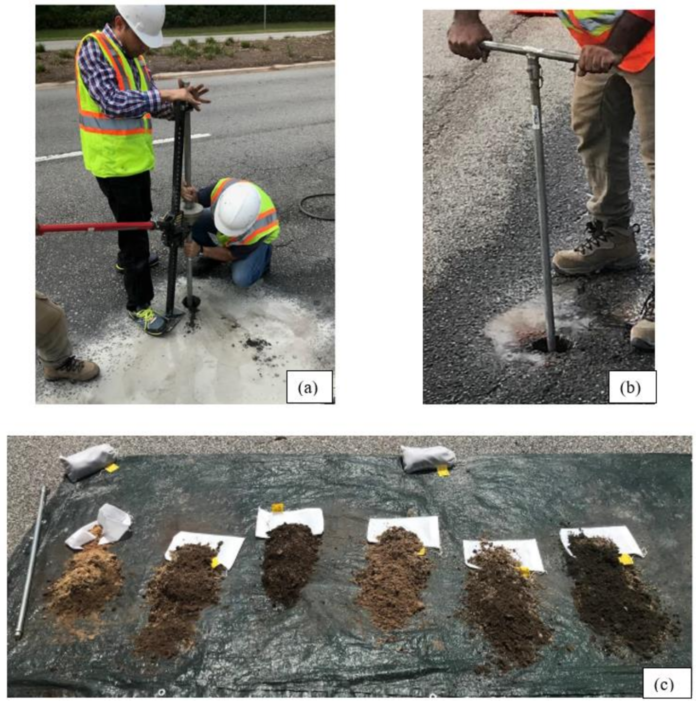


Figure 7 Subgrade Sample Collection Process: (a) Shelby Tube Sample Collected with Assistance from a Jeep Jack, (b) Bulk Sample Collected by Hand Auger, and (c) Bulk Samples

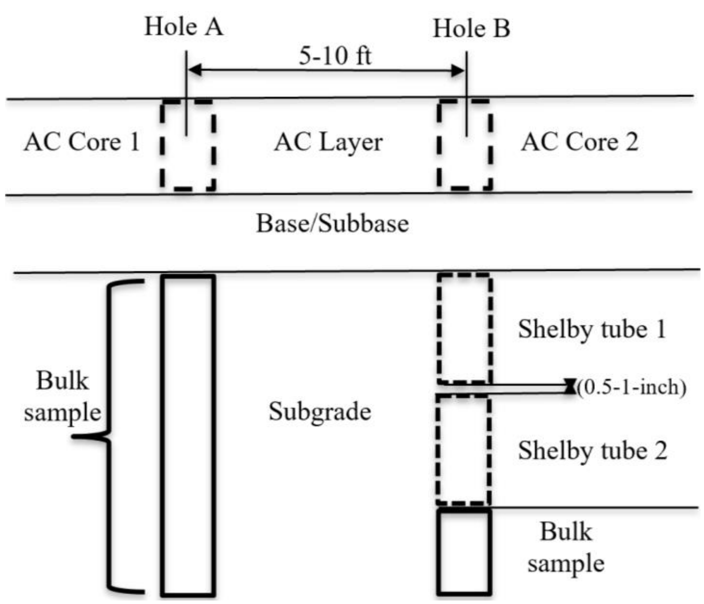


Figure 8 Schematic Diagram Showing the Coring and Sampling Locations with Depth

Table 15 Summary of Laboratory Tests Performed on Subgrade Soils

MEPDG Parameters	ASTM Standard	AASHTO Standard
Soil classification and gradation	ASTM D 6913, ASTM D 2487	AASHTO T 311, AASHTO M 145
Resilient modulus	-	AASHTO T 307
Liquid limit and plasticity index	ASTM D 4318	AASHTO T 89, AASHTO T 90
Maximum dry unit weight and optimum moisture content	ASTM D 698	AASHTO T 99
Specific gravity	ASTM D 854	AASHTO T 100
CBR	ASTM 1883	AASHTO T193

California Bearing Ratio (CBR) tests (AASHTO T193 / ASTM 1883) were performed on bulk samples from each site and used to develop a model between M_R and CBR. Tests were performed in both the soaked and unsoaked conditions. The results were used to develop a catalog for pavement subgrades for use by the SCDOT for future pavement design. The geotechnical index properties (G_s, *LL, PI*, w_n, γ_d, w_{opt}, *and* γ_{dmax}) and soil classification (USCS and AASHTO) obtained for the subgrade soils from each borehole (BH) at each site are shown in Table 3.1 of *Supplemental Report: Pavement ME Input Parameter for Subgrade* Material. Similarly, the grain size distribution results and moisture-density curves for samples from each BH are summarized in Appendix E and F, respectively, of *Supplemental Report: Pavement ME Input Parameter for Subgrade Material*.

M_R found per AASHTO T 307 tests performed on Shelby tube samples from each borehole are shown in Table 3.2 of *Supplemental Report: Pavement ME Input Parameter for Subgrade Material*. The three coefficients, k_1, k_2 and k_3, of the generalized constitutive resilient modulus model (NCHRP-1-37A, 2004) that were derived for each test are also presented in Table 3.2 of *Supplemental Report: Pavement ME Input Parameter for Subgrade* Material. These coefficients can be used to calculate the M_R for the desired stress state.

In this study, two approaches were used to calculate M_R. The first approach uses the stress state recommended by NCHRP-285 and is termed "$M_{R(285)}$." The second approach uses the in-situ stress state and is termed "$M_{R(in\text{-}situ)}$." For $M_{R(285)}$, the confining stress (σ_3) is equal to 2 psi and the cyclic stress (deviator) stress (σ_d) is equal to 6 psi per NCHRP-285. For $M_{R(in\text{-}situ)}$, the in-situ stress state was calculated using the pavement layer profile data available from the asphalt coring and soil sampling program. The procedure is documented in Appendix G of *Supplemental Report: Pavement ME Input Parameter for Subgrade Material*. Use of $M_{R(285)}$ is recommended for preliminary designs before the pavement thickness is known. Use of $M_{R(in\text{-}situ)}$ is recommended when the pavement thickness is known as it is a reasonable representation of soil behavior at the in-situ stress state.

The CBR results obtained from tests performed in the soaked (CBR_S) and unsoaked (CBR_U) conditions on specimens from all boreholes are presented in Table 3.6 of *Supplemental Report: Pavement ME Input Parameter for Subgrade Material*. The CBR_U values ranged from 10 to 30 for the coarse-grained soils and 3 to 13 for the fine-grained soils. Similarly, The CBR_S values ranged from 5 to 26 for the coarse-grained soils and 2 to 7 for the fine-grained soils.

The relations developed herein between CBR_S and the CBR_U for coarse-grained and fine-grained soils are as follows:

$$CBR_S\,(\%) = m * CBR_U\,(\%) \tag{4.1}$$

where m = Coefficient of conversion factor. As presented in Section 3.5 of *Supplemental Report: Pavement ME Input Parameter for Subgrade Material*, m was found to be 0.8 for coarse-grained soils, 0.4 for fine-grained soils, and 0.6 for all soils.

For the soils in this study, a relation between M_R found from repeated load triaxial tests and the unsoaked CBR was developed and used to find the coefficient of conversion (*k*-factor) in the relation:

$$M_{R(Lab)}\,(\text{ksi}) = k * CBR_U\,(\%) \qquad (4.2)$$

where *k* is a coefficient of conversion. As detailed in *Supplemental Report: Pavement ME Input Parameter for Subgrade Material*, Table 16 shows the developed *k*-factors that were found for coarse-grained and fine-grained soils for CBR_U as per AASHTO T 193 and the two values of $M_{R(Lab)}$: $M_{R(285)}$ and $M_{R(in\text{-}situ)}$. These *k*-factors can be used to estimate M_R as a Level 2 input in Pavement ME Design.

The SSV for an individual sample was determined from the relation between CBR and SSV presented in the SCDOT Pavement Design Guide (2008) for the Piedmont and Coastal Plain regions of the state of South Carolina. The details are documented in Section 2.5 of *Supplemental Report: Pavement ME Input Parameter for Subgrade Material*. The SSV_U and SSV_S values obtained through correlation to CBR_U and CBR_S values, respectively, are summarized in Table 3.6 of *Supplemental Report: Pavement ME Input Parameter for Subgrade Material*. These values can be used in a catalog of pavement subgrades (by county and geologic region) to aid the SCDOT in future pavement design.

Table 16 Developed k-factors for South Carolina

Model	Soil Type	k-factors
$M_{R(285)}$ (ksi) = k * CBR_U (%)	Coarse-grained	0.63
	Fine-grained	0.16
$M_{R(in\text{-}situ)}$ (ksi) = k * CBR_U (%)	Coarse-grained	0.68
	Fine-grained	0.20

Task 5: Determination of In-Place Asphalt |E*| and Subgrade M_R

The feasibility of determining the in-place |E*| of the asphalt pavement layers and the M_R of the subgrade soil using the falling weight deflectometer (FWD) was investigated in this task. Characterizing the moduli of pavement layers is a crucial step in determining the most cost-effective treatment type, allocating resources, and budgeting for the maintenance and rehabilitation of deteriorating highway infrastructure (Kutay et al. 2011). The need for characterizing pavement properties has become even more significant with the development of mechanistic-empirical pavement design procedures, such as the MEPDG. As an increasing number of Departments of Transportation in the United States and road authorities worldwide are transitioning towards a mechanistic-empirical pavement design approach, researchers have explored methods to evaluate in-situ layer moduli, namely using either non-destructive methods such as FWD, or directly testing the modulus using specimens from cores.

The research team worked with the SCDOT to use their FWD equipment and trained personnel to conduct the tests on the selected pavement sections. The data were then shared with the research team for analysis. FWD data were collected by SCDOT personnel from 2017 to 2018 on 11 asphalt pavement sections in South Carolina (see locations noted in Figure 9 and Table 23). FWD tests were performed at intervals of approximately 200 ft, and the number of FWD tests performed for each section is provided in Table 17. The FWD tests were performed using the Dynatest system (Dynatest Consulting, Inc., 2009). The apparatus (see Figures 10a,b) consists of seven sensors located at seven different offsets (0, 8, 12, 18, 24, 36, and 47 in. from the loading plate schematically shown in Figure 11). Each FWD test was performed by applying an impulse load of 4 different magnitudes (6800, 9000, 12140, and 15700 lbs) and collecting deflection data

(D_0-D_6 in Figure 11) within the deflection basin. Information on the pavement condition (e.g., layer modulus) was extracted from the analysis of the deflection data.

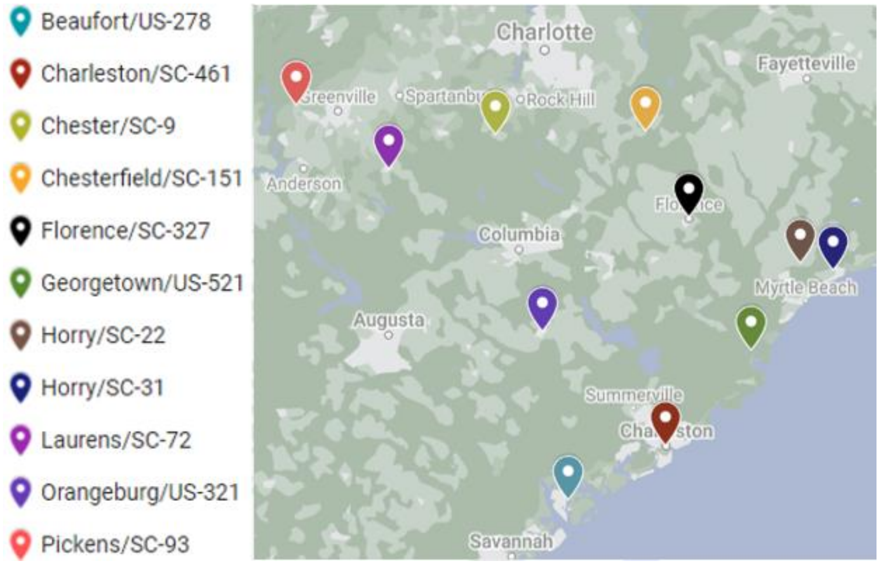


Figure 9 Locations of the FWD Pavement Sections

Figure 10 FWD: (a) Loading Frame and (b) System

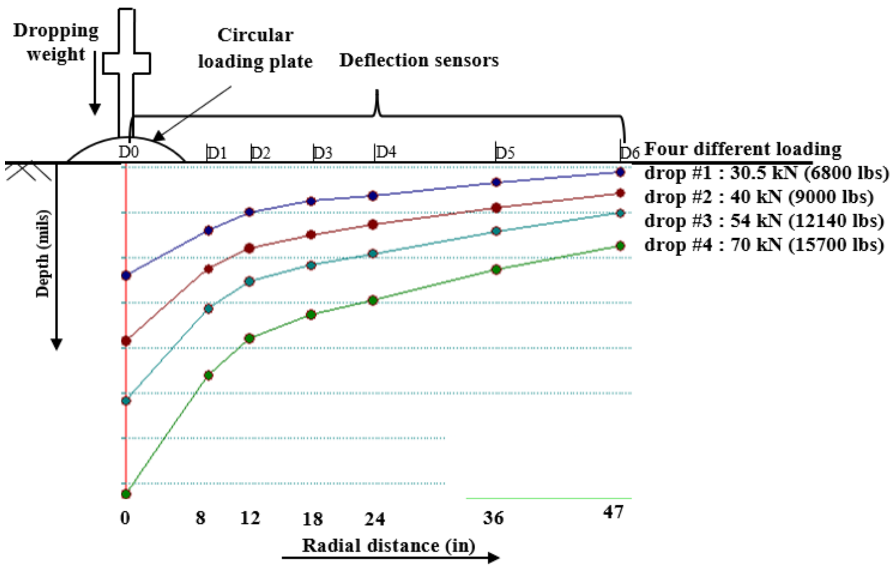


Figure 11 FWD Deflection Basin Profile for Different Loading Conditions

Table 17 Number of FWD Tests on Asphalt Pavement Sections

County/Route	Length of section (miles)	No. of FWD tests @ 200 ft intervals
Beaufort/US-278	1.6	23
Charleston/SC-461	2.5	57
Chester/SC-9	1.9	50
Chesterfield/SC-151	5.4	142
Florence/SC-327	4.9	113
Georgetown/US-521	3.0	80
Horry/SC-22	1.0	25
Horry/SC-31	4.0	157
Laurens/SC-72	6.1	157
Orangeburg/US-321	6.1	155
Pickens/SC-93	1.2	36

Task 5a: Asphalt Dynamic Modulus

The feasibility of determining the in-place dynamic modulus of the asphalt pavement layers was evaluated using two different methodologies: FWD and testing of field cores.

Pavement Layer Modulus Back-calculation from FWD Data

Recent modeling approaches offer the potential to provide accurate layer property values from FWD pavement response data. However, each modeling approach has its limitations and strengths, and there is no one-size-fits-all solution for back-calculation problems (Kutay et al. 2011). Back-calculation is a modeling technique utilized to estimate the moduli of pavement layers by examining the dynamic response (deflection) of the pavement surface when subjected to an impulse load. It proves as a valuable tool in evaluating the structural condition of existing pavements and determining the necessary layer properties for numerical or analytical programs (Han et al. 2022).

FWD data was collected for one of the calibration sections and the layer modulus values were determined using three back-calculation tools: MODULUS, ELMOD, and AASHTOWare. Figure 12 shows a sketch of the pavement structure analyzed in this exercise.

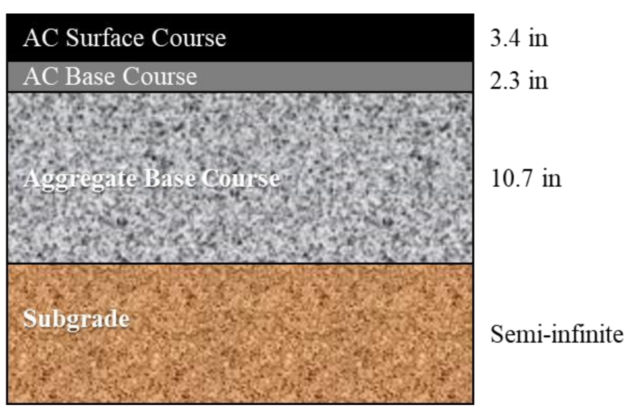


Figure 12 Sketch of Section Pavement Structure based on Core Dimensions

The estimated dynamic modulus values for the two asphalt layers from Figure 12 (AC Surface Course and AC Base Course) are included in Figure 13. ELMOD consistently underestimates the modulus values in both asphalt layers, which can lead to an overestimation of layer thickness during rehabilitation design. Conversely, AASHTOWare tends to provide higher modulus values, potentially overestimating the pavement's structural condition. The graphs show a few abrupt spikes at certain station locations, suggesting the presence of a rigid layer underneath or potential anomalies and measurement errors.

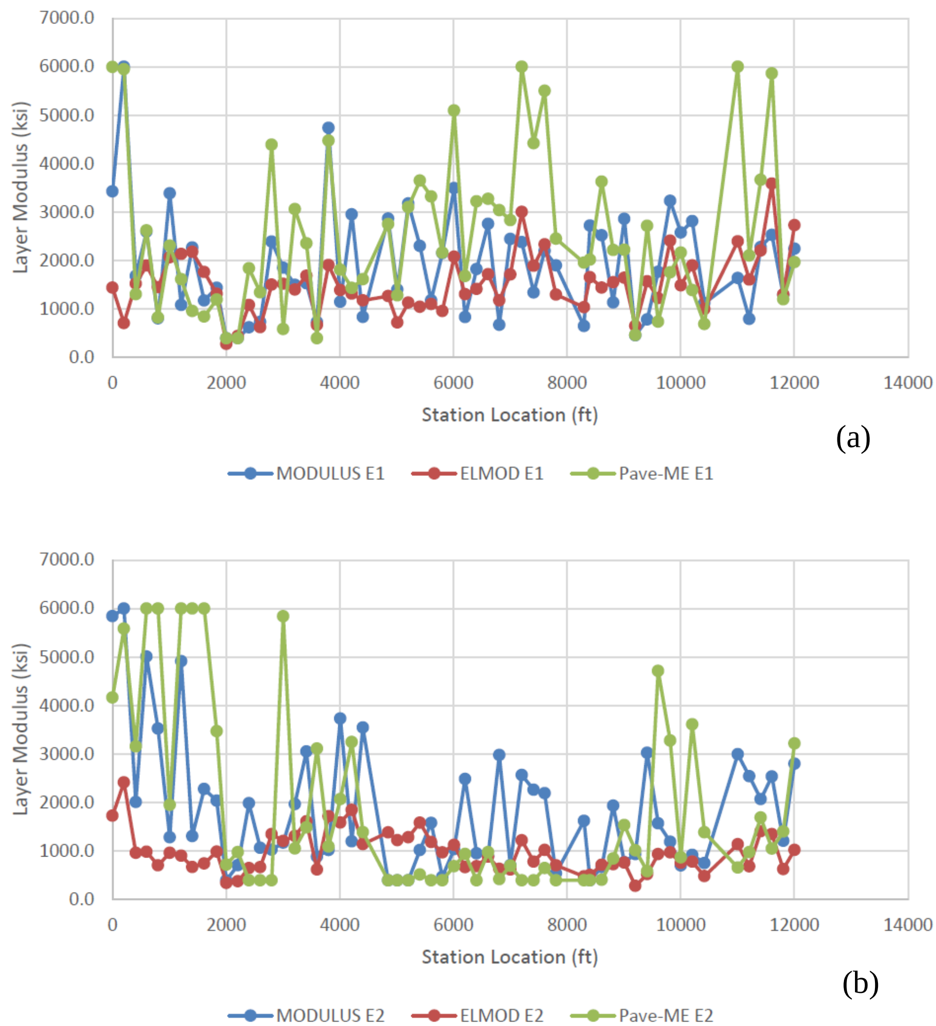


Figure 13 Back-Calculated Layer Modulus from MODULUS, ELMOD, and AASHTOWare for the Asphalt Layers (a) AC Surface Course and (b) AC Base Course

Each of the back-calculation tools used in this study relies on the use of a seed modulus value for each layer to determine the individual layer moduli from the FWD data. The process for selecting appropriate seed moduli values is iterative to the point of convergence. MODULUS requires a range of minimum and maximum seed moduli values, ELMOD only requires seed moduli values, and AASHTOWare necessitates both a range of minimum and maximum values and an initial seed moduli value. In this study it was found that the back-calculated modulus values are highly sensitive to the input seed modulus values, thus no unique solution was achieved.

Based on the work done in this limited study, it is evident that a deeper investigation with more data is required to determine the suitability of using FWD data to accurately estimate the in-place dynamic modulus of individual asphalt layers in a particular pavement structure.

Asphalt Dynamic Modulus Measured with Small-scale Specimens

The standard specimen used to measure dynamic modulus by means of the asphalt mixture performance tester (AMPT) in accordance with AASHTO 378-17 is 100 x 150 mm. Considering that most asphalt pavement layers are less than 150 mm thick, it is not possible to measure the dynamic modulus of as-built asphalt layers using the current procedure. The use of small-scale specimens to measure the dynamic modulus of asphalt layers has been studied by multiple researchers. Multiple studies have suggested that small-scale specimens measuring 38 mm in diameter by 110 mm in height are suitable for measuring the dynamic modulus of asphalt mixtures using the AMPT (Lee et al. 2017; Diefenderfer, et al. 2015).

The thickness of most asphalt layers is less than the height of even small-scale test specimens used in previous research. Therefore, if one wanted to obtain test specimens from pavement cores, it would be necessary to core the specimens in a horizontal orientation instead of vertical. In such cases, it is important to consider the potential effect of anisotropic behavior on the test results. Anisotropy in asphalt mixtures can result from anisotropic particle and void shape, particle orientation distribution, and anisotropic compaction (restraint and force pattern applied during compaction) (Wang et al. 2004).

To evaluate the feasibility of directly measuring the dynamic modulus of an in-place asphalt layer, a study was conducted to compare the dynamic modulus of small-scale specimens cored in both vertical and horizontal direction. The small-scale specimens were 38 mm diameter x 110 mm in height. The dynamic modulus of each test specimen (conventional and small-scale) was measured using the AMPT in accordance with AASHTO T 378-17. The test frequencies chosen were 0.1, 1, 2, 5, 10, 25 Hz and the temperatures were 40, 70, 100 and 130°F (14, 21, 37, 54°C). Figure 14 shows examples of the fabrication process of different specimens.

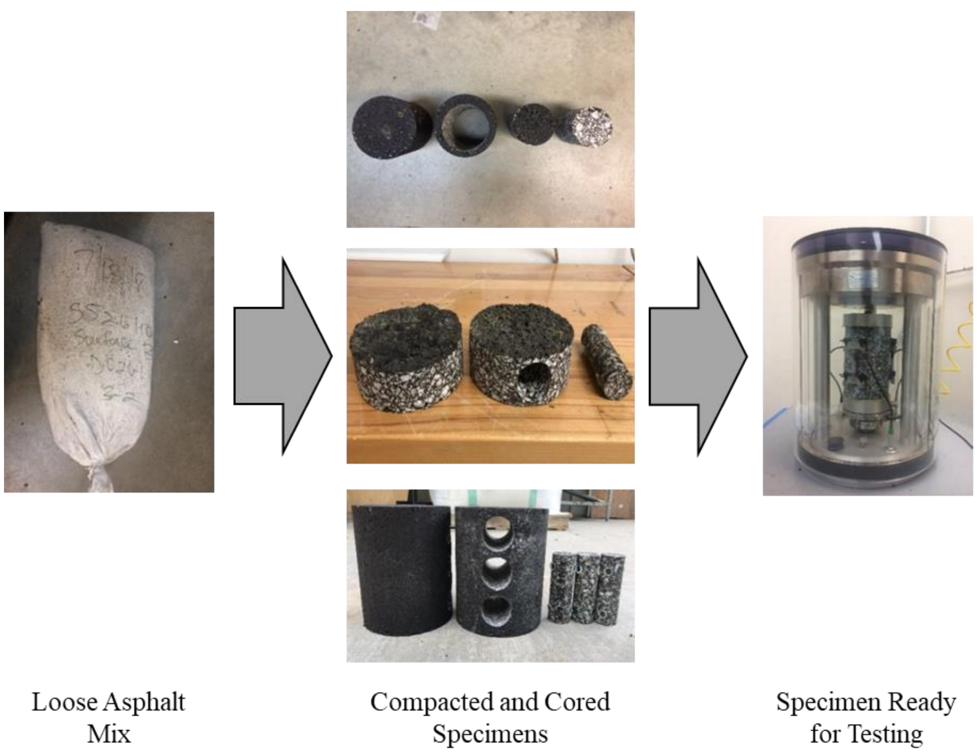


Figure 14 General Process Flow for Preparing Conventional and Small-Scale Dynamic Modulus Test Specimens from Lab and Field Compacted Samples

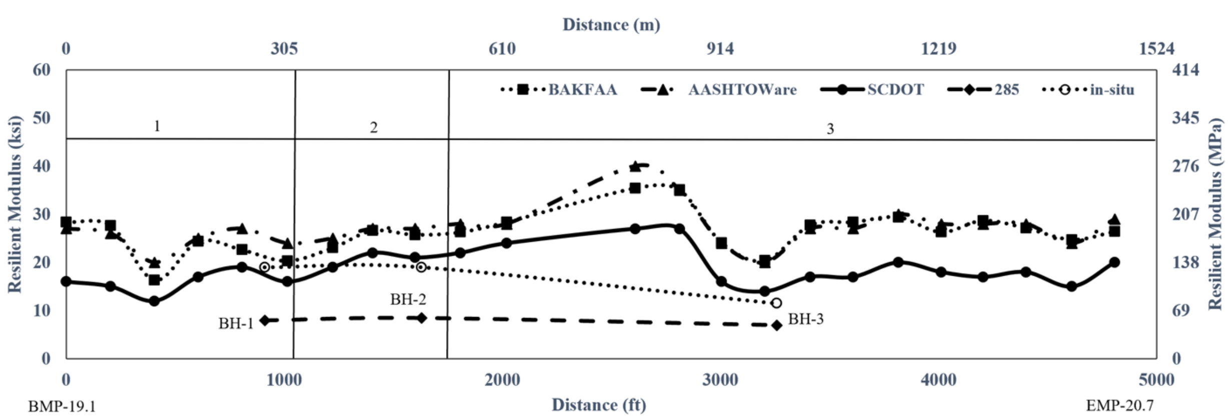
The results of this study are documented in *Supplemental Report: Evaluation of Asphalt Dynamic Modulus Measured With Small-Scale Specimens*. Based on the results of the lab study to determine the feasibility of measuring the dynamic modulus of specimens taken from pavement cores, the following conclusions were drawn.

- The influence of specimen geometry on the dynamic modulus indicated that the small-scale specimen had a slightly lower dynamic modulus than the standard size specimen. The ratio of small-scale to the control specimen ranged from 0.81 to 1.23 with an average of 0.96.
- It is feasible to obtain cores horizontally from individual asphalt pavement layers and determine dynamic modulus using small-scale specimens. The horizontally cored specimens exhibited higher dynamic modulus than the control specimen with a ratio of small-scale horizontal to control specimen ranging from 0.84 to 1.32 with an average of 1.06.
- The vertically cored small-scale specimens were found to have a dynamic modulus value about 12% lower than that of the horizontally cored small-scale specimen. This was due to the density of the horizontally cored specimens compared to the vertically cored specimens.
- It is feasible to directly measure the dynamic modulus of in-place asphalt layers using small-scale specimens taken from pavement cores. In this study, the specimen dimensions were 38 mm in diameter by 110 mm in height. The primary limitation will be the thickness of the individual pavement layer.

Task 5b: Subgrade Modulus

The M_R of the subgrade soil was backcalculated from the FWD data using the SCDOT program (Johnson, 1992). The backcalculation tools EVERCALC in AASHTOWare (2017) and BAKFAA (FAA, 2002) were used for comparison. A discussion of each of these tools is presented in Section 2.4.2 of the *Supplemental Report: Pavement ME Input Parameters for AC Subgrade Material*.

To illustrate the resilient modulus found using the FWD data with three backcalculation tools, profiles along the pavement length for Beaufort/US-278 are shown in Figure 15. The $M_{R(285)}$ (using the NCHRP 285 stress conditions) and the $M_{R(in\text{-}situ)}$ (using in-situ stress conditions) found from the laboratory repeated load triaxial testing are shown for comparison. Similar profiles for all sites are shown in Appendix H of the *Supplemental Report: Pavement ME Input Parameters for AC Subgrade Material.* For Beaufort/US-278, the three backcalculated subgrade $M_{R(FWD)}$ profiles follow a similar pattern along the pavement length; however, the magnitudes are different.



Note: "BAKFAA," "AASHTOWare," and "SCDOT" from backcalculation tools; "285" and "in-situ" are from AASHTO T307 laboratory tests using NCHRP-285 stress conditions and in-situ stress profiles, respectively.

Figure 15 Subgrade Resilient Modulus Profiles for Beaufort/US-278

The average backcalculated subgrade modulus found along the pavement length using BAKFAA and AASHTOWare tools is in close agreement (approximately 4% difference);

whereas, the average subgrade modulus found using the SCDOT program was about 27 to 35 % lower than the other two tools. The lower values found using the SCDOT program are because the SCDOT program uses the minimum surface modulus value corrected for layer stiffness as the subgrade modulus, while BAKFAA and AASHTOWare 2017 use an iterative procedure to find the average backcalculated subgrade modulus for each segment. The $M_{R(FWD)}$, $M_{R(285)}$, and $M_{R(in\text{-}situ)}$ values for each of the 3 segments (e.g., 1, 2, 3) for Beaufort/US-278 are summarized in Table 18. The $M_{R(in\text{-}situ)}$ are in better agreement with the $M_{R(FWD)}$ from the SCDOT backcalculation tool than $M_{R(285)}$ -confirming the M_R found using the in-situ stress state better represents the soil behavior than the NCHRP-285 stress state. Results for all sites are shown in Appendix H of the *Supplemental Report: Pavement ME Input Parameters for AC Subgrade Material*.

Table 18 Summary of Resilient Modulus for Beaufort/US-278

County			Beaufort/US-278			
Pavement Length, ft (miles)			4800 (0.91)			
No. of Roadway segments			3			Avg. M_R (ksi)
			1	2	3	
Segment Boundaries*		BMP	19.1	19.3	19.5	
		EMP	19.3	19.5	20.0	
Resilient Modulus (ksi)	$M_{R(FWD)}$	BAKFAA	24	26	28	26
		AASHTOWare	25	20	28	24
		SCDOT	16	20	19	18
	$M_{R(285)}$		8	8.5	7	8
	$M_{R(in\text{-}situ)}$		19	19	11.5	16.5

*See Table D-2 for segment lengths in Appendix D of the *Supplemental Report: Pavement ME Input Parameter for Subgrade Material*

The results from the repeated load triaxial tests ($M_{R(Lab)}$) and those backcalculated from FWD data ($M_{R(FWD)}$) for each of the 73 boreholes from the 11 sites were used to find the coefficient of conversion (*C*-factor) in the relation:

$$M_{R(Lab)}\,(\text{ksi}) = C * M_{R(FWD)} \tag{5.1}$$

where C is a coefficient of conversion. Table 19 shows the developed C-factors that were found for coarse-grained and fine-grained soils for each of the $M_{R(FWD)}$ found from the three backcalculation tools and the two values of $M_{R(Lab)}$: $M_{R(285)}$ and $M_{R(in\text{-}situ)}$.

Table 19 Developed C-factors for South Carolina

Model	Soil Type	Backcalculation Tools			AASHTO 1993
		SCDOT	AASHTOWare	BAKFAA	
		C-factors			
$M_{R(285)}$ (ksi) = $C * M_{R(FWD)}$	Coarse-grained	0.42	0.30	0.31	0.33
	Fine-grained	0.43	0.35	0.36	
$M_{R(in\text{-}situ)}$ (ksi) = $C * M_{R(FWD)}$	Coarse-grained	0.46	0.33	0.34	
	Fine-grained	0.59	0.47	0.51	

These C-factors can be used to predict M_R as a Level 2 input in Pavement ME Design when there is a limited budget and FWD tests are performed rather than repeated load triaxial tests. The C-factor found from $M_{R(in\text{-}situ)}$ should be used when the pavement thickness is known, and the in-situ stress can be calculated, as it is a reasonable representation of soil behavior at the in-situ stress state. Note that these C-factors are lower than those found in the literature (e.g., 0.645 for Wyoming, 0.55 (coarse-grained), and 0.67 (fine-grained) for Utah) per Ng et al., 2018. The discrepancies may be from differences in FWD equipment, environmental/seasonal field conditions, back-calculation methods, and soil sampling/testing methods, for example, all of which contribute to difficulties in making correlations between laboratory results and field measurements. The AASHTO Road Test (AASHTO, 1993) suggested using an adjustment factor of no more than 0.33.

Task 6: Plan for Special Pavement Validation

To compliment the calibration process and all of the data collection and analysis that it entailed, this task involved identifying concrete and asphalt pavement sections to monitor the in-situ performance over time. These sections were intended to serve as a way to validate the performance predictions based on the MEPDG models.

Task 6a: PCC Special Pavement Validation

Site specific construction data was gathered by the USC team for 3 newly constructed PCC pavement sections: Cherokee/I-85, Lexington/I-20, and Spartanburg/I-85 as summarized in the *Supplemental Report: Selection of Pavement ME Input Parameters for PCC Model Calibration*. Concrete mix was sampled at the time of construction to characterize the material properties as part of Task 4a. These data serve as a baseline for each pavement section and were used in the calibration of the "Rigid New JPCP" distress and performance models in Task 7. The PCC sections were not instrumented, but it is recommended that they be monitored for surface distresses, as in Task 3, each year that they are in service.

Task 6b: AC Special Pavement Validation

Two asphalt pavement sections were identified to be instrumented as part of this study. The first is located on US 123 in Central, SC and the second on Volvo Car Drive, in Ridgeville, SC. The instrumentation consisted of horizontal asphalt strain gauges, earth pressure cells, and moisture probes as illustrated in Figure 16. The asphalt strain gauges were placed on top of the aggregate base course and below the bottom asphalt layer to measure the strain of the bottom of the asphalt under traffic loading. One earth pressure cell was placed on top of the aggregate base course and another on top of the subgrade. This placement was intended to measure the stress at the top of each of these layers under traffic. The strain gauges and earth pressure cells were placed in the outermost wheel path. The moisture probes were placed near the top of the subgrade, then at 6 inch increments below.

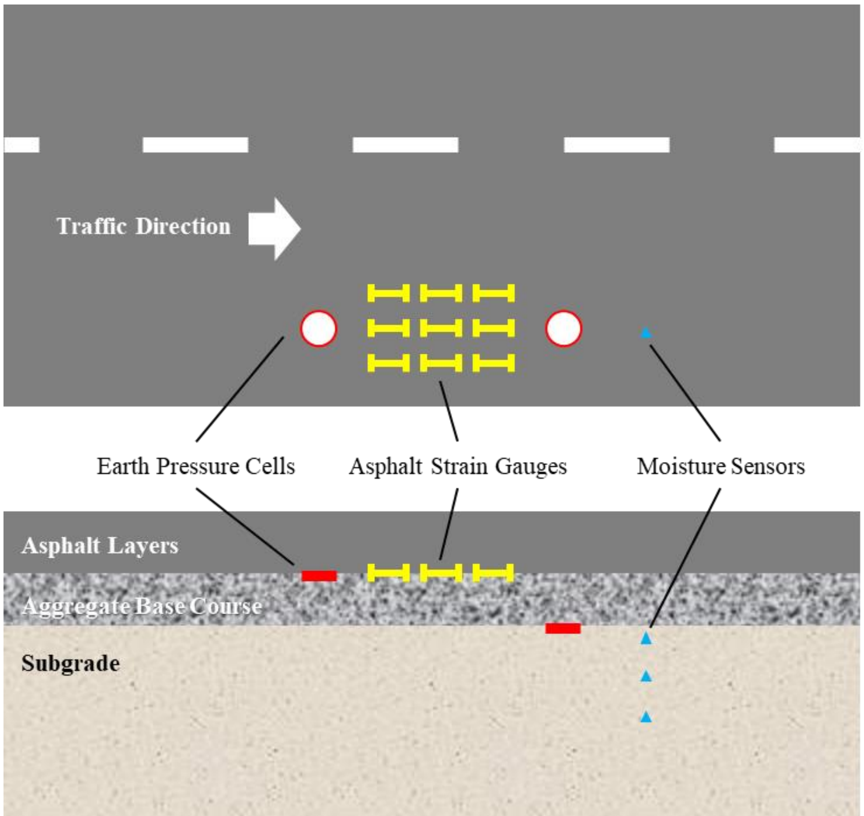


Figure 16 Schematic of Sensor Layout

Unfortunately, challenges with instrumentation function and collecting reliable data at these sites prevented the research team from acquiring any useful data for this study. The challenges included:

- Survivability of Sensors: Some of the sensors installed in the sections did not function after installation. This is not uncommon with this type of installation and is the reason for redundancy in the layout of the instrumentation. *Recommendation: take appropriate precautions to minimize potential damage to sensors.*

- Data Processing: There were issues with processing the collected data to make sense of it. This could have been the result of data acquisition, instrument functionality, or post-processing errors. *Recommendation: ensure that the appropriate data acquisition system(s) is selected for the instrumentation used and work closely with the suppliers' technical support teams to ensure that proper steps are followed to yield usable and reliable data.*
- Remote Data Transmission: There were issues remotely transmitting the data from the data acquisition system on site, this was addressed by manually downloading the data.

Based on lessons learned, a new plan has been developed for another instrumented site following this project. This asphalt pavement section is planned to be located at a new weigh station on I-26 and is anticipated to include three experimental test sections that will be instrumented. Being located at an active weigh station presents a nearly ideal scenario as the truck volume and weights will be readily available without having to install another Weigh-In-Motion station. In addition to live traffic, this configuration will enable the SCDOT to measure the response of these sections under FWD loading without requiring a lane closure on an active roadway.

Task 7: Local Calibration of Distress Models

The local calibration coefficients for the “Rigid New JPCP” and “Flexible New AC” pavement distress and performance models within the Mechanistic-Empirical Pavement Design Guide (MEPDG) were obtained using the AASHTO Online Calibration Assistance Tool, “OCAT” (AASHTO 2020). OCAT was released in 2020 to assist state highway agencies with the local calibration process for AASHTOWare Pavement ME Design. The tool requires the user to upload an AASHTOWare Pavement ME .dgpx file for each local pavement section used in the calibration and a single .csv file containing the pavement distresses and associated pavement ages for all sections combined. The calibration methodology reflects agency-specific design practice to use the error-minimization approach for adjusting model coefficients to improve prediction accuracy. The OCAT automatically compares predicted and measured distress to reduce the bias and lower the standard error of the estimates.

As shown in Figure 17, there are six distinct parts with several steps that a user must complete to perform a calibration using OCAT. These parts include:

- ✓ Part 1: Getting Ready for Calibration
- ✓ Part 2: Distress Data Review
- ✓ Part 3: Initial Verification- Set up Project files and Execute Analyses
- ✓ Part 4: Data Analysis and Interpretations
- ✓ Part 5: Optimization of Coefficient to Eliminate Bias and Minimize Standard Error
- ✓ Part 6: Validation, Accepting the Results

In this study, 11 PCC pavement sections (24 segments) and 11 AC pavement sections (76 segments) were available to perform the calibration. A .dgpx file was created for each of the segments in Part 1, and a .csv file was created for all segments combined in Part 2. In Part 3, the project files were set up, and the analysis was executed. In Part 4, the data analysis and interpretations of the distress prediction results were performed after the initial verification runs in Part 3 were completed, and the data was extracted from the OCAT database. Once the initial verification data was completed and it was determined that local calibration should be performed based on the rejection of the hypothesis tests, the coefficients were optimized to eliminate bias and minimize the standard error of the estimates in Part 5. In Part 6, The OCAT automatically takes 20 percent of sites that were excluded from the optimization process and uses those to validate the calibration results derived from Part 5.

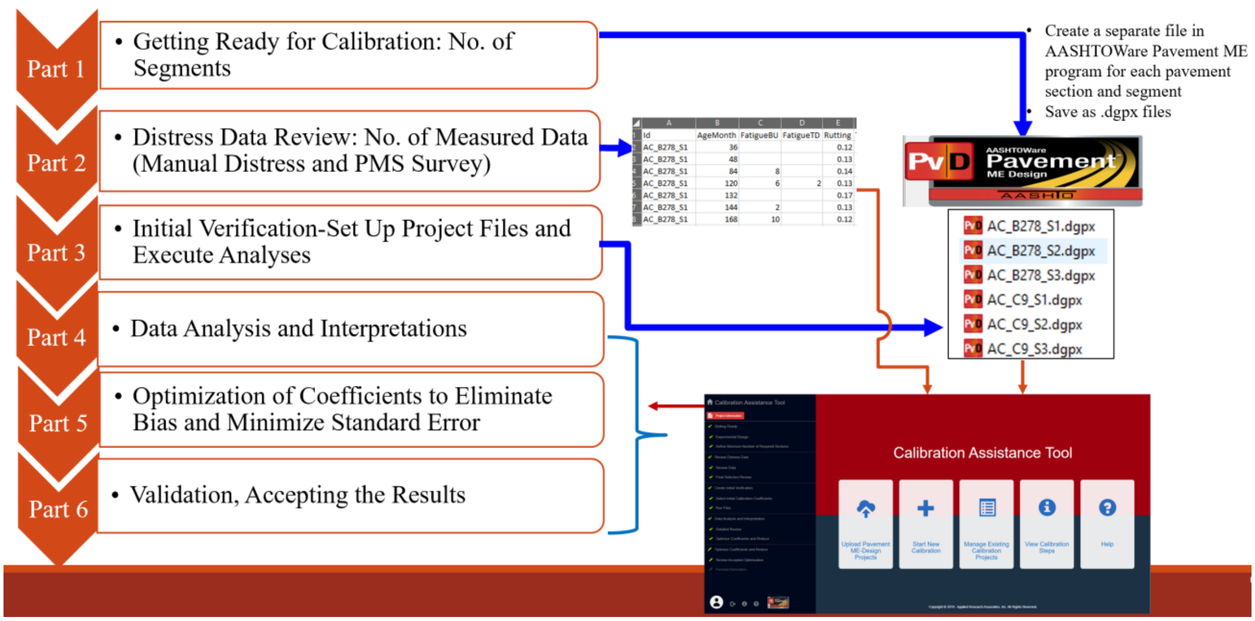


Figure 17 **Schematic of Calibration Process**

The detailed calibration procedures are found in *Supplemental Report: Local Calibration of PCC Distress Models using South Carolina Input Parameters* and *Supplemental Report: Local Calibration of AC Distress Models using South Carolina Input Parameters* for the PCC and AC distress and performance models, respectively.

Task 7a: PCC Distress Models

The local calibration coefficients of the “Rigid New JPCP” pavement distress and performance models within the MEPDG were obtained for South Carolina conditions. In this study, the AASHTOWare Pavement ME Design software was used for design analysis, and the Online Calibration Assistance Tool (OCAT) was used for the calibration. The calibration was performed for the Transverse Cracking, Mean Transverse Joint Faulting, and the IRI models using data compiled for 24 new PCC pavement segments. The required inputs (i.e., pavement structure, materials, traffic, and climate) were compiled into a .dgpx file for each pavement segment. The distress and performance data were compiled as a function of pavement age into a *.csv file. A detailed summary of the input parameters used in the calibration, and their selection process, was documented in the *Supplemental Report: Selection of Pavement ME Input Parameters for PCC Model Calibration*, and the procedures used to perform the calibration were summarized in *Supplemental Report: Local Calibration of PCC Distress Models using South Carolina Input Parameters.*

The model coefficients obtained in this study are summarized in Table 20. The global calibration coefficients and local coefficients from the Phase I study by Gassman and Rahman (2016) are included for comparison. Note that in the Phase I study, Pavement ME Design (v2.2) was used for the design analysis, and Microsoft Excel Solver was used to calibrate each transfer function. As the Pavement ME Design software has evolved, changes have been made to the transfer functions and coefficients.

The findings from this study include:

- For the Transverse Cracking and Mean Transverse Joint Faulting models, the predicted results using the global calibration factors were not in agreement with the measured values, which means that the model does not have a good fit to the data and needs to be optimized using local calibration. After optimization, in which the bias and standard error of the estimates were minimized, the values for bias, SEE, and y-intercept were the same as they were for the global calibration, and the R-squared and Slope were essentially the same, indicating no improvement for both models from the local calibration.

- For the IRI model, as summarized in Table 3.1 of the *Supplemental Report: Local Calibration of PCC Distress Models using South Carolina Input Parameters*, all three hypothesis tests were rejected; the p-values were less than 0.05, indicating a significant difference between the measured and predicted IRI. Thus, local calibration is required to improve the IRI predictions. IRI models were improved by local calibration. One or more hypothesis tests showed a failing result; however, the calibration was accepted because there was a zero bias which was lower than the global model, as suggested by AASHTO (2010). Note that the residual error magnitude ranges from 0 to -245, thus, there is still potential for further improvement of the local calibration for the IRI model. The local calibration coefficients obtained for this model are:
    - IRI: $C_1 = 5.5$, $C_2 = 12.5$, $C_3 = 2.9$ and $C_4 = 40.5$

The main limitation of this study was the small data set available. There were 11 PCC pavement sections (24 segments) available for calibration, each with limited materials data and historical distress data available. Recall that 30 pavement sections are recommended for calibration per MEDPG (AASHTO, 2015). Also, four of the eleven sections have an asphalt layer beneath the PCC layer, which was treated as a base layer, rather than a combination pavement, in order to include these sections in the PCC distress model calibration. Another limitation was the limited cracking and faulting data available. Charleston/I-526 was the only pavement section that had a full set of pavement distress data (cracking, faulting, and IRI). The remaining sections only had annual IRI measurements. Furthermore, the PCC material properties were unknown for most of the sections, thus there was very limited Level 1 materials data available for the calibration. Default (Level 3) or regional (Level 2) values were used.

Based on this study, it is recommended to continue to use the global calibration factors for the Transverse Cracking and Mean Transverse Joint Faulting models until additional pavement distress data is available to improve the calibration; whereas, the local calibration coefficients for the IRI model developed herein may be used with the understanding that they should be updated and improved as more data becomes available. In the future, there should be more pavement sections added to the calibration along with additional, correctly formatted pavement distress data collected for the 11 pavement sections used here.

Table 20 Local Calibration Coefficients for "Rigid New JPCP"

Distress Model	Calibration Coefficient	Global Calibration Coefficient	Phase I Local Calibration Coefficient	Phase II Local Calibration Coefficient
Transverse Cracking	C_1	2	1.25[1]	2.5
	C_2	1.22	1.22[1]	19.5
	C_4	0.52	*	0.0001
	C_5	-2.17	*	-0.0001
Mean Transverse Joint Faulting	C_1	0.595	N/A**	1.3
	C_2	1.636	N/A**	1.61
	C_3	0.00217	N/A**	0.003
	C_4	0.00444	N/A**	0.001
	C_5	250	N/A**	280
	C_6	0.47	N/A**	0.47
	C_7	7.3	N/A**	9.5
	C_8	400	N/A**	500
IRI	C_1	0.8203	N/A**	5.5
	C_2	0.4417	N/A**	12.5
	C_3	1.4929	N/A**	2.9
	C_4	25.24	N/A**	40.5

[1]Gassman and Rahman (2016); *Coefficient added in MEPDG calibration versions post Phase I;
**Distress model not locally calibrated as part of Phase I for rigid pavement.

Task 7b: AC Distress Models

The local calibration coefficients for the "Flexible New AC" pavement distress and performance models within the MEPDG were obtained for South Carolina conditions. In this study, the AASHTOWare Pavement ME Design was used for design analysis, and the OCAT was used for the calibration. The calibration was performed for the Bottom-up Fatigue Cracking, Total Rutting, IRI, and Top-down Fatigue Cracking models using data compiled for 66 new flexible asphalt concrete pavement segments (10 of the total 76 segments had semi rigid bases). The required inputs (i.e., pavement structure, materials, traffic, and climate) were compiled into a .dgpx file for each pavement segment. The distress and performance data were compiled as a function of pavement age into a *.csv file. A detailed summary of the input parameters used in the calibration, and their selection process, was documented in the *Supplement Report: Local Calibration of AC Distress Models using South Carolina Input Parameters* and the procedures used to perform the calibration were summarized in *Supplemental Report: Local Calibration of AC Distress Models using South Carolina Input Parameters.*

The model coefficients obtained in this study are summarized in Table 21. The global calibration coefficients and local coefficients from the Phase I study by Gassman and Rahman (2016) are included for comparison. Note that in the Phase I study, Pavement ME Design (v2.2) was used for the design analysis, and Microsoft Excel Solver was used to calibrate each transfer function. As the Pavement ME Design software has evolved, changes have been made to the transfer functions and coefficients.

The findings from this study include:

- The Bottom-up Fatigue Cracking, Total Rutting, and IRI models were improved by local calibration. For each of these models, one or more hypothesis tests showed a failing result; however, the calibration was accepted because there was a close to zero bias with a SEE lower than the global model, as suggested by AASHTO (2010). The local calibration coefficients obtained for these models are:

  - Bottom-up Fatigue Cracking: $\beta_{f2} = 1.5$, $\beta_{f3} = 0.85$, $C_1 = 1.75$, $C_{2 < 5 \text{ in.}} = 2.75$, and $C_{2 > 12 \text{ in.}} = 3.5$
  - Rutting: $\beta_{r1} = 0.2$, $\beta_{r2} = 0.3$, $\beta_{r3} = 1$, $\beta_{s1}= 1.2$, and $\beta_{sg1}= 1.1$
  - IRI: $C_1 = 20$, $C_2 = 0.35$, $C_3 = 0.001$ and $C_4 = 0.011$

- Calibration of the Top-down Fatigue Cracking model was unsatisfactory due to variability in the measured data. The predicted top-down fatigue cracking variation is statistically different from the measured values and is not improved through multiple calibration trials.

Table 21 Local Calibration Coefficients for "Flexible New AC" models

Distress Model	Factor Name	Calibration Coefficient			
		Global[1]	Phase I[2]	Global[4]	Phase II[5]
Bottom-up Fatigue Cracking	β_{f1}: <5 in.			0.02054	0
	β_{f1}: <5 in.~<12 in.			0	NA
	β_{f1}: >12 in.			0.001032	0
	β_{f2}			1.38	1.5
	β_{f3}			0.88	0.85
	C_1	1		1.38	1.75
	C_2: <5 in.			2.1585	2.75
	C_2: <5 in.~<12 in.	1		0	0
	C_2: >12 in.			3.9666	3.5
Total Rutting	β_{r1}	1	0.240	0.4	0.2
	β_{r2}	1	1	0.52	0.3
	β_{r3}	1	1	1.36	1
	β_{s1}	1	2.979	1	1.2
	β_{sg1}	1	0.393	1	1.1
IRI	C_1	40	NA[3]	40	20
	C_2	0.4		0.4	0.35
	C_3	0.008		0.008	0.001
	C_4	0.015		0.015	0.011
Top-down Fatigue Cracking	C_1	7.00	0.2	2.5219	2
	C_2	3.5	0.1	0.8069	3

[1]Coefficients per AASHTOWare software (v2.2), [2]Gassman and Rahman (2016) using Microsoft Excel Solver, [3]Not calibrated, [4]AASHTOWare Pavement ME Design software (v2.6.2.2), [5]found using OCAT (AASHTO 2020).

The main limitations from the study include:

- Measured distress and IRI data for this study were collected from the SCDOT PMS database. Most PMS data, especially bottom-up and top-down fatigue cracking, do not directly support the MEPDG.
- A manual distress survey was completed once for each pavement segment during this study. High-quality cracking data collected yearly would benefit the calibration of the fatigue cracking models (Fatigue Cracking: Bottom-up and Fatigue Cracking: Top-down).
- Level 1 data was not available for all inputs (e.g., Aggregate Gradation, Poisson's Ratio, Graded Aggregate Base, Stabilized Aggregate Base, Cement Modified Base, Sand Layer,

Macadam, Coefficient of Lateral Earth Pressure, Saturated Hydraulic Conductivity, etc.). Lack of level 1 inputs may have diminished calibration accuracy.

Based on this study, the locally calibrated Fatigue Cracking: Bottom-up, Rutting, and IRI models for South Carolina are considered reasonable and recommended for use in the "Flexible New AC" design. The SCDOT should maintain the global coefficients for the Fatigue Cracking: Top-down model since reliable local calibration coefficients were not obtained. Improvement for all models is still possible by collecting high quality distress and performance data (especially for the cracking models) through field (e.g., LCMS) and forensic investigations (differentiate between bottom-up and top-down cracking) and continuing to collect distress data on an annual basis.

Task 8: Develop a Plan for WIM Clusters

A Pay for Data request was developed in this task that resulted in the installation of 19 Weigh-In-Motion (WIM) stations at pavement sites across the state of South Carolina. The locations are shown in Figure 18. Tables 22 and 23 summarize the pavement sections, WIM site description and location, and the first data collection date for the WIM stations located at AC and PCC pavement sections, respectively. These WIM stations provide site-specific (Level 1) traffic data required for the MEPDG.

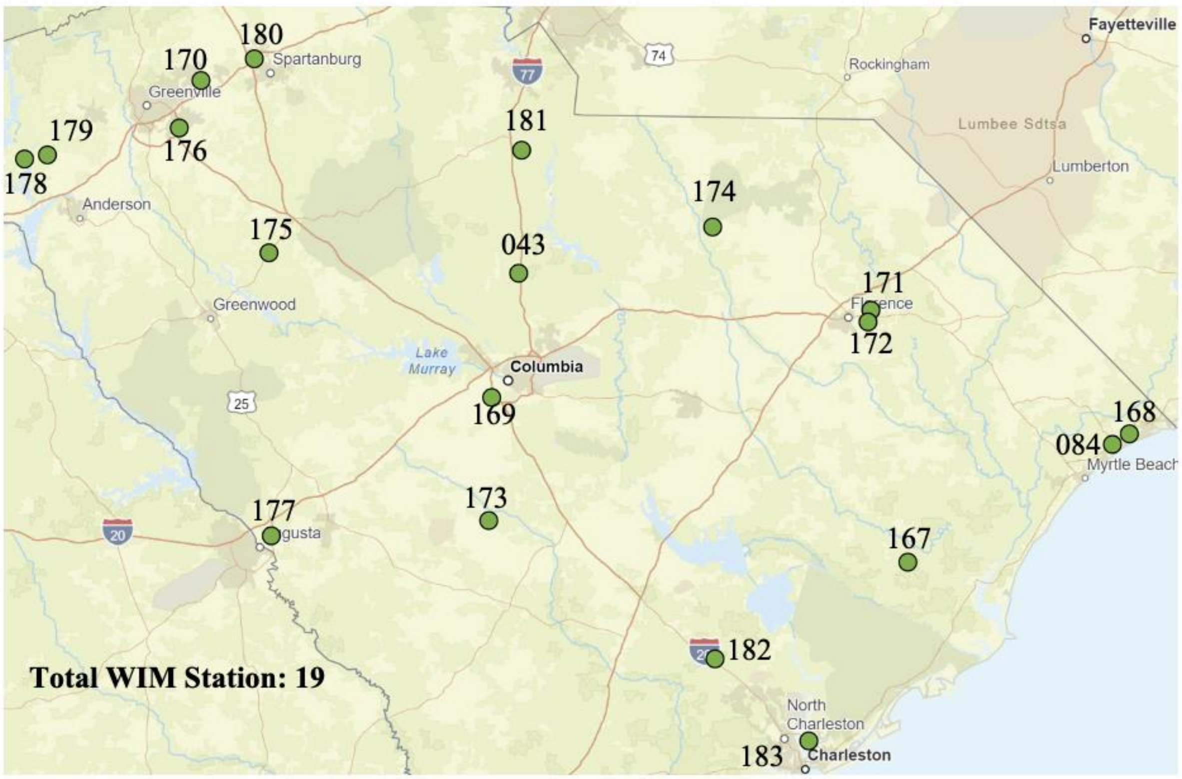


Figure 18 Locations of WIM Stations
(https://scdot.drakewell.com/multinodemap.asp?node=scdot_wim)

Table 22 WIM Stations Installed at AC Pavement Sections

County/Road ID	WIM Site Description	Site Name	First Date of Data Collection	Number of Lanes in each Direction
Horry /SC-22[1]	Between SC-31 & Intracoastal Waterway	0084	May 9, 2020	3
Georgetown/ US-521[1]	Andrews Bypass	167	September 2, 2019	2
Horry /SC-31[1,2]	Carolina Bays Parkway	168	August 30, 2019	NB: 3 SB: 3
Florence /SC-327[1]	North Williston Rd	171	December 6, 2019	NB: 2 SB: 2
Florence /US-301	Freedom Blvd	172	November 22, 2019	1
Orangeburg /US 321[1]	Savannah HWY	173	March 4, 2020	2
Chesterfield /SC-151[1]	North of US-1	174	March 11, 2020	NB: 2 SB: 2
Laurens /SC-72[1]	Between S-49 & S-953	175	March 12, 2020	WB: 2 EB: 2
Greenville /I-385	Between US-1 & SC-126	176	July 2, 2020	NB: 3 SB: 3
Oconee/ S-4	US 76 (Clemson Blvd.) to County Line (Pickens)	178	May 7, 2020	WB: 2 EB: 2
Pickens /SC-93[3]	Between S-30 & S-18	179	May 5, 2020	NB: 2 SB: 2
Chester /SC-9[1]	Lancaster Hwy	181	June 2, 2020	NB: 2 SB: 2
Berkeley /S-1514	R-7901 1E1 & R-7902 2O1 (I-26 Ramps) to S-309	182	June 1, 2020	WB: 2 EB: 3

[1]Used in the calibration of the AC distress models; [2]Data used for Beaufort/US 278; [3]Data used for Charleston/SC 461; NB = North Bound; SB = South Bound, WB = West Bound, EB = East Bound

Table 23 WIM Stations Installed at PCC Pavement Sections

County/Road ID	WIM Site Description	Site Name	First Date of Data Collection	Number of Lanes in each Direction
Fairfield /I-77[1]	Between SC-34 & S-41	0043	April 8, 2020	NB: 2 SB: 2
Lexington /S-378[1,2]	Columbia Airport Expressway West Columbia, Lexington	169	October 2, 2019	WB: 2 EB: 2
Spartanburg /SC-80[1,3]	J Verne Smith Parkway	170	September 19, 2019	WB: 3 EB: 3
Aiken /I-520[1,4]	Between US-1 & SC-126	177	April 10, 2020	NB: 2 SB: 2
Spartanburg /I-85[1,5]	Between I-26 & US-176	180	May 14, 2020	NB: 3 SB: 3
Berkeley /I-526	Between S-33 & S-1520	183	June 25, 2020	WB: 2 EB: 3

[1]Used in the calibration of the PCC distress models; [2]Data used for Charleston/S-97; [3]Data used for Berkeley/I-526; [4]Data used for Charleston/I-526; [5]Data used for Cherokee/I-85 and Lexington/I-20; NB = North Bound; SB = South Bound, WB = West Bound, EB = East Bound

For each WIM station, data for the hourly traffic distributions, truck class distribution, monthly traffic distributions, and axle per truck are collected and stored on Drakewell Ltd.'s C2-Cloud Traffic Data website (URL:https://scdot.drakewell.com/) and formatted for direct input into the Pavement ME software. Inputs include the hourly distribution factor (HDF), the monthly adjustment factor (MAF), the vehicle class distribution (VCD), and the axle load factor (ALF) for each axle type (single, tandem, tridem and quad). The raw data for the single, tandem, tridem, and quadrem load distributions are also stored on the website. Tables summarizing the HDF, MAF, VCD, and ALF for each pavement section that were used in the calibration of the AC distress models are summarized in Appendix E of the *Supplemental Report: Selection of Pavement ME Input Parameters for AC Model Calibration* and in Appendix E of the *Supplemental Report: Selection of Pavement ME Input Parameters for PCC Model Calibration* for the PCC distress models.

Each WIM station provides data compiled into a table for each type of axle distribution that can be imported directly into the Pavement ME Design file for each pavement section. For calibration of the distress models in Pavement ME Design, the input for pavement sections with a WIM station was considered Level 1. For pavement sections that did not have a WIM station, WIM data from a nearby station with similar AADTT/AADT was selected and considered Level 2. See Section 2.6 (Table 2.30) in *Supplemental Report: Selection of Pavement ME Input Parameters for AC Model Calibration* and Section 3.6.2 (Table 3.44) in *Supplemental Report: Selection of Pavement ME Input Parameters for PCC Model Calibration* for further discussion.

Task 9: AC Pavement Design Catalog

The goal of this task was to develop an asphalt pavement thickness design catalog for interstates and other high-volume routes in South Carolina based on the AASHTOWare Pavement ME Design using global calibration coefficients. The thickness design catalog was developed based on a sensitivity analysis of the pavement design to multiple relevant variables. Conducting an extensive sensitivity analysis is beneficial to understand the relative sensitivity of models used in MEPDG to the available data related to the local traffic, climate, and materials. Additionally, such an analysis can help the designers identify the inputs having the most effect on pavement performance and design (Li et al., 2011). The scope of this study focused on bottom-up fatigue cracking, which, in South Carolina, is considered as a deep structural distress. While the MEPDG accounts for other distresses including longitudinal cracking, fatigue cracking, rutting, and thermal cracking, bottom-up cracking is critical to the perpetual pavement design philosophy typically followed by the SCDOT when designing high-volume roadways such as interstates.

In this study, sensitivity analyses were conducted to determine a design asphalt thickness for each combination of variables included in Table 24 based on the bottom-up fatigue cracking results (see *Supplemental Report: Development of an Asphalt Pavement Design Catalog for High-Volume Roads in South Carolina* for more details). Pavement ME Design was used to estimate the distress values for a given asphalt thickness, as illustrated in the example in Figure 19. As shown in Figure 20, the design asphalt thickness was determined as the thickness at which the bottom-up fatigue cracking first reaches 2% lane area. While the threshold value recommended for use in judging the acceptability of the trial design for bottom-up fatigue cracking is 10% of the lane area for interstate routes (AASHTO 2020), the thickness corresponding to 2% lane area was considered for this analysis based on SCDOT experience and practice. Also, in the calibration sensitivity analysis conducted by Rahman and Gassman (2018), using the global coefficient underestimated the value of bottom-up fatigue cracking with 50% reliability, hence a conservative threshold value of 2% lane area for 95% reliability was chosen. Additionally, this approximately corresponds to the thickness at which the pavement is no longer sensitive to distress when using the global calibration factors. For example, increasing an extra inch of asphalt thickness from 11.5 to 12.5 inches does not significantly change the value of fatigue cracking in the example shown in Figure 19.

Table 24 MEPDG Sensitivity Analysis Variables

Variable	Values
AADTT (two-way)	6000 – 30000 (increments of 4000)
Subgrade Type	A-2-4 A-7-6
Subgrade Resilient Modulus (M_R)	6 ksi 10 ksi 14 ksi
Aggregate Base Thickness (M_R = 18 ksi)	0 in 8 in
Asphalt Mix Type	Surface A, B, and C Intermediate B and C Base A
Climate Station	Abbeville, SC Hamer, SC Greenville, SC Goose Creek, SC Lexington, SC

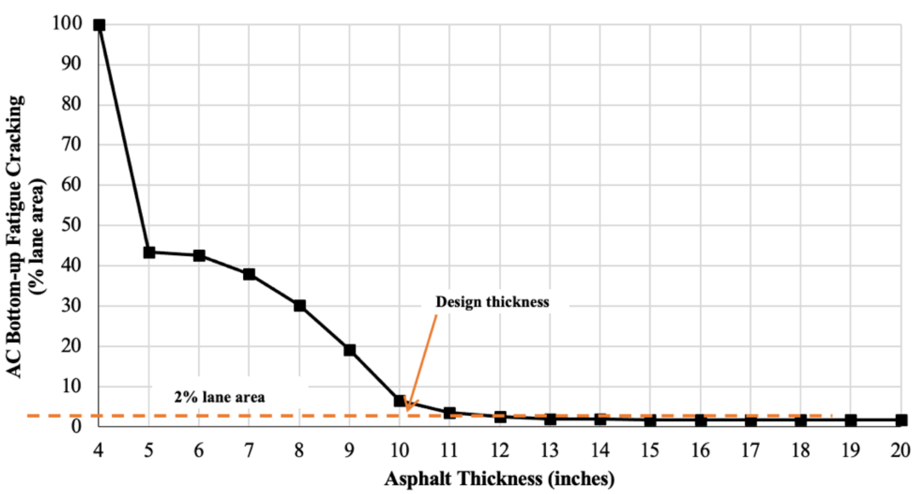


Figure 19 Sensitivity Analysis Example

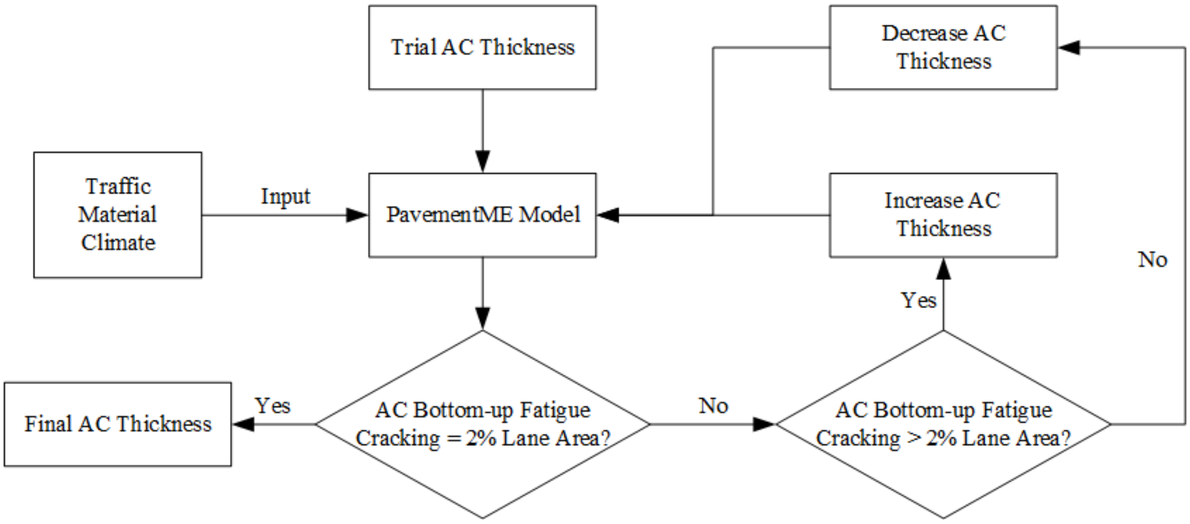


Figure 20 Flowchart Showing the Methodology used to Determine the Pavement Thicknesses in Tables 25 and 26 based on the Sensitivity Analysis

Additional details and results of the sensitivity analysis are included in *Supplemental Report: Development of an Asphalt Pavement Design Catalog for High-Volume Roads in South Carolina*. The results of the sensitivity analysis informed the following findings and the development of the thickness design catalog contained in Tables 25 and 26.

- When using the MEPDG to design pavements, it is sufficient to have a single generic input for surface asphalt mixes, for intermediate asphalt mixes, and for base asphalt mixes.
- The analysis of the effect of climate station on the pavement thickness indicated that the state can be divided into two main climate regions. The climate stations including and surrounding the Abbeville station generally resulted in asphalt thicknesses that were approximately 2 in thicker than other stations representing the majority of the state. Therefore, the design catalog was developed for two climate regions as shown in Tables 25 and 26. The counties in each climate region are defined in Tables 25 and 26.

- The analysis evaluated two typical types of subgrade found in South Carolina and the results showed that the type of subgrade had a significant influence on the pavement design. Therefore, the design tables were developed for two representative subgrades that align with current SCDOT regional soil designations: Piedmont Region Soil (representative of the A-7-6 subgrade) and Coastal Plains Region soil (representative of the A-2-4 soil) (Robbins et al., 2014).
- In addition to the soil type, the subgrade resilient modulus also had an effect on the pavement design. The stronger the soil (higher the resilient modulus), the thinner the required pavement section. This factor is important to consider when designing a pavement as strengthening the subgrade could potentially be more cost-effective than adding more asphalt thickness.
- The addition of an 8 in. thick layer of a graded aggregate base (GAB) resulted in an approximately 1-2 in. thinner asphalt layer when all other factors remained the same. This was lower than expected as other design methodologies, such as the SCDOT method that equates 8 in. of GAB to approximately 3-4 in. of asphalt base, indicating that the MEPDG does not give as much credit to GAB as other methods.
- The traffic volume followed the expected trend of increasing pavement thickness with AADTT. The sensitivity of the pavement to traffic, however, decreased as the two-way AADTT increased from 6,000 to 30,000.
- The terminal IRI, permanent deformation (total and AC only), AC thermal cracking, AC top down cracking values corresponding to AC bottom-up fatigue cracking of 2% lane area are well within the threshold values for the input range and parameters used in the study.
- The thicknesses in this catalog differed from the current SCDOT Pavement Design Guide depending on the design inputs, especially the soil type and the presence of GAB. More detail

about this comparison can be found in *Supplemental Report: Development of an Asphalt Pavement Design Catalog for High-Volume Roads in South Carolina*.

Table 25 Thickness Design Table for Climate Region C1

Climate Region C1 **Climate Stations: Abbeville, Anderson, Blackstone, Clinton, Kershaw, Saluda**

AADTT		6000			10000			14000			18000			22000			26000			30000		
A-2-4 Subgrade Resilient Modulus (ksi)		6	10	14	6	10	14	6	10	14	6	10	14	6	10	14	6	10	14	6	10	14
Type 1	Asphalt Layer (in)	15.5	14.0	13.6	17.0	15.9	14.8	18.3	16.7	15.8	19.2	17.1	16.5	20.0	18.0	17.0	20.7	18.8	17.7	21.1	19.2	18.0
	Base Layer (in)	8	8	8	8	8	8	8	8	8	8	8	8	8	8	8	8	8	8	8	8	8
	Subgrade																					
Type 2	Asphalt Layer (in)	17.2	15.4	14.3	18.9	16.9	15.7	20.0	17.9	16.7	21.0	18.9	17.1	21.9	19.7	18.1	22.2	20.5	18.8	23.1	20.8	19.2
	Subgrade																					

AADTT		6000			10000			14000			18000			22000			26000			30000		
A-7-6 Subgrade Resilient Modulus (ksi)		6	10	14	6	10	14	6	10	14	6	10	14	6	10	14	6	10	14	6	10	14
Type 1	Asphalt Layer (in)	16.9	15.6	14.8	18.8	17.2	16.2	20.0	18.4	17.3	21	19.3	18.4	21.9	19.9	18.9	22.2	20.7	19.5	23.1	21.2	20.0
	Base Layer (in)	8	8	8	8	8	8	8	8	8	8	8	8	8	8	8	8	8	8	8	8	8
	Subgrade																					
Type 2	Asphalt Layer (in)	18.4	17.0	15.8	20.4	18.5	17.3	21.6	19.7	18.4	22.9	20.6	19.3	23.5	21.3	19.9	24	21.9	20.6	24.2	22.5	
	Subgrade																					

Table 26 Thickness Design Table for Climate Region

Climate Region C2

Climate Stations: Beaufort, Beech Island, Elloree, Fairfax, Georgetown, Goose Creek, Greenville, Hamer, Longs, Lynchburg, Marion, Patrick, Rembert, Spartanburg, Springfield, St Stephen, Walterboro, York

AADTT		6000			10000			14000			18000			22000			26000			30000		
A-2-4 Subgrade Resilient Modulus (ksi)		6	10	14	6	10	14	6	10	14	6	10	14	6	10	14	6	10	14	6	10	14
Type 1	Asphalt Layer (in)	13.5	12.4	11.1	14.9	13.7	12.9	15.9	14.6	13.8	16.7	15.2	14.5	17.4	15.9	14.9	17.9	16.4	15.5	18.3	16.9	15.9
	Base Layer (in)	8	8	8	8	8	8	8	8	8	8	8	8	8	8	8	8	8	8	8	8	8
	Subgrade																					
Type 2	Asphalt Layer (in)	14.6	13.0	12.3	16.0	14.5	13.6	17.0	15.5	14.4	17.9	16.1	15	18.6	16.8	15.7	19.0	17.3	16.1	19.6	17.8	16.7
	Subgrade																					

AADTT		6000			10000			14000			18000			22000			26000			30000		
A-7-6 Subgrade Resilient Modulus (ksi)		6	10	14	6	10	14	6	10	14	6	10	14	6	10	14	6	10	14	6	10	14
Type 1	Asphalt Layer (in)	14.7	13.6	12.8	16.2	14.9	14.0	17.3	15.9	15.0	18.0	16.7	15.8	18.8	17.4	16.4	19.4	17.9	16.9	19.9	18.4	17.4
	Base Layer (in)	8	8	8	8	8	8	8	8	8	8	8	8	8	8	8	8	8	8	8	8	8
	Subgrade																					
Type 2	Asphalt Layer (in)	15.7	14.3	13.4	17.0	15.7	14.8	18.1	16.8	15.7	18.9	17.5	16.5	19.7	18.0	17.0	20.3	18.7	17.6	20.8	19.0	18.0
	Subgrade																					

Summary and Conclusions

A summary of the findings for the research activities in the Phase II study to calibrate the distress and performance models of the AASHTO Mechanistic-Empirical Pavement Design Guide (MEPDG) to South Carolina conditions are presented below. These include findings for: 1) the collection of high quality and high priority materials data (Task 1, Task 2, Task 4, and Task 5), 2) the collection of pavement distress and performance data (Task 3), 3) the installation of Weigh-In-Motion stations (Task 8), 4) the development of an AC pavement thickness design catalog (Task 9), and 5) the calibration of the "Rigid New JPCP" and "Flexible New AC" distress and performance models to South Carolina conditions (Task 7).

1) Collection of High Priority Materials Data

Extensive laboratory and field studies were performed to collect high priority, high quality materials data to support the calibration of the distress and performance models of the MEPDG to South Carolina conditions. A review of historic pavement design files produced 11 AC pavement sections and 11 PCC pavement sections for this study. These sections were further subdivided into smaller segments based on FWD testing and manual distress surveys, resulting in a total of 76 AC pavement segments (66 for "Flexible New AC" models and 10 for "Flexible New Semi-Rigid" models) and 24 PCC pavement segments (for "Rigid New JPCP" models) available for calibration. Materials data was collected for each segment and used to establish Level 1, Level 2, or Level 3 inputs. For AC pavements, six different asphalt mix types from multiple asphalt plants across the state were sampled and characterized. For PCC pavements, concrete mix was sampled from 3 new pavements at the time of construction. The resilient modulus of the subgrade soil was obtained from repeated load triaxial testing on Shelby tube samples collected from beneath each of the pavement segments. Relations between the laboratory-derived resilient modulus and Falling

Weight Deflectometer data, CBR tests, and SSV, were developed in this study to aid the SCDOT in future pavement design.

2) Pavement Distress and Performance Data

Pavement distress and performance data were collected and assessed to support the overall calibration of the MEPDG in South Carolina. Two sources of data were available: manual distress surveys performed in this study and historical data retrieved from the SCDOT Pavement Management System (PMS). Manual distress surveys were performed at locations coincident with FWD test locations along eleven AC pavement sections and four PCC pavement sections. For asphalt pavements, measurements of bottom-up fatigue cracking, top-down fatigue cracking, transverse cracking, and rutting were collected. For PCC pavements, mid-slab cracking and joint faulting measurements were collected. In addition to collecting visual surface distress data, two 4 in. diameter pavement cores were taken from each segment. Tube and bulk samples of subgrade soil were collected from beneath the pavement core. The SCDOT PMS provided historic IRI, rutting, and cracking data, although the cracking data was not originally quantified nor reported in the format required by the MEPDG. Data from both the manual distress surveys and the historic PMS data were used as Level 1 input compiled by distress and age for the calibration of the MEPDG distress and performance models.

3) Weigh-In-Motion Stations

A total of 19 Weigh-In-Motion (WIM) stations were installed at pavement sites across the state of South Carolina. Each WIM station provides site-specific (Level 1) traffic data required for the MEPDG: the hourly distribution factor (HDF), the monthly adjustment factor (MAF), the vehicle class distribution (VCD), and the axle load factor (ALF) for each axle type (single, tandem, tridem and quad). The data is reported in a format that can be directly imported into the Pavement ME Design file for each pavement section.

4) AC Pavement Thickness Design Catalog

The results of a sensitivity analysis using global calibration factors informed the following findings and the development of an AC pavement design catalog for high-volume roads in South Carolina:

- When using the MEPDG to design pavements, it is sufficient to have a single generic input for surface asphalt mixes, intermediate asphalt mixes, and base asphalt mixes for the respective layers in a pavement design.
- The state was divided into two climate regions based on differences in pavement thicknesses resulting from designs using different climate stations across the state.
- The type of subgrade had a significant influence on the pavement design; therefore, the design tables were developed for two representative subgrades that align with current SCDOT regional soil designations: Piedmont Region Soil (representative of the A-7-6 subgrade) and Coastal Plains Region soil (representative of the A-2-4 soil).
- In addition to the soil type, the subgrade resilient modulus also had an effect on the pavement design. The stronger the soil (higher the resilient modulus), the thinner the required pavement section. This factor is important to consider when designing a pavement as strengthening the subgrade could potentially be more cost-effective than adding more asphalt thickness.
- The addition of an 8 in. thick layer of a graded aggregate base (GAB) resulted in an approximately 1-2 in. thinner asphalt layer when all other factors remained the same.

- The traffic volume followed the expected trend of increasing pavement thickness with AADTT. The sensitivity of the pavement to traffic, however, decreased as the two-way AADTT increased from 6,000 to 30,000.
- The terminal IRI, permanent deformation (total and AC only), AC thermal cracking, and AC top-down cracking values corresponding to AC bottom-up fatigue cracking of 2% lane area are well within the threshold values for the input range and parameters used in the study.

5) Local Calibration of the Distress and Performance Models

Rigid New JPCP

Based on the local calibration of the "Rigid New JPCP" distress and performance models in this study, it is recommended to continue to use the global calibration factors for the Transverse Cracking and Mean Joint Faulting models until additional pavement distress data is available to improve the calibration; whereas, the IRI model was improved by local calibration, and thus the coefficients for the IRI model developed herein may be used with the understanding that they should be updated and improved as more data becomes available. The local calibration coefficients obtained for this model are:

  - IRI: $C_1 = 5.5$, $C_2 = 12.5$, $C_3 = 2.9$ and $C_4 = 40.5$

In the future, there should be more pavement sections added to the calibration along with additional, correctly formatted pavement distress data collected for the 11 pavement sections used herein.

Flexible New AC

The findings from the local calibration of the “Flexible New AC” distress and performance models in this study include the following:

- The Bottom-up Fatigue Cracking, Total Rutting, and IRI models were improved by local calibration. For each of these models, one or more hypothesis tests showed a failing result; however, the calibration was accepted because there was a close to zero bias with a SEE lower than the global model, as suggested by AASHTO (2010). The local calibration coefficients obtained for these models are:
    - Bottom-up Fatigue Cracking: $\beta_{f2} = 1.5$, $\beta_{f3} = 0.85$, $C_1 = 1.75$, $C_2 < 5$ in. = 2.75, and $C_2 > 12$ in. = 3.5
    - Rutting: $\beta_{r1} = 0.2$, $\beta_{r2} = 0.3$, $\beta_{r3} = 1$, $\beta_{s1} = 1.2$, and $\beta_{sg1} = 1.1$
    - IRI: $C_1 = 20$, $C_2 = 0.35$, $C_3 = 0.001$ and $C_4 = 0.011$
- Calibration of the Top-down Fatigue Cracking model was not satisfactory due to variability in the measured data. The variation in the predicted top-down fatigue cracking is statistically different from the measured values and is not improved through multiple calibration trials.

Recommendations

PCC Pavement

The following recommendations are put forth for PCC pavements and the calibration of the "Rigid New JPCP" pavement models based on the findings from this study:

- Given the lack of historical distress data available for the JPCP sections in South Carolina, it is highly recommended that the SCDOT begin collecting transverse cracking and mean transverse joint faulting data that is quantified in the format required by the MEPDG. This distress data should be collected annually, at a minimum, to improve the calibration of the three JPCP distress and performance models. SCDOT's new Laser Crack Measurement System (LCMS) vehicle and Traffic Speed Deflectometer (TSD) data (AARB's iPAVe vehicle) can be utilized for this purpose. IRI data should continue to be collected annually as per current SCDOT practice.
- Perform annual distress surveys on the three newly constructed JPCP pavement sections (Cherokee/I-85, Lexington/I-20, and Spartanburg/I-852) to establish a comprehensive history of the pavement distresses over time for PCC pavements in South Carolina. These pavement sections have baseline material property data that were obtained in this study.
- Continue to acquire WIM data and update the files for hourly and monthly traffic distribution and truck classification as more data becomes available. The distress models should be recalibrated after a full two years of data has been collected.
- Add additional PCC sections to increase the data set for the "New Rigid JPCP" distress and performance models.
- Based on the findings from this study, the SCDOT should consider using the local calibration coefficients for the IRI model with the intention of continuing to update and

improve the model as more distress data becomes available. For the cracking and faulting models, it is recommended to continue to use the global calibration factors until additional long-term distress data is available for calibration.

AC Pavement

The following recommendations are put forth for AC pavements based on the findings from this study:

- The sets of calibration coefficients obtained using AASHTOWare Pavement ME Design software for the “Flexible New AC” Bottom-up Fatigue Cracking, Rutting, and IRI models were found to reduce the bias and SEE in predicting distresses and IRI. Thus, the locally calibrated models for South Carolina are considered reasonable and recommended for use in design. These coefficients need to be continually re-evaluated as additional cycles of high quality distress measurements become available.
- The SCDOT should maintain the global coefficients for the “Flexible New AC” Top-down Fatigue Cracking model since reliable local calibration coefficients were not obtained.
- Improvement for all “Flexible New AC” models is still possible by collecting high-quality distress and performance data (especially for the cracking models) through field (e.g., LCMS) and forensic investigations (differentiate between bottom-up and top-down cracking) and continuing to collect distress and performance data on an annual basis.
- Inputs (i.e., pavement structure, materials, and traffic) need to be obtained for pavements with semi-rigid bases to facilitate the calibration of the "Flexible New Semi-Rigid" models. Inputs for 10 segments were compiled in this study, and inputs for at least 10 additional segments are needed. This is important because the SCDOT is increasingly constructing pavements with semi-rigid bases (e.g., cement-modified recycled base).

- It is recommended that the models be recalibrated in three years. In particular, the Bottom-up Fatigue Cracking and Top-down Fatigue Cracking models will benefit from an additional 2-3 years of high-quality condition survey data, as per the AASHTO (2010) recommendation that at least three condition survey data points are needed for each segment to estimate the incremental increase in distress over time.
- For each new pavement that is constructed in South Carolina, the inputs necessary for calibration should be collected and used to populate the local calibration database before the next calibration cycle. This will facilitate continuous improvement of the local calibration models.
- The vast amounts of input data compiled in this study (pavement structure, materials, traffic, and climate) for 66 “Flexible New AC” and 10 “Flexible New Semi-Rigid” pavement segments serve as a resource database for future pavement design and model calibrations. This database should be continually updated as data for new pavement sections is obtained, and more years of field distress and performance data become available.
- Future research should include fine-tuning the calibration using advanced statistical approaches such as those studied by Brink (2015), Smith and Nair (2015), and Islam et al. (2019). The proposed research includes identifying and ranking a set of MEPDG inputs sensitive to particular pavement distress models (Schwartz 2013), analyzing the sensitivity of input variables (Sumee 2011) (see Table 27), and performing one-to-one sensitivity analysis using South Carolina pavement segments.

Table 27 Proposed Test Matrix for Sensitivity Analysis

Variable	Range value
Air void (%)	2 to 10
Binder content	8 to 15
Fine content	2 to 12
AC thickness (inch)	3 to 15
Depth of GWT	10 to 25
AADTT	200 to 5000
Base thickness (inch)	4 to 18
Performance grade	PG 58-28, PG 64-28, PG 70-22, PG 76-22, PG 82-22, AC 20

- Future research should include establishing an AASHTOWare Pavement ME User Manual for the SCDOT. This document will establish guidelines based on local calibration factors for South Carolina. Recently, the Virginia Department of Transportation (VDOT 2017) and Michigan Department of Transportation (MDOT 2021) established an AASHTOWare Pavement ME user manual that was implemented for analyzing and designing pavement structures.
- Future research should include updating the *Asphalt Pavement Design Catalog for High-Volume Roads in South Carolina* that was developed herein with the proposed local calibration coefficients.
- The Phase III study should include the calibration of the “Flexible AC Semi Rigid” and “Flexible AC Rehab” distress and performance models.

Implementation Plan

- The locally adjusted calibration coefficients shown in Table 28 should be incorporated into AASHTOWare Pavement ME Design when analyzing or designing new asphalt and rigid pavement structures with the understanding that there is still room for improvement. The models should be continuously updated and improved as more meaningful distress data becomes available.

Table 28 Recommended Pavement ME Design Coefficients from Local Calibration in South Carolina

Pavement	Model	β_{f2}	β_{f3}	C_1	C_2		β_{r1}	β_{r2}	β_{r3}	β_{s1}	β_{sg1}	C_3	C_4
					<5 in.	>12 in.							
"Flexible New AC"	Bottom-up Fatigue Cracking	1.5	0.85	1.75	2.75	3.5							
	Rutting						0.2	0.3	1	1.2	1.1		
	IRI			20	0.35							0.001	0.011
"Rigid New JPCP"	IRI			5.5	12.5							2.9	40.5

- The vast amounts of input data compiled in this study (pavement structure, materials, traffic, and climate) are available as a resource database for future pavement design and model calibrations. This database should be continually updated as data for new pavement sections is obtained and more years of field distress and performance data become available.
- The resilient modulus for subgrade soils obtained from repeated load triaxial testing, and the relations between Falling Weight Deflectometer data, CBR tests, and SSV, developed in this study, can be used in a catalog of pavement subgrades (by county and geologic region) to aid the SCDOT in future pavement design.

- The asphalt pavement thickness design catalog that was developed based on the MEPDG using global calibration coefficients is available for the design of interstates and other high-volume routes in South Carolina.
- The results of this study should be used to inform Phase III of the calibration research effort.

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