DETERMINATION AND EVALUATION OF INPUTS FOR PORTLAND CEMENT CONCRETE PAVEMENT TO SUPPORT LOCAL CALIBRATION OF MEPDG FOR NORTH CAROLINA

by

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ABSTRACT

EDWARD HARRISON BLANCHARD. Determination and evaluation of inputs for portland cement concrete pavement to support local calibration of MEPDG for North Carolina. (Under the direction of DR. TARA L. CAVALLINE)

The Mechanistic-Empirical Pavement Design Guide (MEPDG) is a state-of-the-practice tool that is used for the analysis and design of pavements, using mechanical and statistical models that have been developed over the past several decades for the prediction of pavement deteriorations, and is currently available as the AASHTOWare Pavement ME software. AASHTO states that local calibration is necessary for optimal performance of MEPDG. The goals of this research study were to develop a catalog of inputs for portland cement concrete (PCC) to be utilized in MEPDG pavement design and analysis and to gain an understanding of the impact of these new concrete inputs on design and predicted performance for North Carolina pavements.

Eighteen different concrete mixtures, produced using a variety of materials local to North Carolina, were batched and tested to provide data to support development of a catalog of new, locally appropriate PCC inputs. To facilitate analysis of the impact of the new PCC inputs, selected typical pavement designs for different types of North Carolina roadways were re-analyzed using the AASHTOWare Pavement ME software, utilizing the new PCC inputs in place of the PCC inputs previously used (typically defaults). It was consistently found that the predicted performances of pavement sections re-analyzed using the new suggested input values found through laboratory testing of concrete with locally available materials outperform those sections as designed using the input values for PCC currently utilized by NCDOT. Additionally, it was determined that use of the new PCC input values may also result in the design of slightly thinner concrete pavements in the future. Thinner pavements will reduce the amount of materials used in pavement construction, resulting in lower costs and environmental impact of concrete pavement.

A sensitivity analysis was also conducted to compare the relative sensitivity of distress measures to changes in inputs. Findings of the sensitivity analysis using the proposed North Carolina PCC inputs were similar to those of sensitivity analyses performed by other researchers.

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LIST OF ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
ASTM	American Society for Testing and Materials
BTU	British Thermal Unit
°C	degrees Celsius
cf	cubic feet
CRCP	continuously reinforced concrete pavement
CTE	coefficient of thermal expansion
DOT	Department of Transportation
°F	degrees Fahrenheit
FHWA	Federal Highway Administration
ft	foot
HMA	hot mix asphalt
hr	hour
in	inch
JPCP	jointed plain concrete pavement
kg	kilogram
lb	pound
LTPP	Long Term Pavement Performance
m	meter
MEPDG	Mechanistic-Empirical Pavement Design Guide
mi	mile
min	minute
MOE	modulus of elasticity

MOR	modulus of rupture
NCDOT	North Carolina Department of Transportation
NCHRP	National Cooperative Highway Research Program
OAT	one-at-a-time analysis
OPC	ordinary portland cement
oz	ounce
PCC	portland cement concrete
PLC	portland limestone cement
pcy	pounds per cubic yard
pcf	pounds per cubic foot
psi	pounds per square inch
SCM	supplementary cementitious material
w/c	water to cementitious material ratio

CHAPTER 1: INTRODUCTION

The Mechanistic-Empirical Pavement Design Guide (MEPDG) is a state-of-the-practice tool that is used for the analysis and design of pavements. In an effort to improve the previous method of pavement design utilized by many state agencies, the 1993 AASHTO Guide for Design of Pavement Structures, MEPDG uses mechanical and statistical models that have been developed over the past several decades for the prediction of pavement deteriorations. However, in order to calculate those predictions, MEPDG utilizes a method that is reversed from the traditional method (Pavement Interactive 2012). Traditional methods utilize a variety of inputs to develop design requirements for the pavement structure such as thickness and dowel spacing. Conversely, MEPDG uses selected design requirements for the pavement structure (including layer materials, associated properties and thicknesses) as assumed input parameters along with the variety of inputs like traffic and climate in order to produce a predicted model that represents how the pavement will perform based on distresses over time.

1.1 Background and Significance

In the past, NCDOT used the 1974 version of the AASHTO Guide for Design of Pavement Structures. After introduction of the 1993 version of the AASHTO Guide for Design of Pavement Structures, NCDOT elected not to adopt updated methods. Instead, NCDOT chose to utilize regional factors to adjust the 1974 version of the AASHTO Guide, utilizing this hybrid method for design of pavement structures for North Carolina pavements.

Developed in the 1980's, 1990's, and early 2000's, MEPDG represents the efforts of millions of research dollars and the efforts of a number of researchers to advance the practice of pavement design. Currently available as the AASHTOWare Pavement ME software program, MEPDG has been adopted by most state highway agencies, and is currently utilized by NCDOT for design of both rigid and flexible pavements. To better predict the performance and useful life of pavements, a local calibration is strongly recommended by AASHTO. The local calibration of MEPDG consists of calibrating local input parameters as well as validating the local input parameters. Calibration in MEPDG is used to eliminate bias and reduce error in observed results. The empirical calibration parameters are modified to best fit local materials. Validation is the process of confirming the observed and predicted distress are similar for samples observed in the field (AASHTO, 2010).

Currently, for rigid pavement design, NCDOT utilizes the Level 3 (default) values for a number of inputs for concrete pavements. Local calibration of the MEPDG software includes utilizing inputs from locally available (North Carolina) materials such as Portland Cement Concrete (PCC). For flexible pavements, many local inputs were determined through testing supporting the Federal Highway Administration's (FHWA) "Local Calibration of the M-EPDG Using Pavement Management Systems (FHWA Report No. HIF-11026, 2010)." Therefore, there is a need for the local calibration of MEPDG utilizing locally available PCC materials.

There is a need for the testing of concrete produced with representative, locally available materials from across North Carolina in order to strengthen the MEPDG predictions for NCDOT. North Carolina has a variety of materials available for concrete pavements due to having distinct geological features in the Coastal region, Piedmont region, and Mountain region. Laboratory testing to develop a catalog of MEPDG Level 1 inputs for concrete mixtures representing North Carolina pavement concrete would provide the Level 1 inputs that could be utilized in MEPDG to provide more reliable predictions for pavement performance, and ultimately, improved design of concrete pavements in North Carolina. With the local calibration completed, the predicted cracking, faulting, smoothness, and other distresses predicted by MEPDG for anticipated climate and traffic conditions should be improved.

MEPDG inputs for concrete pavements include mechanical and thermal properties for concrete. Mechanical property inputs in MEPDG include 28-day compressive strength, modulus of rupture (MOR), modulus of elasticity (MOE), and Poisson's ratio. Each of these inputs were found to be sensitive inputs in MEPDG by multiple researchers such as Schwartz et al. (2011), Guclu and Ceylan (2005), and Guclu et al. (2009).

Thermal properties for concrete utilized as MEPDG inputs include coefficient of thermal expansion (CTE), thermal conductivity, and heat capacity. Each of these properties (especially CTE) were also found to be sensitive by researchers including Schwartz et al. (2011), Guclu and Ceylan (2005), and Guclu et al. (2009). The CTE measures expansion or contraction of a material with temperature (Tarun et al. 2011), and has been the focus of a number of research studies for MEPDG design of concrete pavements. The CTE values are influenced by multiple characteristics of the concrete, including the cement paste, aggregates, moisture conditions, age, and environmental factors such as temperature fluctuations and relative humidity. However, the greatest variation in the CTE value in concrete has been shown to be associated with the type of aggregate (particularly the coarse aggregate) that is used in the concrete (McCarthy et al. 2014; Naik et al. 2011; Sakyi-Bekoe 2008; Tanesi et al. 2007).

After laboratory testing is completed to identify a catalog of new inputs for concrete pavements, a comparison between four NCDOT selected pavement projects as well as a sensitivity analysis could be completed. The comparison could be utilized to compare the predicted distresses of the previously used AASHTOWare Pavement ME software inputs to the predicted distresses of the new suggested inputs that are developed to evaluate the impact of the new inputs on North Carolina pavement design. A sensitivity analysis could also be performed to evaluate the effect of the new inputs on the predicted performance of North Carolina concrete pavements, as well as to identify the input values to which design of North Carolina concrete pavements will be most sensitive.

Ultimately, identification of new inputs for MEPDG design of North Carolina concrete pavements should provide NCDOT additional tools to design safer, more reliable concrete pavements in North Carolina. Insights could be gained into both the predicted service lives of concrete pavements, as well as modes of failure and potentially insight into required maintenance. This study will help NCDOT's confidence in the performance of concrete pavements by providing more representative input values which in turn should produce better predictive models of distresses.

1.2 Organization of the Thesis

Chapter 1 provides an overview of the purpose for this research study. Chapter 2 is a literature review that provides relevant information on MEPDG, efforts to support local calibration of MEPDG, North Carolina concrete materials, and sensitivity analysis. Chapter 3 describes the methodology utilized for performance of the laboratory portion of this research study. Chapter 3 includes a description of the materials used, concrete mixtures batched, preparation of test specimens, and test procedures utilized. Chapter 4 contains the results of the laboratory testing performed as part of this study, with a primary focus on the test results utilized to develop values identified for inclusion in the catalog of inputs for PCC pavement design provided to NCDOT for use in design and evaluation of North Carolina concrete pavements. Chapter 5 provides the analysis and comparison of the pavement design for four projects selected by NCDOT using the previous PCC inputs to the same design re-analyzed with the new suggested PCC inputs. Chapter 6 provides the parameters and results of a sensitivity analysis performed to evaluate the impact of the newly obtained PCC inputs on the predicted performance of North Carolina concrete pavements. Chapter 7 provides the conclusions of this study and recommendations for further work.

CHAPTER 2: LITERATURE REVIEW

This chapter provides an overview of the MEPDG process and AASHTOWare Pavement ME software. Also provided is a review of literature on the materials-related inputs utilized in the MEPDG process, with a focus on the material properties that have been shown to be sensitive in previous research studies on MEPDG pavement design.

2.1 Mechanistic-Empirical Pavement Design Guide (MEPDG)

MEPDG provides a state-of-the-practice tool for design of pavements for the transportation industry (AASHTO 2008). MEPDG is used as a prediction performance tool that predicts the pavement performance of a user-specified pavement section through its design life instead of providing the pavement thickness required for design (Gulcu et al. 2009). Simplified, the mechanistic-empirical design process uses mechanistic models to compute pavement responses to traffic and climate loads, and predicts damage over time. The cumulative damage is empirically related to observed pavement distresses. By modifying input values such as climate, traffic, and pavement information, the MEPDG software (currently available as AASHTOWare Pavement ME) will modify its prediction of how that pavement section will perform. However, the reliability of these predictions is related to the accuracy of the mechanistic inputs.

Global (or default) input values are provided in the software, and recommended input values are published in a number of sources (AASHTO 2015). However, the local calibration of MEPDG, including identification of input values representative of local materials and construction practices, is highly recommended because local conditions and materials may vary significantly from the provided global calibration models and inputs (AASHTO 2010). The overall fidelity of the MEPDG performance prediction is improved when the input values utilized for the pavement components are obtained through testing of locally available materials, and subgrade values represent site conditions that will affect the predicted life (Gulcu et al. 2009).

AASHTOWare Pavement ME uses three main distress failure modes for jointed plain concrete pavements (JPCP): transverse cracking, mean joint faulting, and terminal international roughness index (IRI) (AASHTO 2015). Each threshold for the distresses are set by the local agency depending on a variety of conditions. A brief description of each distress along with influencing input parameters follow.

Transverse cracking is measured in “percent slabs” (cracked) for use in MEPDG and includes both predicted bottom-up and top-down cracking. Bottom-up cracking typically comes from large bending stresses generated by truck wheel loads near the longitudinal edge of the slab midway between the transverse joints. A high positive temperature gradient (the bottom of the slab is cooler than the top of the slab) increases this bending stress greatly (AASHTO 2010). Top-down transverse cracking primarily comes from fatigue loading from truck traffic loads with certain axle spacing’s as well as a negative temperature gradient (the top of the slab is cooler than the bottom of the slab) (AASHTO 2010, Mallick and El-Korchi 2009). According to Gulcu et al. (2009), an increase in unit weight, Poisson’s ratio, and CTE leads to an increase in cracking. An increase in thermal conductivity, MOR, and compressive strength leads to a decrease in cracking.

Joint faulting is measured in “inches” and quantifies the elevation difference between two adjacent slabs (Mallick and El-Korchi 2009). Since the degree of joint faulting varies by individual joint, and along joints throughout the pavement analyzed, the actual distress used by AASHTOWare Pavement ME predictions is mean joint faulting, which accounts for all joints throughout a pavement section. According to Gulcu et al. (2009), an increase in Poisson’s ratio and CTE leads to an increase in faulting and an increase in unit weight and thermal conductivity leads to a decrease in faulting.

Terminal IRI, also referred to as smoothness, is a measure of the roughness of a roadway, and therefore includes the impacts of both transverse cracking and joint faulting. Terminal IRI measures the smoothness or roughness of the roadway, and is measured in “inches per mile.” Several concrete inputs have been shown to affect smoothness. An increase in Poisson’s ratio and an increase in CTE each leads to an increase in smoothness (Guclu et al. 2009). An increase in thermal conductivity, MOR, and compressive strength leads to a decrease in smoothness.

Previous research has been performed to identify the role of concrete’s material properties in predicted pavement performance using MEPDG. In summary, to get an optimized pavement, it is better to have a lower input value for Poisson’s ratio and CTE and a higher input value for thermal conductivity, MOR, and compressive strength. It should also be noted that a greater input value for unit weight increases cracking but decreases faulting and therefor is found to be insensitive for smoothness (Guclu et al. 2009).

2.2 Local Calibration of MEPDG

The local calibration of MEPDG for all types of pavements is highly recommended by AASHTO because the United States has such a variety of available materials utilized for construction of pavements, as well as different subgrade, climatic, construction preferences, and other influencing factors. The suggested input values for materials in the global calibration of MEPDG were determined using a representative sample of test sites around North America, primarily those included in the long-term pavement performance (LTPP) (AASHTO 2010). The variability in properties of locally available materials throughout the United States, coupled with the increasing availability of new materials (such as recycled aggregates, supplementary cementitious materials, etc.) can render the predicted performance of pavements designed using global defaults available in the MEPDG software unreliable (AASHTO 2010).

In order for future users of the MEPDG software to have the best working predictions and models, leading to confidence in the design process, the calibration-validation process is important (AASHTO 2008). The local calibration of MEPDG is used to eliminate bias and reduce error in observed results, and consists of identifying locally-relevant input parameters for materials, subgrade, traffic, climate, and other conditions, as well as validating the empirically observed distresses for an area. The empirical calibration process uses laboratory testing to modify the parameters based on local materials. Validation is the process of confirming the predicted distress are similar to the actual observed distresses in the field (AASHTO 2010).

The MEPDG process utilizes three levels of inputs, each with a different level of accuracy. The three levels of inputs are as follows (AASHTO 2008):

- Level 1 inputs are the most accurate with site-specific, mixture-specific input values. Level 1 inputs should be used to develop correlations and defaults included for Level 1 and Level 2 inputs as well as projects that have unusual characteristics. These input parameters are typically the most expensive to develop and implement.
- Level 2 inputs are estimated input values based upon correlations or regression equations that use site-specific information from similar projects. These values are typically less expensive to develop than Level 1 inputs.
- Level 3 inputs are default values and based on global or regional values. The values are median values representing a group of data with similar characteristics. These inputs have the least amount of knowledge, however, they have the lowest data collection costs.

When Level 1 inputs are obtained through laboratory testing and are utilized with locally calibrated deterioration models, the new designs will better predict pavement distresses and have a tighter prediction of performance (AASHTO 2008). Since the development of MEPDG, a number of state highway agencies have performed extensive studies to determine locally accurate inputs for use in the AASHTOWare Pavement ME software (Darter et al. 2009, Guclu and Ceylan 2005, Kodide 2010, Ley et al. 2013, Tran et al. 2008). This includes inputs for concrete, asphalt, subgrade, and other materials utilized in pavement layers. As this study is focused on concrete pavements, the subsequent sections of this literature review focus on local calibration to support rigid pavement design.

2.2.1 PCC Inputs in MEPDG for Concrete Pavement Design

Rigid pavement design using MEPDG requires a number of materials-specific input parameters, including information on materials comprising the base course, subbase course, and concrete pavement layers. Additional information specific to joint reinforcement and dowels is also required. Specific materials-related inputs for PCC include mechanical properties such as compressive strength, MOR, MOE, and Poisson's ratio. Thermal properties utilized in MEPDG are thermal conductivity, heat capacity, and coefficient of thermal expansion. The following sections contain information on the key concrete materials-related inputs in MEPDG. These sections provide background on the recommended default input values, the values utilized by other states as Level 1 inputs, and key findings of other research projects identifying and evaluating local PCC inputs for MEPDG.

2.2.1.1 Mechanical Properties

MEPDG inputs for rigid pavement include several mechanical properties of PCC: 28-day compressive strength, 28-day MOR, 28-day MOE, and 28-day Poisson's ratio. These mechanical properties are commonly utilized for overall characterization of a concrete mixture as well as for quality assurance and control in the field and laboratory. The compressive strength of concrete utilized as an MEPDG input is the compressive strength predicted at an age of 28 days utilizing ASTM C39, "Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens." A typical Level 3 input value is not provided for compressive strength however there are multiple options in the software available for this input. The first option is to input compressive strength and MOE and allow the software to calculate the MOR and the second option is to input the MOR and the MOE and have the software calculate the compressive strength (AASHTO 2015).

MOR testing is often used by state agencies (including NCDOT) for quality assurance and control of concrete for pavement construction. ASTM C78, "Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)" is the standard recommended for testing and calculating the Level 1 and 2 input values for the MOR for MEPDG, at an age of 28-day (AASHTO 2015). A typical Level 3 input value is not provided for MOR but is stated in conjunction with compressive strength (AASHTO 2015).

The MOE corresponds to the compressive strength of the concrete, and defines the relationship between deformation and applied stress. As the compressive strength of the concrete is increased, the MOE is increased (Neville 2011). The characteristics (type, size, angularity, etc.) of aggregate in the concrete influences the MOE, based upon the MOE of the aggregate as well as the proportions of aggregate in the concrete (Neville 2011). In MEPDG, MOE is specified to be determined at an age of 28-day in accordance with ASTM C469, "Standard Test Method for Static MOE and Poisson's Ratio of Concrete in Compression." There is not a typical Level 3 input provided for MOE, only a range of 0.3 x 10^6 psi (for inadequate pavement condition) to 4 x 10^6 psi (for adequate pavement condition), with 1 x 10^6 psi to 3 x 10^6 psi being the suggested input range for pavement in marginal condition (AASHTO 2015).

Poisson's ratio is the relationship between strain in the longitudinal direction and strain in the lateral direction when a known load is applied. The longitudinal strain is in the direction of the applied load where the lateral strain is perpendicular to the applied load, with the longitudinal strain with the specimen in compression and the lateral strain with the specimen in tension. Unlike the MOE, a connection cannot be made between measured values of Poisson's ratio and the aggregates being used in the mixture (Neville 2011). In MEPDG, Poisson's ratio is specified to be determined at an age of 28-day in accordance with ASTM C469, "Standard Test Method for Static MOE and Poisson's Ratio of Concrete in Compression." A typical Level 3 Poisson's Ratio of 0.20 is provided by AASHTO (AASHTO 2015).

2.2.1.2 Thermal Properties

Thermal properties of PCC that are utilized as inputs in MEPDG are CTE, heat capacity, and thermal conductivity. Thermal properties have been shown by a number of research studies to be significant in influencing the performance of pavements in MEPDG (Kodide 2010, Mallela et al. 2005), affecting the rates of increase in IRI, cracking, and joint faulting. Since a large proportion (by both mass and volume) of concrete is comprised of aggregate, the thermal properties of aggregates have been shown to heavily influence the thermal performance of the bulk concrete (Mehta and Montiero 2014, Neville 2011).

Cracks in a concrete structure can be caused by thermal effects, including the heat of hydration or occurrence of temperature gradient (Kook-Han 2003). Temperature profiles developed along with any given structure can be precisely estimated along with locations at a certain time through understanding the analysis of heat conduction. One of these parameters is heat flow, which accounts for the temperature gradients between two materials. Thermal conductivity is the ratio of heat flux to temperature gradient (Kodide 2010). Conduction is the movement of heat within a solid material or due to the contact of solid objects. Along with the moisture profiles produced utilizing the climatic data and other inputs, the Pavement ME software performing the MEPDG process analyzes the thermal stresses and strains in PCC pavements. The thermal properties of the material control the amount of heat flow. ASTM E1952, "Standard Test Method for Thermal Conductivity Diffusivity by Modulated Temperature Differential Scanning Calorimetry," is recommended for testing for the Level 1 and 2 inputs for MEPDG. No recommendations are provided for the age or moisture conditioning of the specimen. Level 3 default values for thermal conductivity range from 0.2 to 2.0 BTU/(ft)(hr)(°F) but 1.25 BTU/(ft)(hr)(°F) is the default value set to use (AASHTO 2015).

The CTE value is also a fundamental thermal property of PCC. Higher concrete CTE values have been associated with higher predicted instances of early-age or premature random cracking, higher midpanel transverse and longitudinal cracking, faulting caused by a greater loss of slab support during construction, and joint spalling (Mallela at al. 2005). Early-age cracking or premature random cracking have also been shown to result from excessive longitudinal slab movement caused by a higher CTE value and where the slab is restrained (Mehta and Monteiro 2014). The loss of slab support during construction often causes curling which allows for larger corner deflections and joint openings (Mallela at al. 2005). Joint spalling is a result of excessive joint opening and closing which increases with a larger CTE (Mallela at al. 2005).

The CTE was originally used in the predecessor to MEPDG, the AASHTO 1993 Pavement Design Guide. The CTE was used for transverse joint sealant design and longitudinal reinforcement design (Tran 2008). A focus of a number of recent studies supporting local calibration of MEPDG for highway agencies, the CTE of concrete has been shown to be influenced by multiple components including the cement paste, aggregates, moisture conditions, age, and environmental factors such as temperature fluctuations and relative humidity (Tarun 2011). The greatest variation in the CTE value in concrete comes from the aggregate that is used in the concrete (McCarthy et al. 2014, Naik et al. 2011, Sakyi-Bekoe 2008, Tanesi et al. 2007). A detailed discussion on the influence of CTE on predicted pavement performance in MEPDG is provided in Section 2.4.2.1. The typical CTE values for known aggregates in PCC pavements range from 4.6 to 6.6 x 10^{-6} in/in/°F with a default value of 5.5 x 10^{-6} in/in/°F for unknown coarse aggregates (AASHTO 2015).

2.3 Sensitivity Analysis

An analysis commonly utilized after local calibration of MEPDG is a sensitivity analysis, as it is important for designers to understand which variables will have the greatest influence on the predicted pavement performance. The *Sensitivity Evaluation of MEPDG Performance Prediction* (Schwartz et al. 2011) was a study performed for the National Cooperative Highway Research Program (NCHRP) of the Transportation Research Board. In this research study, selected input values were utilized to perform a one-at-a-time (OAT) analysis in order to create a hierarchical list of sensitive input values. In this study, the researchers studied the performance of five different pavements types, New Hot Mix Asphalt (HMA), HMA Over Stiff Foundation, New JPCP, JPCP Over Stiff Foundation, and New Continuously Reinforced Concrete Pavement (CRCP). The stiff foundation represented the pavement overlays on an existing pavement. Five different climate conditions (Hot-Wet, Hot-Dry, Cold-Wet, Cold-Dry, and Temperate) and three traffic levels (Low, Medium, and High) were evaluated for each of the pavement conditions. Findings of this study for the JPCP are the most relatable for this research, and will be useful for comparison of a similar sensitivity analysis with North Carolina inputs, and are therefore detailed herein.

In Table 2.1, the findings of the sensitive inputs for MEPDG as determined by Schwartz et al. (2011) are shown. In Table 2.1, the inputs are sorted, and are presented based upon sensitivity. Further discussion is presented on each of these inputs to support this research project. However, only materials-related inputs will be discussed in this literature review, as they are the focus of this study. Specifically, these inputs include PCC 28-day MOR, PCC 28-day MOE, PCC thermal conductivity, PCC coefficient of thermal expansion, and PCC unit weight. Input values associated with traffic, climate, subgrade, shoulder, and site inputs were not the focus of this study, and are therefore not discussed further in this literature review.

As can be seen in Table 2.1, the concrete materials inputs deemed most sensitive for new JPCP are PCC 28-day MOR, PCC 28-day MOE, PCC thermal conductivity, PCC CTE, PCC unit weight, and PCC Poisson's ratio. All of the sensitive inputs listed along with PCC heat capacity are being researched in this study for use in gathering locally representative input values for North Carolina.

Table 2.1: Ranking of design inputs by maximum absolute NSI: New JPCP (Schwartz et al. 2011)

New JPCP	Sensitivity[1]	
	OAT Analysis	Initial Triage
PCC 28-Day Modulus of Rupture	**-16.55**	VS
PCC Thickness	**-15.03**	VS
Surface Shortwave Absorptivity	**10.99**	VS
Joint Spacing	**9.91**	VS
PCC 28-Day Modulus of Elasticity	**9.87**	S[2]
Design Lane Widthe (14ft Widened Slab)	**-7.20**	S
Edge Support - Widened Slab	**-6.60**	VS
PCC Thermal Conductivity	**-5.33**	VS
PCC Coef. of Thermal Expansion	***4.63***	VS
PCC Unit Weight	***3.60***	S
Dowel Diameter	***-2.46***	S
PCC Poisson's Ratio	***1.53***	S
Traffic Volume (AADTT)	***1.25***	VS
Base Resilient Modulus	***1.07***	VS
Subgrade Resilient Modulus	*-0.86*	S
PCC Cement Content	*0.83*	S
Construction Month	*0.67*	-
PCC Water-to-Cement Ratio	*0.42*	S
Groundwater Depth	*-0.32*	NS
Erodibility Index	*0.25*	S
Base Thickness	*-0.20*	S
Design Lane Width (No Edge Support)	-0.08	S
Edge Support - Load Transfer Efficiency	-0.07	-
Design Lane Width (80% LTE)	0.00	S

[1]Maximum sensitivity (in absolute value sense) over all baseline cases and distresses. Sensitivity ratings are indicated by font type: Bold designates Hypersensitive, NSI > 5; Bold Italics designates Very Sensitive, 1 < NSI < 5; Italics designates Sensitive, 0.1 < NSI < 1; and Regular font designates Insensitive, NSI < 0.1. Bold lines indicate breaks between sensitivity categories. Shaded entries indicate discrepancies between OAT results and the initial triage.

[2]Inputs that were only implicitly evaluated during the initial triage.

It is noted that additional research is resulting in updates to the MEPDG process and therefore the AASHTOWare Pavement ME software. Therefore, the sensitivity of some parameters could change over time as research supporting the initiative advances.

For example, Schwartz et al. (2011) found that in versions of the MEPDG software prior to version 1.0, some sensitivity results changed greatly. However, changes in sensitivity were not found between Versions 1.0 and 1.1. Guclu et al. (2009) found similar results between versions 0.7, 0.9, and 1.0, and noted that in the later versions (newer), greater impacts in changes of input sensitivity were observed. It is noted that at the time of this study, the version of AASHTOWare Pavement ME software currently available and used in this study is version 2.1.

2.4 Sensitivity of MEPDG to Input Values for Concrete Material Properties

Concrete input values in MEPDG fall under two main categories, mechanical properties and thermal properties. Research on the influence of these inputs on the performance of PCC pavement design has been the focus of a number of studies. To date, a number of states (including Alabama, Florida, Iowa, Louisiana, and Oklahoma) have sponsored research projects to identify inputs specific to local materials utilized in PCC pavements produced in these states (Sakyi-Bekoe 2008, Tia et al. 2005, Wang et al. 2008, Shin and Chung 2011, Ley et al. 2013).

As discussed previously, Schwartz et al. (2011) used version 1.1 to perform sensitivity analysis for faulting, transverse cracking, and IRI along with a OAT sensitivity analysis using the parameters previously described (HMA, HMA over stiff foundation, JPCP, JPCP over stiff foundation, and CRCP). This study did not use locally calibrated values, only default values ranging from minimum to maximum, while also changing the climate conditions (Hot-Wet, Hot-Dry, Cold-Wet, Cold-Dry, and Temperate) and traffic conditions (low, medium, and high). The input values ranged from “hypersensitive” to “very sensitive” to “sensitive” to “non-sensitive.” Guclu and Ceylan (2005) used an older version of the software that was not stated in the report, but determined the sensitivity of a number of Iowa materials-specific input values in influencing faulting, transverse cracking, and smoothness with sensitivity ratings of “extreme sensitivity,” “sensitive to very sensitive,” and “low sensitive to insensitive.” Hall and Beam (2005) used an older version of the software (version also not stated in the report), and found sensitivity results for faulting, cracking, and smoothness on a “sensitive” or “insensitive” basis. Guclu et al. (2009) also revisited their previously published sensitivity analysis using version 1.0 of the software, which has been reported to provide more representative sensitivity analysis results. Their results were again providing analysis results for faulting, cracking, and smoothness and sensitivity ratings of “very sensitive,” “sensitive,” and “insensitive.” A summary of the findings of this work, as well as other similar sensitivity analyses, is provided in the subsequent sections of this literature review.

2.4.1 Mechanical Properties

The following sections provide some detail on the findings of other sensitivity analyses to the mechanical property inputs for concrete in MEPDG.

2.4.1.1 Unit Weight

According to Schwartz et al. (2011) unit weight was found to be a “very sensitive” input value during the OAT analysis. Guclu and Ceylan (2005) found unit weight to be a “low sensitive to insensitive” input value for both faulting and smoothness. However, unit weight was found to be a “sensitive to very sensitive” input value for transverse cracking. In the newer software version, Guclu et al. (2009) found unit weight to be “sensitive” for faulting and cracking and “insensitive” for smoothness. Gulcu et al. (2009) also found that as the input value for unit weight increases the predicted faulting decreases and cracking increases. Hall and Beam (2005) performed a sensitivity analysis that found unit weight to be a sensitive input for all of the distresses (faulting, cracking, and smoothness).

2.4.1.2 Modulus of Rupture

According to Schwartz et al. (2011), MOR was found to be a "hypersensitive" input value during the OAT analysis. Guclu et al. (2009) found MOR to be "insensitive" for faulting, "very sensitive" for cracking, and "sensitive" for smoothness. Gulcu et al. (2009) also found that as the input value for MOR increases the predicted damage of each distress decreases. Hall and Beam (2005) found MOR to be a sensitive input for both cracking and smoothness.

2.4.1.3 MOE

The input value for MOE can be replaced by compressive strength in AASHTOWare Pavement ME. This means that most of the sensitivity analysis performed such as Guclu et al. (2009) and Hall and Beam (2005) have compressive strength in place of MOE as in input. However, Schwartz et al. (2011) did use MOE and found it to be a "hypersensitive" input value during the OAT analysis.

2.4.1.4 Poisson's Ratio

According to Schwartz et al. (2011), Poisson's ratio was found to be a "very sensitive" input value during the OAT analysis. Hall and Beam (2005) found Poisson's ratio to be sensitive for cracking only. Guclu and Ceylan (2005) found Poisson's ratio to be a "low sensitive to insensitive" input value for faulting and found Poisson's ratio to be a "sensitive to very sensitive" input value for both transverse cracking and smoothness. Guclu et al. (2009) found Poisson's ratio to be "sensitive" for all three, faulting, cracking, and smoothness. Gulcu et al. (2009) also found that as the input value for Poisson's ratio increases the predicted damage of each distress increases.

2.4.2 Thermal Properties

Three thermal properties of concrete are utilized as MEPDG inputs. The first is CTE, which as discussed earlier, has been shown to be a very sensitive input value for rigid pavements, Mallela at al. (2005), and much research exists in this area (Gudimettla et al. 2012, McCarthy et al. 2014, Naik et al. 2011, Sakyi-Bekoe 2008, Shin and Chung 2011, Tanesi et al. 2010, Tran et al. 2008). The other two thermal properties, thermal conductivity (which describes heat flow) and heat capacity (which quantifies the amount of heat required to raise the temperature by unit increments) have not been the focus of many research studies on concrete pavements, and therefore far less information on the influence of these properties on pavement performance is available in the literature.

2.4.2.1 Coefficient of Thermal Expansion

According to Tanesi et al. (2007), "MEPDG is believed to be the first design approach to incorporate the CTE directly as an input parameter in the design of rigid pavements." Because concrete is a composite material, the CTE depends greatly on the concrete's composition. Concrete is composed of four main materials, cement, coarse aggregate, fine aggregate, and water. However, the quantities of each are changed to develop different characteristics of concrete. Aggregates account for 70% to 80% of the concrete volume and therefore have a significant influence on the CTE (Tanesi et al. 2007). However, with the paste makeup accounting for only 20% to 30% of the concrete volume, if the paste has a high enough CTE value it can influence the result of the CTE for the concrete (Tanesi et al. 2007). Published values of CTE for concrete in key references range from 4.1 to 7.3 x 10^{-6} in/in/°F (Neville 1995). Typically, "the higher the CTE value, the higher the effect of the test variability on the differences in predicted distresses and smoothness," as determined by Tanesi et al. (2007) who performed a sensitivity analysis in MEPDG using CTE values ranging from 3.6 to 8.1 x 10^{-6} in/in/°F.

Ley et al. (2013) performed CTE testing on concrete produced using nine different Oklahoma aggregates. Each aggregate was used in a set mixture only swapping out the aggregate. Seven of the nine different aggregate mixtures CTE values ranged from 5 x 10^{-6} in/in/°F to 5.45 x 10^{-6} in/in/°F. According to Ley et al. (2013) Oklahoma DOT could use 5.4 x 10^{-6} in/in/°F as an input value for MEPDG and have a representable value for all aggregates except for the remaining two, one of which had a CTE value of 4.5 x 10^{-6} in/in/°F and the other had a CTE value of 6.8 x 10^{-6} in/in/°F. It was also noted that the software should be compatible with the newer AASHTO T336 test method for CTE before CTE values are used.

Sakyi-Bekoe (2008) tested three different Alabama aggregates in concrete for a representative CTE value. A siliceous river gravel that had an average CTE value of 6.95 x 10^{-6} in/in/°F, a granite that had an average CTE value of 5.60 x 10^{-6} in/in/°F, and a dolomitic limestone that had an average CTE value of 5.52 x 10^{-6} in/in/°F. Both the granite and dolomitic limestone are slightly lower than the recommended input values of MEPDG.

According to Schwartz et al. (2011), CTE was found to be a "very sensitive" input value during the OAT analysis for faulting, transverse cracking, and IRI. Hall and Beam (2005) found CTE to be "sensitive" for faulting, cracking, and smoothness. CTE was found by Guclu and Ceylan (2005) to be a "sensitive to very sensitive" input value for faulting and found CTE to be an "extreme sensitivity" input value for both transverse cracking and smoothness. Guclu et al. (2009) found CTE to be "very sensitive" for faulting and cracking and "sensitive" for smoothness. Gulcu et al. (2009) also found that as the input value for CTE increases the predicted damage of each distress increases.

2.4.2.2 Thermal Conductivity

According to Schwartz et al. (2011), thermal conductivity was found to be a "hypersensitive" input value during the OAT analysis. Hall and Beam (2005) found thermal conductivity to be sensitive for cracking. Guclu and Ceylan (2005) found thermal conductivity to be a "sensitive to very sensitive" input value for faulting and found it to be an "extreme sensitivity" input value for both transverse cracking and smoothness. Guclu et al. (2009) found thermal conductivity to be "sensitive" for faulting and smoothness and "very sensitive" for cracking. Gulcu et al. (2009) also found that as the input value for thermal conductivity increases the predicted damage of each distress decreases.

2.4.2.3 Heat Capacity

As for heat capacity, Schwartz at al. (2011), Hall and Beam (2005), and Guclu et al. (2009) did not identify heat capacity as a sensitive input. It is identified in Guclu and Ceylan (2005) to be a "low sensitive to insensitive" input for transverse cracking but no mention is made by Guclu and Ceylan (2005) regarding the influence of heat capacity on joint faulting and smoothness.

2.5 Materials Used in North Carolina Pavements

This study was funded by NCDOT in order to develop a catalog of inputs for concrete mixtures typical of those used in construction of concrete pavements in several regions of North Carolina. North Carolina utilizes many different materials in concrete pavements throughout the state. North Carolina is typically divided into three regions

(Coastal, Mountain, and Piedmont), and there is variation in characteristics and engineering properties of materials occurring in each region. The following sections provide some background information on materials commonly utilized in North Carolina pavements along with information relevant to identification and use of materials-specific inputs in MEPDG. In Figure 2.1, a map of North Carolina is provided, which indicates the Mountain, Piedmont, and Coastal region boundaries as generally accepted.

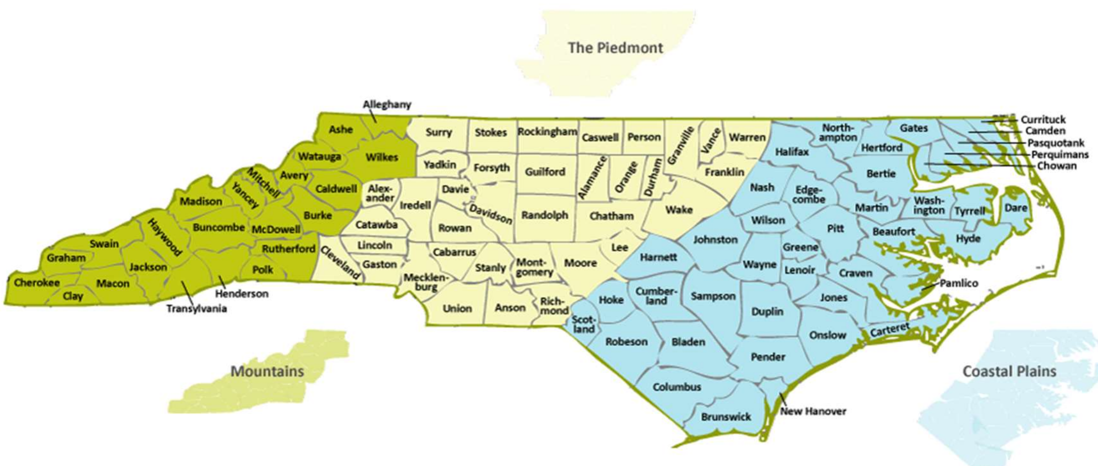


Figure 2.1: North Carolina regional boundary map (NCPedia 2016)

2.5.1 Cementitious Materials

North Carolina typically uses rigid pavement designs for heavily trafficked pavements such as interstates and other high traffic highways. Rigid pavements typically include PCC as a wearing surface. PCC is used throughout the world in many types of construction because it is a versatile material that can be produced virtually worldwide with locally available materials. Concrete consists of four main components: portland cement, coarse aggregate, fine aggregate, and water. Other materials such as supplementary cementitious materials (SCMs) and chemical admixtures can be added to improve the fresh and/or hardened performance of concrete, alter the properties, or improve the durability of the concrete.

2.5.1.1 Portland Cement

Ordinary portland cement (OPC) is the most commonly utilized cementitious material in North Carolina but NCDOT has recently changed its specifications to allow use of portland limestone cement (PLC). North Carolina currently does not have a cement manufacturing plant. At one time a cement plant existed in Castle Hayne, North Carolina. However, it closed in 1982. It is noted that a new company has recently shown interest in building a new plant at this location (Nunn 2011). Therefore, cement is supplied to North Carolina from other states, including Tennessee, South Carolina, and Virginia. Cement used in pavements is typically Type I or Type I/II OPC. OPC is a necessary material for the concrete industry. However, production of OPC is associated with a large carbon footprint. According to a survey by Portland Cement Association an average of 927 kg of CO_2 are emitted for every 1000 kg of portland cement (NRMCA 2012), and so other methods have been tested in the search of ways to reduce the need of OPC in order to achieve the required properties from the PCC.

2.5.1.2 Portland Limestone Cement

One method for reducing the carbon footprint of OPC is by producing and utilizing a PLC. PLC is produced by the addition of a limestone that is interground into the OPC at the clinker during manufacturing. PLC is a relatively new (to the US market) method of manufacturing portland cement concrete that helps reduces the carbon footprint and increase the sustainability of pavement infrastructure (Rupnow and Icenogle, 2014). By using additional interground limestone in portland cement, less clinker is produced, which reduces the amount of energy consumed and carbon dioxide emissions (Tennis et al. 2011). Similar achievements of the reduction in carbon dioxide are associated with use of other SCMs such as fly ash. However, limestone, as a key component in production of OPC clinker, is something readily available for each cement manufacturer, providing the possibility of obtaining its benefits without the associated transportation costs associated with inclusion of other SCMs in blended cements (Tennis et al. 2011).

According to Rupnow and Icenogle (2014) the mechanical properties and durability performance test results of concrete produced using a Type IL cement (a PLC) were very comparable to the performance of concrete produced using a Type I/II PCC. Concrete samples produced for Rupnow and Icenogle (2014) included test results performed on both fresh (slump, unit weight, air content, and set time) and hardened (compressive strength, flexural strength, surface resistivity, freeze-thaw durability, and shrinkage) concrete. Based on the findings of this study, the authors recommended standards and specifications for Louisiana be modified to allow Type I or Type II or Type IL portland cement for all structural and paving types.

2.5.1.3 Fly Ash

North Carolina has also allowed the use of secondary cementitious materials (SCMs) such as fly ash. Fly ash is used as an SCM in concrete because it can reduce cost, increase workability, and improve life-cycle performance (Teixeira et al. 2015). Fly ash is a byproduct of burning coal, and as a result of the combustion process is a very fine material. Fly ash is forced into the air during combustion and is caught in filters. When fly ash is collected, it must be managed and disposed of in holding ponds, land disposal, or beneficial reuse (EPA 2015). Beneficial reuse of fly ash in concrete is a very enticing method of “disposal,” as disposing of fly ash in holding ponds and land disposals is can be costly and has well-documented environmental impacts (Teixeira et al. 2015). Use of fly ash in concrete reduces the amount of portland cement required, as well as reducing the amount of fly ash that must be disposed of. Additionally, use of fly ash in concrete typically results in improved durability performance associated with lower permeability, decreased susceptibility to chemical attack, and other modes of degradation (Teixeira et al. 2015).

According to NCDOT (2012) a Class F fly ash can be used at a rate of 20% by weight replacement with 1.2 pounds of Class F fly ash per pound of cement replaced. However, a provision developed in 2015 states that 20% to 30% Class F fly ash replacement can be used at a rate of one pound of fly ash for one pound of cement (NCDOT 2015).

2.5.2 Aggregates

The types of aggregates available in North Carolina vary greatly by region of the state. This is evident in Figure 2.2, which shows the North Carolina geological survey from 1985 (NCDEQ 2015). A legend for the North Carolina geological survey is not included due to the font being scaled for printing on a much larger scale. However, information provided on the legend can be found through the same source as the map (NCDEQ 2015). With aggregates accounting for 70% to 80% of the total volume of concrete it is important to understand the properties of the aggregates, and ultimately their influence on the performance of the concrete (Tanesi et al. 2007). From Figure 2.2 the Coastal region of North Carolina consists mostly of sedimentary rocks such as limestone, sandstone, conglomerate, mudstone, sand, and clay. The Piedmont and Mountain regions are mainly comprised of intrusive rocks and sedimentary and metamorphic rocks. The intrusive rocks are primarily located in the Piedmont region with some overlay in the eastern part of the Mountain region where the Piedmont region borders the Mountain region. The intrusive rocks include granite, syenite, and gneiss. The sedimentary and metamorphic rocks are primarily located in the westernmost landscape of North Carolina's Mountain region. Some of the eastern-most Piedmont region includes some of these sedimentary and metamorphic rocks. The sedimentary and metamorphic rocks include sandstone, dolomite, shale, siltstone, schist, phyllite, marble, metavolcanic rock, quartzite, and slate.

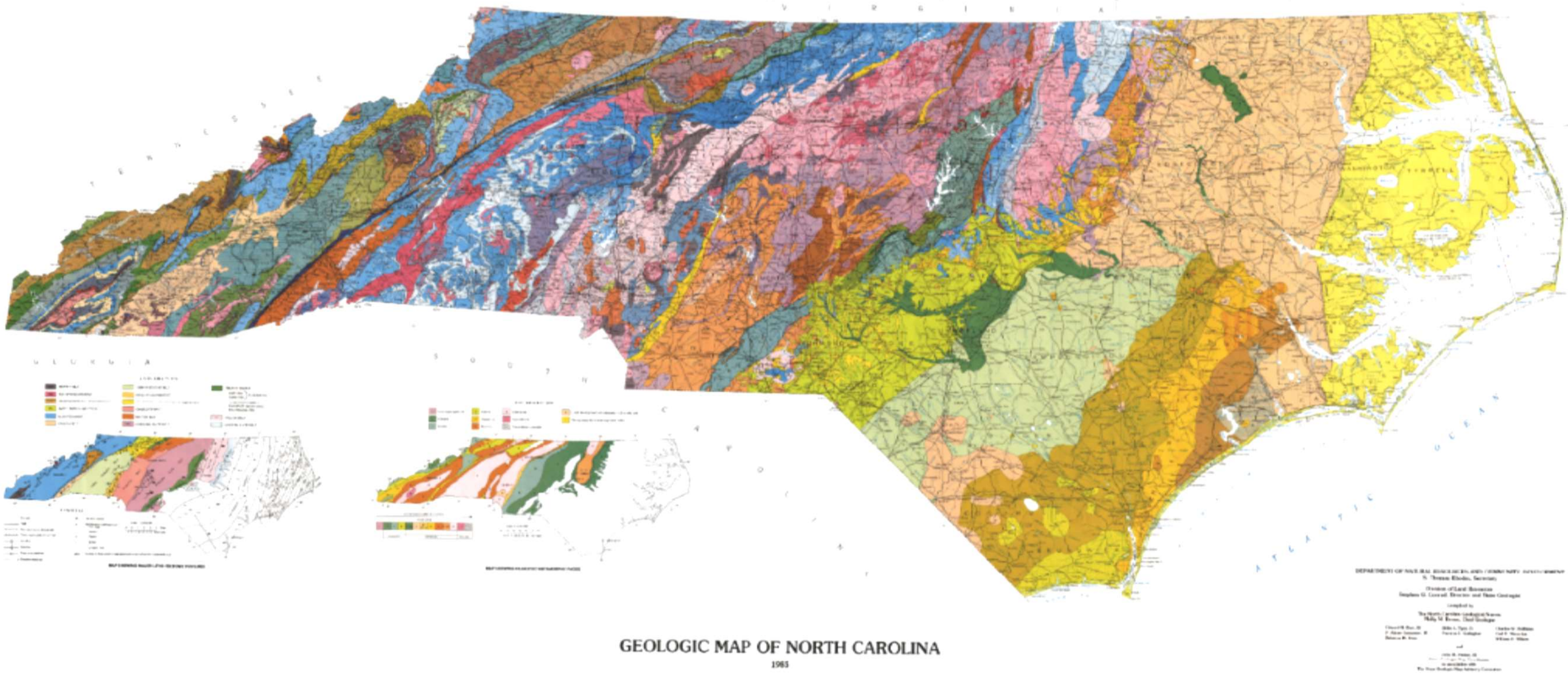


Figure 2.2: Geological survey of North Carolina (NCDEQ 2015)

2.6 Research Needs

AASHTO states that local calibration is necessary for optimal performance of AASHTOWare Pavement ME for the design and performance processes (AASHTO 2010). Although locally calibrated inputs for Asphalt pavements were determined as part of a national study (FHWA 2010), local calibration of concrete pavements has not been done for North Carolina. North Carolina is planning to construct (or reconstruct) more than 170 lane miles of rigid pavements over the next few years (Surti 2016). Ongoing and future projects include: 80 lane miles of rigid pavement for I-85 from Virginia state line southward to Henderson, 50 lane miles of rigid pavement on I-85 between Concord and Salisbury, 20 lane miles for a northern beltway around Winston-Salem, 20 lane miles widening US 52, a Greensboro outer loop, and rigid pavements used for a new I-40/I-77 interchange. A diverse range of materials (cement sources, aggregate types, manufactured sand, natural sand, etc.) is used in construction of rigid pavements in North Carolina, and an improved understanding of the performance of concrete incorporating these materials is needed to support use of MEPDG in North Carolina pavement analysis and design.

Using new locally calibrated inputs, the design of new pavements will be improved and predicted performances should be more reliable. Additionally, if new, locally calibrated inputs for concrete materials are found to differ from the global default values, the predicted performance of pavements already designed and constructed could deviate significantly from actual performance. Additionally, North Carolina has recently modified their standard specifications for roads and structures to allow PLC and NCDOT does not currently have performance data on concrete mixtures utilizing PLC. This information is also needed to support design of rigid pavements with PLC. Lastly, locally available sources of natural aggregates have been predicted to become more scarce or costlier, and an increased use of manufactured sand in pavement applications has been forecasted (Kumar and Niranjan 2003). NCDOT currently does not have data regarding the impact of the change from natural sand to manufactured sand (and blends of natural/manufactured sand) that can be used in pavement design and analysis.

CHAPTER 3: EXPERIMENTAL METHODOLOGY

In this research project, concrete pavement mixtures typical of those used for concrete pavement in North Carolina were batched and tested in order to develop a catalog of inputs for use in AASHTOWare Pavement ME design by NCDOT. Materials utilized in this study, including cements, fly ashes, and aggregates, were selected by NCDOT and project personnel based on the sources and characteristics of materials used for recently constructed concrete pavements, as well as those prospectively constructed in the future, in the three regions of North Carolina, the Piedmont, Coastal, and Mountain regions.

3.1 Concrete Mixtures

A concrete mixture design typical of use in North Carolina PCC pavements was identified, and the materials selected as representative materials for the state were utilized in this single mixture design in order to evaluate the changes in the PCC properties attributable to changes in the materials. This typical mixture has a water/cementitious material (w/c) ratio of 0.48. Eighteen different mixtures were batched and tested utilizing the mixture matrix in Figure 3.1. Three cements were selected for testing, two ordinary portland cements (OPC1 and OPC2) and one portland limestone cement (PLC). OPC2 and the PLC were manufactured from the same clinker to facilitate comparison of performance of the mixtures. Two fly ashes were also selected for use in some of the concrete mixtures, as NCDOT anticipates increased use of fly ash in concrete pavements in the future. Based on the known sensitivity of the Darwin Pavement ME predictions to concrete thermal inputs (particularly CTE) to the type of coarse aggregate utilized, coarse aggregate selection was a key project consideration. Three coarse aggregates were chosen for use in the study, one from each of the three regions in North Carolina (Coastal, Piedmont, and Mountain). For the fine aggregate, a manufactured sand obtained from the Piedmont region was utilized in the majority of mixtures (fifteen out of eighteen total), since NCDOT anticipates further reliance on manufactured sand in concrete utilized for highway pavements. A natural sand from the Piedmont region was utilized in three mixtures to facilitate comparison of performance to the concrete mixtures utilizing manufactured sand.

3.1.1 Development of Mixture Matrix

As shown in Figure 3.1, the mixture matrix used to support the experimental program consists of 18 concrete mixtures. The base mixture utilized a water/cement ratio of 0.48 and a manufactured sand as the fine aggregate. Variations on the base mixture design to explore the effects on mixture performance are designated by color in Figure 3.1. Standard mixtures that only vary by the source of the coarse aggregate utilized are shown in orange. To explore the effects of use of fly ash on the mixtures, as well as the effects of use of natural sands, a single coarse aggregate (from the Piedmont) was selected for use in other mixtures. Mixtures were prepared utilizing two different fly ashes at 20% replacement (designated in yellow and green). Mixtures in which the manufactured sand was replaced with natural sand are shown in blue.

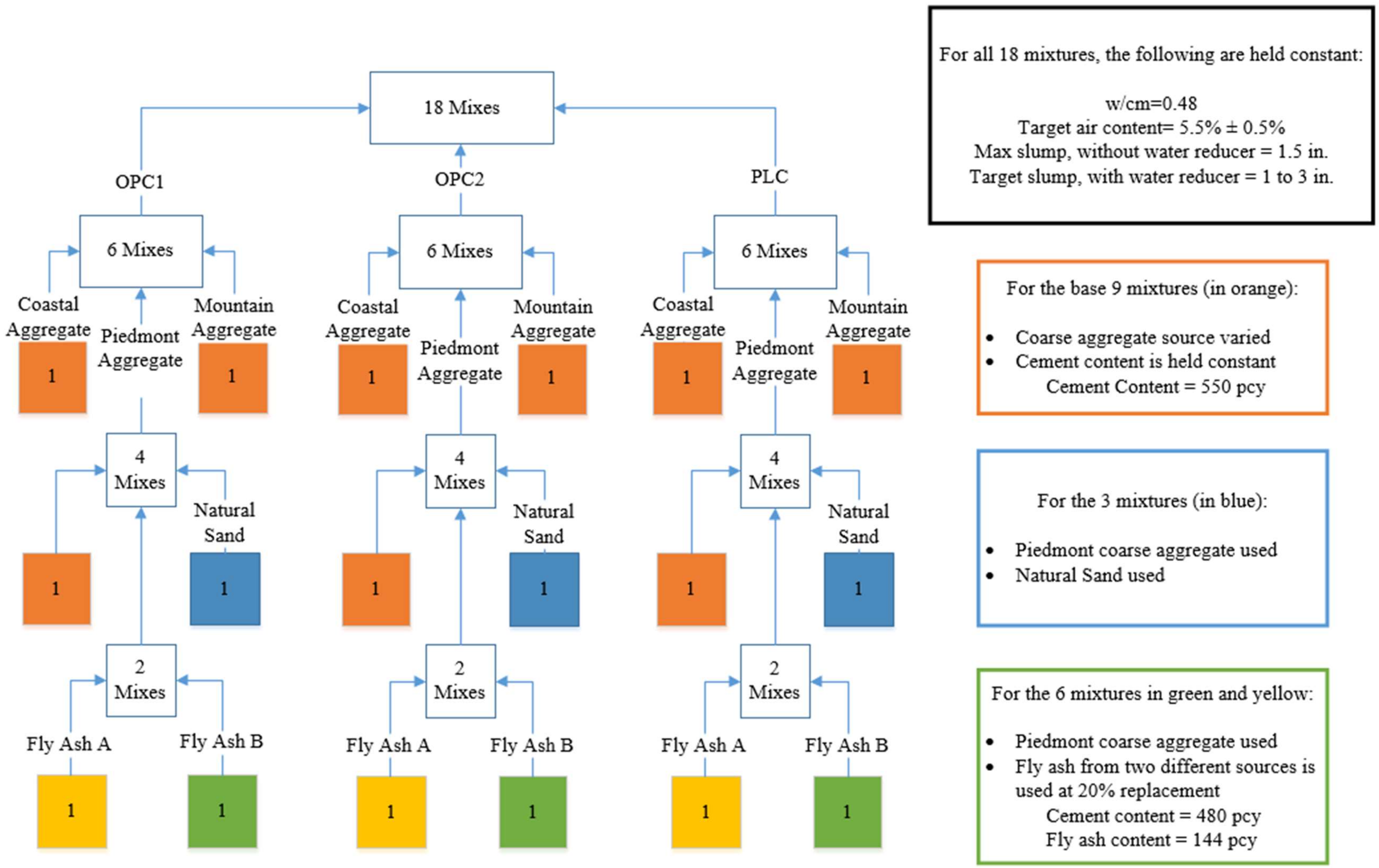


Figure 3.1: Mixture matrix

3.2 Materials Description and Characterization

The following sections provide additional details regarding the sources of the materials utilized in this study, along with relevant properties obtained in laboratory tests as part of this study or by the suppliers of the materials (as noted).

3.2.1 Cementitious Material

Several cementitious materials were used in this study: three cements and two supplementary cementitious materials (SCMs). As described earlier, two different OPCs, one portland limestone cement (PLC), and two different fly ashes were used in this research. A description of the source of each is provided below.

3.2.1.1 Portland Cement

Two different portland cements, OPC1 and OPC2, were used in the concrete mixtures batched in this research. OPC1 was produced by a manufacturing plant in Tennessee and was shipped to UNC Charlotte. This cement is a typically used cement for the Mountain region of North Carolina pavements and therefore chosen for this research study. OPC2 was produced and manufactured by a plant located in South Carolina. This cement is a commonly used cement for the Mountain and Piedmont regions of North Carolina. Mill reports for both of these cements are provided in Appendix A as Figures A.1 and A.2. Both OPC1 and OPC2 are Type I/II cements meeting ASTM C150, "Standard Specification of Portland Cement."

3.2.1.2 Portland Limestone Cement

The PLC used in the concrete mixtures batched in this research is a Type IL cement that was produced at the same Holly Hill, South Carolina plant as OPC2. The PLC was produced using the same clinker as the OPC2, with less than 15% limestone added per

ASTM C595, "Standard Specification for Blended Hydraulic Cements." The mill report for this PLC is provided as Figure A.2 in Appendix A.

3.2.1.3 Fly Ash

Several concrete mixtures in this research study utilized two different fly ash replacements. North Carolina allows for 20% replacement of cement by mass in accordance with North Carolina Standard Specifications section 1024, "Materials for Portland Cement Concrete." The replacement is at a rate of 1.2 pounds of class F fly ash per pound of cement replaced up to 20%. Both fly ashes, fly ash A and fly ash B, used in the concrete mixtures are classified as class F fly ash. Fly ash A was sourced from the Hyco power plant in Semora, North Carolina. Fly ash B was sourced from the Belews Creek power station in Belews Creek, North Carolina. Information for both fly ashes are provided as Figure A.3 and Figure A.4 in Appendix A.

3.2.2 Coarse Aggregates

As mentioned in the introduction, three coarse aggregates were selected for use in this study, one from the Piedmont region, one from the Mountain region, and one from the Coastal region of North Carolina. Quarries were selected based upon NCDOT personnel input regarding some of the most currently utilized aggregates for concrete mixtures currently (and forecasted to be) used in the state's concrete pavements. Each coarse aggregate was obtained at the selected quarries by the project team, placed into 55-gallon drums, and transported back to UNC Charlotte for storage until use. For the Mountain region the coarse aggregate (a granitic gneiss) was obtained from a quarry in the Asheville, North Carolina area. The coarse aggregate from the Piedmont region (also a granitic gneiss) was obtained from a quarry in the Raleigh, North Carolina area. The Coastal coarse aggregate (a coastal limestone) was collected from the Wilmington, North Carolina area. Table 3.1 provides detail on the identification of the coarse aggregates based on the 1985 North Carolina geological survey.

Table 3.1: Coarse aggregate identification from the 1985 geologic map of North Carolina (NCDEQ 2015)

Material/ Location	ID	Description
Coastal	Tec	Comfort Member and New Hanover Member, undivided Comfort Member: Bryozoan-echinoid skeletal limestone, locally dolomitized, solution cavities common New Hanover Member: Phosphate-pebble conglomerate, micritic, thin; restricted to basal part of Castle Hayne Formation in southeastern counties
Piedmont	CZbg	BIOTITE GNEISS AND SCHIST - Inequigranular and megacrystic; abundant potassic feldspar and garnet; interlayered and gradational with calc-silicate rock, sillimanite-mica schist, mica schist, and amphibolite Contains small masses of granitic rock
Mountain	Zatm	Muscovite-biotite gneiss - Locally sulfidic; interlayered and gradational with mica schist, minor amphibolite, and hornblende gneiss

Although characterization data on each aggregate was obtained from the producers, laboratory testing was performed at UNC Charlotte to verify that the physical properties of the aggregates obtained for the project were reasonably close to the values provided by the producers. These values were also used to obtain properties utilized to adjust the mixture design. These properties include gradation, density, specific gravity, absorption, and unit weight. For the aggregates, sieve analysis to characterize the aggregates' gradation was performed based upon ASTM C136, "Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates." Density, specific gravity, and absorption were performed according to ASTM C127, "Standard Test Method for Density, Relative Density (Specific Gravity), and Absorption of Coarse Aggregate." Unit weight was performed according to

ASTM C29, "Standard Test Method for Bulk Density ("Unit Weight") and Voids in Aggregate." The marine limestone aggregate selected from the Coastal region has a specific gravity of 2.42 and absorption of 2.4%. The granitic aggregate selected to represent the Piedmont region has a specific gravity of 2.62 and an absorption of 0.8%. The aggregate selected to represent the Mountain region, also a granite, has a specific gravity of 2.62 and an absorption of 1.1%. The results of the sieve analysis for each aggregate are presented in Appendix A as Tables A.1, A.2, and A.3.

3.2.3 Fine Aggregates

With a key focus of this study being evaluation of the impact of coarse aggregate type on concrete properties a limited number of fine aggregates utilized in the testing program. Fine aggregates were selected from the Piedmont region due to their central location in the state and the fact that much of the concrete pavement constructed in North Carolina has been in the Piedmont region. The two fine aggregates used in this study (one manufactured sand and one natural sand) were obtained by the project team from two Piedmont region quarries and put into two cubic yard hoppers for transportation and storage until testing and use. Sieve analysis to characterize the fine aggregates' gradation and determine the fineness modulus was performed based upon ASTM C136, "Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates." Density, specific gravity, and absorption were performed according to ASTM C128, "Standard Test Method for Density, Relative Density (Specific Gravity), and Absorption of Fine Aggregate."

The primary fine aggregate chosen for this research, used in fifteen of the eighteen mixtures, is a manufactured sand that corresponds to NCDOT's 2MS specification, obtained from the Charlotte, North Carolina area. This manufactured sand has a specific gravity of 2.65, an absorption of 0.3%, and a fineness modulus of 2.54. An alternative fine aggregate, used in the remaining three mixtures, is a natural sand that was obtained from a pit in the Charlotte, North Carolina area and meets the requirements of ASTM C33, "Standard Specification for Concrete Aggregates." The natural sand has a specific gravity of 2.64, an absorption of 0.74, and a fineness modulus of 2.54. Table 3.2 provides detail on the identification of the fine aggregates based on the 1985 North Carolina geological survey. A gradation curve for both aggregates can be found in the Appendix as Tables A.4 and A.5.

Table 3.2: Fine aggregate identification from the 1985 geological map of North Carolina (NCDEQ 2015)

Material/ Location	**ID**	**Description**
Natural Sand	Km	MIDDENDORF FORMATION - Sand, sandstone, and mudstone, gray to pale with an orange cast, mottled; clay balls and iron-cemented concentrations common, beds laterally discontinous, cross-bedding common
Manufactured Sand	PzZq	METAMORPHOSED MAFIC ROCK - Foliated to massive

3.2.4 Admixtures

Two commercially available admixtures were used in this study, an air entraining admixture and a mid-range water-reducing admixture. Since the concrete mixtures in this study are pavement mixtures used in slipform pavers, the target slump was 1.5 inches, although some reasonable range of slump variation was anticipated as it was deemed important to maintain a consistent w/c ratio between different mixtures and between batches of the same mixture. Although NCDOT specifications allow an air content of (5.0% plus or minus 1.5%), a relatively tight allowable air content tolerance of 5.0% to

6.0% was utilized for all batches in order to ensure consistency between test results and to ensure that differences in laboratory test results could be mostly attributed to changes in materials, rather than changes in air content.

The air entraining admixture used in each of the concrete mixtures in this study was MasterAir AE 200 manufactured by BASF. For this product, the manufacturer recommends using a dosage of 0.125 to 1.5 fluid oz/cwt. In order to achieve an air content of 5.0% to 6.0%, the actual dosage in the concrete mixtures ranged between 0.48 fluid oz/cwt (typical in some of the natural sand mixes that used no mid-range water-reducer) to 12.6 fluid oz/cwt (typical in some of the mixes where the fly ash added workability but had negative effects on the air entrainment).

All of the concrete mixtures utilizing the manufactured sand required use of a mid-range water-reducing admixture. MasterPolyheed 997 manufactured by BASF was selected for use in the study. For this product, the manufacturer recommends using a dosage of 3 to 15 fluid oz/cwt. In order to achieve a slump value of approximately 1.5 inches, the actual dosage in the concrete mixtures ranged from 3.9 fluid oz/cwt (typical in the fly ash mixtures where the fly ash had provided added workability) to 17.3 fluid oz/cwt (typical in the coastal aggregate mixtures where the mixture had reduced workability). No mid-range water-reducer was used in the natural sand mixtures due to workability in excess of the target slump being obtained utilizing the specified w/c ratio of 0.48.

3.3 Testing Program

The overall testing program for this project is shown in Table 3.3. It is noted that although a number of tests were performed as part of this work, only those utilized in the analytical portion of this thesis are discussed here. Information on other tests will be presented in the project report and in other publications (Chimmula 2016, Medlin 2016).

Table 3.3: Testing program

	Test	Protocol	Age(s) in days	Replicates
Fresh	Air content	Pressure meter (ASTM C231) and Super air meter (developer protocol)	Fresh	1 each type of test, each batch (2 each total)
	Slump	ASTM C143	Fresh	1
	Fresh density (unit weight)	ASTM C138	Fresh	1
	Temperature	AASHTO T309	Fresh	1
Hardened	Compressive strength	ASTM C39	3, 7, 28, 90	3 each age
	Resistivity	AASHTO TP95-11	3, 7, 28, 90	3 each age
	Modulus of rupture	ASTM C78	28	2
	Modulus of elasticity and Poisson's ratio	ASTM C469	28	2
	Coefficient of thermal expansion	AASHTO T336	28	3
	Heat capacity	ASTM C2766	56	3
	Thermal conductivity	ASTM E1952	56	3
	Shrinkage	ASTM C157	per standard	3
	Cracking potential	ASTM C1581	per standard	3
	Rapid chloride permeability	ASTM C1202	28	2
	Freezing and thawing resistance	ASTM C666, procedure A	per standard	3
	Thaumasite attack	CSA A3004-C5	per standard	6

3.4 Batching and Mixing Procedure

For all of the testing shown in Table 3.3 to be performed, a sizeable amount of concrete was required in order to make all of the test specimens. However, laboratory equipment limitations had to be considered, since each batch of concrete was prepared utilizing a six cubic foot portable concrete mixer. To ensure uniform and consistent mixing, the total amount of concrete needed for each mixture was broken into four batches of approximately 2.3 cubic feet. Each batch was mixed in accordance with ASTM C685, “Standard Specification for Concrete Made by Volumetric Batching and Continuous Mixing.”

Batch one of each mixture was utilized to prepare four 4” x 8” cylinders for rapid chloride permeability testing, three cracking rings for ASTM C1581 testing, and four cylinders for compressive strength testing per ASTM C39 (to evaluate conformity with the other three batches of the same mixture). Batch two was utilized to prepare four 4” x 8” cylinders for hardened air void analysis, provide fresh concrete for two super air meter (SAM) tests, and three freeze/thaw test (ASTM C666) beams, and four 4” x 8” cylinders for (conformity) compressive strength testing. Batch three was utilized to prepare twelve 4” x 8” cylinders for 3, 7, 28, and 90 day compressive strength tests, one 4” x 8” cylinder for thermal conductivity and heat capacity tests (ASTM C518), two 6” x 12” cylinders for MOE and Poisson’s ratio testing (ASTM C496), and three beams for testing for susceptibility to drying shrinkage (ASTM C157). Batch 4 was utilized to cast two beam specimens for MOR testing (ASTM C78), three 4” x 8” cylinders for coefficient of thermal expansion, and four 4” x 8” cylinders for compressive strength testing (conformity).

3.5 Testing of Fresh Concrete Properties

Each batch of concrete was tested when it was in the fresh state to determine several key properties and to ensure consistency between batches. For each concrete batch, slump, air content, fresh density, and temperature were measured. Additionally, the SAM number was determined using the Super Air Meter (SAM) on a number of batches.

3.5.1 Slump

To ensure slumps were consistent with pavement concrete, the target slump for each batch was 1.5 in. Slump tests were performed on each batch to confirm the batch met (or was reasonably close to) the target slump. Slump tests of each batch also provided a simple quality control check during the mixing of the concrete, ensuring that consistency was maintained between batches. Slump was performed according to ASTM C143, "Standard Test Method for Slump of Hydraulic-Cement Concrete." As stated previously, the target slumps of concrete mixtures batched as part of this work was 1.5 inches. Since the goals of this research involve identification of changes in mechanical and thermal properties of concrete associated with changes in materials, it was important to maintain a consistent w/c ratio. As can be expected when exchanging materials in a constrained mixture design, some deviation from slump was observed in a number of mixtures, and water reducing admixture was adjusted accordingly to obtain a slump reasonable of paving concrete (1 to 2 inches). Changing the base mixture to utilize natural fine aggregate resulted in a slump greater than 2 inches, but to accomplish the objectives of the research, no adjustment was made to the w/c.

3.5.2 Air Content

Providing adequate entrained air in concrete is important in order to ensure that the concrete can resist adequately resist freeze-thaw stresses. Typically, adequate air entrainment is between 5% and 8% (Neville 2011). However as mentioned previously, to ensure the best likelihood that differences in mechanical and thermal properties attributable to changes in materials could be discerned, the total air content of each batch was restricted to the relatively tight range of 5.0% to 6.0% by adjusting the air entraining admixture dosages. For all batches, air content testing was performed in accordance with ASTM C231, "Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method" in order to obtain the total air content of the fresh concrete. Testing using the Super Air Meter (SAM) as well as the Type B air meter was performed on Batch 2, as it was the batch utilized for freeze-thaw testing. An AASHTO Provisional Standard for the SAM has been approved but for this study, the SAM was performed according to the manufacturer's specifications (Super Air Meter 2015).

3.5.3 Unit Weight

Fresh density is another way of performing a quality control check on the conformity of batch to batch by making sure the unit weight remains consistent. Fresh density can also be used to provide early warning signs that air content may be too high or low, or that material properties may have changed. Fresh density was performed with the same equipment as the air content using the pressure method, utilizing a container of known volume. Fresh density was performed according to ASTM C138, "Standard Test Method for Density (Unit Weight), Yield, and Air Content (Gravimetric) of Concrete."

3.6 Preparation and Curing of Test Specimens

After batching, test specimens were prepared in accordance with the appropriate test standards. Specimens were produced following the procedures from ASTM C192, "Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory." Although multiple individuals were involved in batching and preparation of test specimens, to ensure consistency, each member of the project team was responsible for preparation of the same set of specimens (e.g., compressive strength cylinders, freeze-thaw testing beams, etc.) for each mixture.

3.7 Testing of Hardened Concrete

Tests of hardened properties of the concrete were performed as shown in Table 3.3. Inputs to AASHTOWare Pavement ME include mechanical properties and thermal properties of concrete. The mechanical properties determined for concrete batched as part of this study include compressive strength, MOR, MOE, and Poisson's ratio. The thermal properties include coefficient of thermal expansion (CTE), heat capacity, and thermal conductivity. Durability tests of surface resistivity, length change, cracking potential, rapid chloride permeability, freeze and thaw resistance, and thaumasite attack were also performed, but are the focus of other publications developed from this project (Chimmula 2016, Cavalline et al. 2016).

3.7.1 Mechanical Properties

The mechanical properties of concrete found by other researchers to be sensitive MEPDG inputs are shown in Table 2.1 in Chapter 2. For this research, the following mechanical properties of each of the eighteen batches of concrete were determined in order to identify locally appropriate values for use in North Carolina MEPDG pavement design, as well as to evaluate the sensitivity of MEPDG to each of these inputs and to assist with completing North Carolina's local calibration of MEPDG for PCC pavements.

3.7.1.1 Compressive Strength

Compressive strength tests were performed on 4" x 8" cylinders on days 3, 7, 28, and 90 following the mixture date for batch 3, and to monitor conformity, on days 7 and 28 following the mixture date for all other batches. The compressive strength tests were performed according to ASTM C39, "Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens." According to NCDOT (2012) roadway standard specifications in section 1000-3, "portland cement concrete for pavement," a minimum compressive strength of 4,500 psi at 28 days is required.

3.7.1.2 Modulus of Rupture

The MOR is used in measuring compliance with NCDOT specifications as well as the tensile strength of the concrete. The MOR testing was performed when specimens are at 28 days of age, in accordance with ASTM C78, "Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)." According to NCDOT (2012) roadway standard specifications in section 1000-3, "Portland Cement Concrete for Pavement", a minimum flexural strength of 650 psi at 28 days is required for concrete mixtures used in pavement applications.

3.7.1.3 Modulus of Elasticity and Poisson's Ratio

Tests to determine the MOE and Poisson's ratio were performed on specimens at 28 days of age. The MOE and Poisson's ratio are performed on the same specimen according to ASTM C469, "Standard Test Method for Static MOE and Poisson's Ratio of Concrete Compression."

3.7.2 Thermal Properties

The thermal properties, especially CTE and thermal conductivity, of concrete have been found to be very sensitive when it comes to the input of MEPDG as shown in Table 2.1 in Chapter 2. For this research, the following thermal properties of each of the eighteen batches of concrete were determined in order to identify locally appropriate values for use in North Carolina MEPDG pavement design, as well as to evaluate the sensitivity of MEPDG to each of these inputs and to assist with completing North Carolina's local calibration of MEPDG for PCC pavements.

3.7.2.1 Coefficient of Thermal Expansion

The CTE test was performed at 28 days in accordance with AASHTO T336. "Standard Method of Test for Coefficient of Thermal Expansion of Hydraulic Cement Concrete." The test was performed using a Pine AFCT2 coefficient of thermal expansion of concrete measurement system. This piece of equipment has three frames allowing three specimens to be tested at the same time. As per AASHTO T336, the temperature of the water bath ranges from 10°C to 50°C four times in order to provide averaged results that are checked amongst each other for consistency. For each mixture, three 4" x 8" cylinders cut to a length of approximately 7 in were used for the test. The three results from tests of the three specimens were averaged to get a representative CTE value for the mixture. Validation and calibration of the equipment was performed prior to testing per the AASHTO T336 standard.

3.7.2.2 Thermal Conductivity

Thermal conductivity tests were performed at 56 days after the mixture date using the Fox50 Heat Flow Meter Instrument by Laser Comp. Of note, the Fox50 test equipment provides an advantage for this research project in that it can test bulk specimens. To minimize variation and to provide consistency, thermal conductivity testing was performed on the same specimens as heat capacity testing. Three specimens were tested for each mixture, and specimens were prepared from a 4" x 8" cylinder. Three representative rectangular prisms approximately 1.5 inch x 1.5 inch x 1 inch thick were saw cut from the cylinder seven days before the test date. Care was taken during saw cutting to ensure that each of the three specimens did not contain entrapped air voids and represented the mixture composition (aggregates were well distributed within the paste). To ensure a consistent moisture content in each specimen, the three specimens were placed into an environmental chamber set at 72°F and 50% relative humidity for seven days prior to testing. The thermal conductivity tests were performed using the Laser Comp Fox50 in accordance to ASTM C518, "Standard Test Method for Steady-State Thermal Transmission Properties by Means of the Heat Flow Meter Apparatus."

The Fox50 test apparatus utilizes software to control calibration, and the calibration sequence was run prior to each day's tests. Calibration was also confirmed using a manufacturer supplied reference sample (Pyrex) during each series of tests with the equipment. Manufacturer supplied pads are used to cushion the specimens in the Fox 50 chamber and to ensure optimal contact for the heating elements and sensors. Specimens were tested one at a time, with the thermal conductivity test program run first, followed by heat capacity testing. Test values were corrected for the cushioning pads, as directed by the manufacturer. The thickness of the specimen is computed by the Fox50 apparatus, and variability in specimen thickness is therefore considered in the test result.

3.7.2.3 Heat Capacity

As discussed in Section 3.7.2.2, heat capacity tests were performed at 56 days after the mixture date on the same specimens tested for thermal conductivity using the Fox50 Heat Flow Meter Instrument by Laser Comp. Again it is noted that this testing was performed on bulk specimens of concrete sawcut from cylinder specimens. Other equipment (such as a differential scanning calorimeter, or DSC) often utilized to determine the heat capacity of a material typically utilizes very small sample sizes on the order of 270 microliters. Often for composite building materials, these types of testing equipment require crushing building materials to a powder and pouring into a crucible, compromising the porous microstructure of the material, as well as introducing variability and error into the measurements.

Preparation and conditioning of the test specimens is as outlined above in 3.7.2.2. The heat capacity tests were performed using the Laser Comp Fox50 in accordance to ASTM C518, “Standard Test Method for Steady-State Thermal Transmission Properties by Means of the Heat Flow Meter Apparatus.”

CHAPTER 4: RESULTS OF LABORATORY TESTING

This chapter provides a summary of results of tests performed as part of this study to support identification of MEPDG inputs. As stated previously in this thesis, other tests (such as tests to evaluate the durability performance of the mixtures), were performed as part of this project. However, they are the subject of other theses developed as part of this work, and are published elsewhere (Medlin 2016, Chimmula 2016).

Mixtures batched for this work were coded in order to support identification of the key mixture components. A breakdown of the identification code for each mixture (as shown in Table 4.1) is as follows:

- The first letter represents the type of coarse aggregate, C indicates the Coastal source of coarse aggregate, P is Piedmont source of coarse aggregate, and M indicates the Mountain source of coarse aggregate.
- The second letter represents the type of cement used in the mixture: A indicates OPC1, B indicates OPC2, and BL indicates the mixtures that contain a PLC that is from the same clinker as OPC2.
- The third letter identifies if there is fly ash, N indicates that no fly ash was used, A indicates a 20 percent replacement of fly ash from Source A, B indicates a 20 percent replacement of the second fly ash used in this project.
- The final letter identifies the type of fine aggregate utilized in the mixture: M indicates the manufactured sand and N indicates the natural sand.

Table 4.1: Mixture designation

Code for Mixture Designations			
First Letter Aggregate	Second Letter Cement	Third Letter Flyash	Fourth Letter Sand
C=Coastal	A=OPC1	N=None	M=Manufactured
P=Piedmont	B=OPC2	A=20% of A	N=Natural
M=Mountain	BL=PLC	B=20% of B	

4.1 Testing of Fresh Concrete Properties

This section provides the results for the fresh concrete properties such as slump, air content, and unit weight. Each test was performed on every batch to ensure consistency in batching as well as conformity of each batch to the target values (slump, air) for the mixture. The target for slump is 1.5 inches and the air content must fall within a range of 5.0% to 6.0%. As stated in section 3.4, these relatively low and tight restraints are set to ensure the change in the results is from the change in materials instead of the fluctuation between batches. Table 4.3 provides a compiled list of the results of the fresh properties which are discussed in detail in the following sections. In order to maintain the slump and air content required for this study, the dosage rates may have been varied slightly based on external influences, such as temperature or minor fluctuations in material properties.

Table 4.2: Compiled fresh concrete properties

Designation	Slump (in)	Air Content (%)	Unit Weight (PCF)
P.A.N.M	1.4	5.4	145
P.B.N.M	1.9	6.0	143
P.BL.N.M	2.2	5.6	144
C.A.N.M	1.1	5.8	138
C.B.N.M	1.4	5.6	139
C.BL.N.M	1.1	5.5	139
M.A.N.M	2.0	5.3	145
M.B.N.M	2.4	5.4	144
M.BL.N.M	2.3	5.1	145
P.A.A.M	2.7	5.7	141
P.B.A.M	2.3	5.2	142
P.BL.A.M	2.5	5.2	142
P.A.B.M	2.4	5.6	142
P.B.B.M	2.3	5.7	141
P.BL.B.M	2.3	5.6	141
P.A.N.N	1.9	5.3	143
P.B.N.N	3.3	5.4	142
P.BL.N.N	2.8	5.5	143

4.1.1 Slump

Test results for slump are shown in Table 4.2. As can be expected, the different materials comprising each batch warranted use of different dosages of water reducing admixture to reach the desired slump. The mixtures utilizing the manufactured sand (P.A.N.M, P.B.N.M, P.BL.N.M, C.A.N.M, C.B.N.M, C.BL.N.M, M.A.N.M, M.B.N.M, and M.BL.N.M) required the most midrange water reducing admixture (13.8 to17.3 fluid oz/cwt) in order to achieve a slump within the desired range. The mixtures using a piedmont aggregate and fly ash (P.A.A.M, P.B.A.M, P.BL.A.M, P.A.B.M, P.B.B.M, and P.BL.B.M) were found to be much more workable and used less (4.0 fluid oz/cwt) midrange water reducer in order to achieve a slump with in the desired range. The mixtures using a piedmont coarse aggregate and a natural sand (P.A.N.N, P.B.N.N, and P.BL.N.N) did not use any midrange water reducer due to the fact of the increased workability in excess of the target slump in some cases using the natural sand and maintaining a w/c ratio of 0.48. In general, the manufactured sand required a lot more mid-range water reducer to achieve the required workability for this testing than the natural sand mixtures.

4.1.2 Air Content

Results for testing for total air content (using the Type B air meter) are provided in Table 4.2. As can be expected, it was found that the changes in types of materials resulted in required changes in the air entraining admixture dosage. As cement and SCM characteristics have been found to influence the performance of air entraining admixtures, it was anticipated that a number of trial batches would be required to identify the air entraining admixture dosage required for each mixture in order to obtain the very tight range of acceptable total air contents, 5.0% to 6.0%. It was found that an increase in the required midrange water reducing admixture increased the amount of air entraining admixture required. A similar trend can also be seen when natural sand is used in mixtures instead of the manufactured sand. The added workability facilitated by the more rounded natural sand resulted in no midrange water reducer required. Therefore, the amount of air entraining admixture required for the natural sand mixtures was very low compared to the mixtures containing manufactured sand. Another influencing factor was fly ash. It was found that fly ash increased the amount of air entraining admixture required to reach the specified air content.

4.1.3 Unit Weight

Results for unit weight are also provided in Table 4.2. It should be noted that typically the Piedmont and Mountain aggregate had unit weights that were comparable. The mixtures containing the Coastal coarse aggregate (C.A.N.M, C.B.N.M, and C.BL.N.M) had lower unit weights, because this coarse aggregate is more porous and has a lower specific gravity than the other coarse aggregates used in this study. Mixtures containing fly ash (P.A.A.M, P.B.A.M, P.BL.A.M, P.A.B.M, P.B.B.M, and P.BL.B.M) and natural sand (P.A.N.N, P.B.N.N, and P.BL.N.N) also had a lower unit weight compared to the base mixtures utilizing Piedmont aggregate (P.A.N.M, P.B.N.M, and P.BL.N.M).

4.2 Testing of Hardened Concrete

This section provides the results from the testing of the hardened concrete, and is divided into sections that detail the results of mechanical property tests and thermal property tests. The mechanical property test results discussed in this research study include compressive strength, MOR, MOE, and Poisson's ratio. The thermal property results discussed in the research study include CTE, thermal conductivity, and heat capacity.

4.2.1 Mechanical Properties

The compressive strength, MOR, MOE, and Poisson's ratio are discussed in the subsequent sections. In Table 4.3, a summary of the mechanical property test results for all of the eighteen mixtures is provided. Detailed discussion on the results of each of the tests is provided in subsequent sections. Information on the variability of each of these test results (including standard deviation) is provided in Appendix B in Tables B.4 through B.7.

Table 4.3: Compiled mechanical property testing results

Designation	Compressive Strength (psi)				MOE (psi)	Poisson's Ratio	MOR (psi)
	3 Day	7 Day	28 Day	90 Day			
P.A.N.M	3,370	4,020	5,020	5,230	2,920,000	0.20	680
P.B.N.M	3,660	3,960	4,850	5,500	3,340,000	0.20	670
P.BL.N.M	3,720	4,340	5,020	6,170	2,430,000	0.18	660
C.A.N.M	3,650	4,890	5,360	6,010	3,730,000	0.22	730
C.B.N.M	4,340	4,770	5,960	5,690	3,490,000	0.21	750
C.BL.N.M	4,290	4,850	5,560	5,610	3,690,000	0.22	680
M.A.N.M	3,060	3,930	5,030	5,530	2,540,000	0.18	570
M.B.N.M	3,800	4,130	5,100	5,390	2,760,000	0.20	640
M.BL.N.M	3,670	4,130	4,790	5,530	3,020,000	0.20	610
P.A.A.M	2,620	3,550	4,270	5,560	3,220,000	0.23	650
P.B.A.M	2,460	3,050	4,050	4,380	2,700,000	0.21	540
P.BL.A.M	2,210	2,960	3,750	4,620	2,690,000	0.16	650
P.A.B.M	2,130	2,390	3,780	5,490	2,840,000	0.22	570
P.B.B.M	2,040	2,410	3,140	4,340	2,510,000	0.18	620
P.BL.B.M	2,330	2,500	3,780	4,370	2,720,000	0.19	560
P.A.N.N	2,720	4,080	5,400	6,060	3,400,000	0.15	740
P.B.N.N	3,010	3,420	4,390	5,450	3,510,000	0.19	720
P.BL.N.N	3,270	3,930	5,190	5,800	3,040,000	0.15	750

4.2.1.1 Compressive Strength

Compressive strength test results for each mixture (at ages of 3, 7, 28, and 90 days) are shown in Table 4.3. Additional data, including the ranges and standard deviations of the test results are provided in Appendix B in Table B.4. According to the NCDOT Standard Specifications (2012), a minimum 28-day compressive strength of 4,500 psi is required for concrete used in pavement applications. All of the base mixtures (P.A.N.M, P.B.N.M, P.BL.N.M, C.A.N.M, C.B.N.M, C.BL.N.M, M.A.N.M, M.B.N.M, and M.BL.N.M) met this requirement. Since the hydration of fly ash occurs at later ages, and the cement content and w/c ratio was kept constant throughout all mixtures, the fly ash mixtures (P.A.A.M, P.B.A.M, P.BL.A.M, P.A.B.M, P.B.B.M, and P.B.BL.M) did not reach the NCDOT specification. However, it should be noted that the fly ash mixtures with cement A did end up reaching the requirement by day 90. The natural sand mixtures (P.A.N.N, P.B.N.N, and P.BL.N.N) had one outlier that did not reach the required specification (P.B.N.N) at 28 day. Ultimately, in a production setting, modifications to the mixture proportions (including use of high-range water reducers) would be performed to achieve the specified strengths. However, as the goal of this research project was to elucidate the effects of different materials on the same base mixture, and some deviation from specified was anticipated as an artifact of this research approach.

4.2.1.2 Modulus of Rupture

Results from MOR testing are provided in Table 4.3. Additional data, including the ranges and standard deviations of the test results are provided in Appendix B in Table B.5. According to the NCDOT Standard Specifications (2012), a minimum of 650 psi is required to be met for 28 MOR. The base piedmont mixtures (P.A.N.M, P.B.N.M, and P.BL.N.M) and coastal mixtures (C.A.N.M, C.B.N.M, and C.BL.N.M) as well as the piedmont with natural sand mixtures (P.A.N.N, P.B.N.N, and P.BL.N.N) met the requirement. The rest of the mixtures, mountain (M.A.N.M, M.B.N.M, and M.BL.N.M) and both fly ash mixes (P.A.A.M, P.B.A.M, P.BL.A.M, P.A.B.M, P.B.B.M, and P.BL.B.M) fell short of the requirement or just barely met the requirement. Typically, a mixture consisting of the Coastal coarse aggregate had higher values for MOR and mixtures consisting of Mountain coarse aggregate had the lowest results for MOR.

4.2.1.3 Modulus of Elasticity

Results for MOE testing are provided in Table 4.3. Additional data, including the ranges and standard deviations of the test results are provided in Appendix B in Table B.6. As different materials were utilized in each mixture, some fluctuation occurred. However, 28-day MOE values typically remained within a reasonably tight range of 2,430,000 psi to 3,730,000 psi, regardless of the different materials that were used in this study. This range is within the marginal to adequate MOE values provided in AASHTO 2015. The coastal coarse aggregate mixtures (C.A.N.M, C.B.N.M, and C.BL.N.M) and the piedmont coarse aggregate mixtures with the natural sand (P.A.N.N, P.B.N.N, and P.BL.N.N) were found to have the largest moduli of elasticity (3,040,000 psi to 3,730,000 psi). The remaining mixtures consisting of piedmont coarse aggregate (P.A.N.M, P.B.N.M, and P.BL.N.M), mountain coarse aggregate (M.A.N.M, M.B.N.M, and M.BL.N.M), and both fly ash mixtures (P.A.A.M, P.B.A.M, P.BL.A.M, P.A.B.M, P.B.B.M, and P.B.BL.M) contain the lower moduli of elasticity (2,430,000 psi to 3,340,000 psi).

4.2.1.4 Poisson’s Ratio

Test results for Poisson’s ration are provided in Table 4.3. Additional data, including the ranges and standard deviations of the test results are provided in Appendix B in Table B.7. Poisson’s ratio values for mixtures batched in this study ranged from 0.15 to 0.23. According to AASHTO 2015, typical Poisson’s ratios range from 0.10 to 0.21. The range of Poisson’s ratios for the Base mixtures with Piedmont coarse aggregate and manufactured sand (P.A.N.M, P.B.N.M, and P.BL.N.M) and Mountain coarse aggregate (M.A.N.M, M.B.N.M, and M.BL.N.M) is between 0.18 and 0.20. The Coastal aggregate mixtures (C.A.N.M, C.B.N.M, and C.BL.N.M) had Poisson’s ratios between 0.21 and 0.22.

The fly ash mixtures (P.A.A.M, P.B.A.M, P.BL.A.M, P.A.B.M, P.B.B.M, and P.B.BL.M) do not seem to have a trend, since the range for test results for these mixtures is between 0.16 and 0.23. The Piedmont coarse aggregate and natural sand mixtures (P.A.N.N, P.B.N.N, and P.BL.N.N) exhibited some of the lower Poisson's ratios, between 0.15 and 0.19.

4.2.2 Thermal Properties

This section provides the results for the thermal properties tested in the hardened state. The CTE, thermal conductivity, and heat capacity are discussed in more detail in the subsequent sections. Table 4.4 provides a compiled list of the thermal property results of the hardened concrete testing. Additional data, including the ranges and standard deviations of the test results are provided in Appendix B in Tables B.8 to B.10.

Table 4.4: Compiled thermal property testing results

Designation	CTE (in/in °F)	Heat Capacity (Btu/lb-°F)	Thermal Conductivity (Btu/(ft)(hr)(°F))
P.A.N.M	4.57 × 10-6	0.20	0.92
P.B.N.M	4.63 × 10-6	0.20	0.95
P.BL.N.M	4.54 × 10-6	0.20	0.80
C.A.N.M	4.23 × 10-6	0.22	0.81
C.B.N.M	4.28 × 10-6	0.22	0.89
C.BL.N.M	4.30 × 10-6	0.20	0.87
M.A.N.M	4.46 × 10-6	0.20	0.87
M.B.N.M	4.57 × 10-6	0.21	0.95
M.BL.N.M	4.56 × 10-6	0.20	0.91
P.A.A.M	4.42 × 10-6	0.20	0.90
P.B.A.M	4.46 × 10-6	0.20	0.90
P.BL.A.M	4.57 × 10-6	0.20	0.88
P.A.B.M	4.43 × 10-6	0.20	0.89
P.B.B.M	4.52 × 10-6	0.20	0.90
P.BL.B.M	4.56 × 10-6	0.20	0.90
P.A.N.N	5.40 × 10-6	0.20	1.25
P.B.N.N	5.31 × 10-6	0.20	1.12
P.BL.N.N	5.32 × 10-6	0.20	1.18

4.2.2.1 Coefficient of Thermal Expansion

Results for CTE testing are provided in Table 4.4. Additional data, including the ranges and standard deviations of the test results are provided in Appendix B in Table B.8. . As stated in Chapter 2, the coarse aggregates utilized in concrete mixtures have historically been targeted as most influential in the CTE of concrete. However, in this study, the material with the greatest effect on CTE appears to be fine aggregate, with a large difference evident between mixtures containing manufactured sand, and mixtures containing natural sand. As can be seen in Table 4.4, mixtures that included the manufactured sand had CTE values between 4.23 x 10^{-6} in/in°F and 4.57 x 10^{-6} in/in°F. However, the CTE of the mixtures using the natural sand are closer to that of the MEPDG default value, with a range of 5.31 x 10^{-6} in/in°F to 5.40 x 10^{-6} in/in°F. The coarse aggregates on CTE was also observed, but to a lesser extent. The coastal aggregate mixtures (C.A.N.M, C.B.N.M, and C.BL.N.M) had lower CTE values with a range of 4.23 x 10^{-6} in/in°F to 4.30 x 10^{-6} in/in°F. The Piedmont aggregate mixtures that incorporated manufactured sand (P.A.N.M, P.B.N.M, and P.BL.N.M), including the piedmont mixtures with fly ash (P.A.A.M, P.B.A.M, P.BL.A.M, P.A.B.M, P.B.B.M, and P.B.BL.M) all had similar CTE values ranging from 4.42 x 10^{-6} in/in°F to 4.63 x 10^{-6} in/in°F. The Mountain aggregate mixtures (M.A.N.M, M.B.N.M, and M.BL.N.M) had similar CTE values to that of the Piedmont and manufactured sand mixtures. This is expected, as the Mountain and Piedmont coarse aggregates have similar minerology. The CTE for the Mountain coarse aggregate mixtures ranges from 4.46 x 10^{-6} in/in°F to 4.56 x 10^{-6} in/in°F.

4.2.2.2 Thermal Conductivity

Results for thermal conductivity are provided in Table 4.4. Additional data, including the ranges and standard deviations of the test results are provided in Appendix B in Table B.10. Like CTE, the type of fine aggregate utilized in a mixture appears to heavily influence the thermal conductivity. Thermal conductivity test results for all of the manufactured sand mixtures range from 0.80 Btu/(ft)(hr)(°F) to 0.95 Btu/(ft)(hr)(°F), whereas the thermal conductivity test results for the natural sand mixtures is closer to that of the MEPDG default value of 1.25 Btu/(ft)(hr)(°F). The range for the natural sand mixtures is between 1.12 Btu/(ft)(hr)(°F) and 1.24 Btu/(ft)(hr)(°F).

4.2.2.3 Heat Capacity

Results for heat capacity are provided in Table 4.4. Additional data, including the ranges and standard deviations of the test results are provided in Appendix B in Table B.9. The type of coarse aggregate utilized in a mixture appears to have the greatest influence over the heat capacity. Heat capacity test results for all of the Piedmont and Mountain coarse aggregate mixtures are typically 0.20 Btu/(lb-°F), whereas typically the heat capacity test results for the Coastal coarse aggregate mixtures are typically 0.22 Btu/(lb-°F). According to AASHTO 2015, typical heat capacity values for asphalt concrete range from 0.1 Btu/(lb-°F) to 0.5 Btu/(lb-°F) with a default value of 0.28 Btu/(lb-°F).

4.3 Catalog of MEPDG Inputs for Design of Concrete Pavements in North Carolina

In this research, it was determined that the cement type (including use of PLC) does not have as much influence on the mechanical and thermal properties used as MEPDG inputs as do the type of coarse and fine aggregate. Although the values presented in Tables 4.4 and 4.5 could be utilized as MEPDG inputs for concrete pavement design, a simplified version of the "catalog" of input values is presented in Table 4.6, below. The proposed catalog of inputs is broken into local region coarse aggregates (Coastal, Piedmont, and Mountain) and type of fine aggregate utilized. Based on the testing performed as part of this study, the data provided in Table 4.5 are locally appropriate input values for concrete pavements in North Carolina based on the materials selected for inclusion in this study.

Table 4.5: Catalog of MEPDG PCC inputs for North Carolina

Materials			Unit Weight (pcf)	MOE (psi)	Poisson's Ratio	MOR (psi)	CTE (in/in °F)	Heat Capacity (Btu/lb-°F)	Thermal Conductivity (Btu/(ft)(hr)(°F))
Coarse Aggregate	Fine Aggregate	Fly ash							
Piedmont	Manufactured Sand	No	145	3,000,000	0.19	660	4.63×10^{-6}	0.22	0.95
Piedmont	Manufactured Sand	Yes	142	3,000,000	0.19	660	4.57×10^{-6}	0.22	0.90
Piedmont	Natural Sand	No	142	3,300,000	0.16	740	5.40×10^{-6}	0.22	1.20
Coastal	Manufactured Sand	No	139	3,000,000	0.19	660	4.30×10^{-6}	0.22	0.90
Mountain	Manufactured Sand	No	146	3,000,000	0.19	660	4.56×10^{-6}	0.22	0.95

North Carolina Standard Specifications require a MOR of 650 psi at 28-days. MEPDG guidance indicates the input values should be the mean of what is achieved in the field, which would likely be above the specification value. As suggested by NCDOT personnel, the input values for MOR, MOE, and Poisson's ratio were selected based upon data for mixtures with 28-day MOR test results slightly above the 28-day specification. Using this approach, the following table was prepared to assist in selection of the MOR, MOE, and Poisson's ratio for the catalog. As approved by NCDOT personnel, averaging the test results for mixture achieving 28-day MOR between 650 psi and 680 psi (shown in Table 4.6 in yellow) resulted in identification of average MOE of approximately 3,000,000 psi and an average Poisson's ratio of 0.19 for the mixtures containing manufactured sand. For these mixtures, the previously utilized NCDOT MOR input of 660 psi is suggested. For natural sand mixtures (shown in green in Table 4.6), which exhibited slightly higher mechanical properties, the average values computed resulted in suggested inputs of 740 psi for MOR, 3,300,000 psi for MOE, and 0.16 for Poisson's ratio.

Table 4.6: Test results with mixtures sorted by MOR (low to high)

	Unit Weight	MOE	Poisson's	MOR	CTE	Heat Capacity	Thermal Conductivity
P.B.A.M	143	2,700,000	0.21	540	4.46×10^{-6}	0.20	0.90
P.BL.B.M	141	2,720,000	0.19	560	4.56×10^{-6}	0.20	0.88
M.A.N.M	146	2,540,000	0.18	570	4.46×10^{-6}	0.20	0.87
P.A.B.M	141	2,840,000	0.22	570	4.43×10^{-6}	0.20	0.89
M.BL.N.M	145	3,020,000	0.20	610	4.56×10^{-6}	0.20	0.91
P.B.B.M	141	2,510,000	0.18	620	4.52×10^{-6}	0.20	0.90
M.B.N.M	145	2,760,000	0.20	640	4.46×10^{-6}	0.21	0.95
P.A.A.M	141	3,220,000	0.23	650	4.42×10^{-6}	0.20	0.89
P.BL.A.M	142	2,690,000	0.16	650	4.57×10^{-6}	0.20	0.90
P.BL.N.M	144	2,430,000	0.18	660	4.54×10^{-6}	0.20	0.80
P.B.N.M	143	3,340,000	0.20	670	4.63×10^{-6}	0.20	0.95
P.A.N.M	145	2,920,000	0.20	680	4.57×10^{-6}	0.20	0.92
C.BL.N.M	139	3,690,000	0.22	680	4.30×10^{-6}	0.20	0.87
P.B.N.N	142	3,510,000	0.19	720	5.31×10^{-6}	0.20	1.12
C.A.N.M	138	3,730,000	0.22	730	4.23×10^{-6}	0.22	0.81
P.A.N.N	142	3,400,000	0.15	740	5.40×10^{-6}	0.20	1.24
C.B.N.M	139	3,490,000	0.21	750	4.28×10^{-6}	0.22	0.89
P.BL.N.N	141	3,040,000	0.15	750	5.32×10^{-6}	0.20	1.18

MOE	Poisson's	MOR	
3,048,333	0.19	665	AVERAGE - Manufactured Sand Mixtures with MOR 650-680 psi at 28-days
3,316,667	0.16	737	AVERAGE - Natural Sand Mixtures

4.4 Summary of Findings

Laboratory testing of the eighteen concrete mixtures in this study provided data for the development of the catalog of proposed MEPDG inputs for PCC presented in Table 4.5. Some key findings from the laboratory testing are:

- The cement type (OPC or PLC) used in the concrete mixture does not highly influence the laboratory test results for the suite of tests used to determine the MEPDG PCC inputs.

- The fine aggregate utilized in the concrete mixture (manufactured sand versus natural sand) had significant influence on the workability of the concrete as well as the thermal properties used for MEPDG PCC inputs.
- Although coarse aggregates vary greatly across North Carolina, the type of coarse aggregate utilized in this study did not highly influence the laboratory test results for the suite of tests used to determine the MEPDG PCC inputs.
- Use of fly ash in concrete pavement mixtures may make it unsuitable to utilize the 28-day strength as a PCC input in MEPDG due to the delayed strength gain.

The results from the laboratory testing should provide confidence to North Carolina pavement designers in their selection of PCC inputs for concrete pavement design.

CHAPTER 5: EVALUATION OF NEW CONCRETE INPUTS FOR MEPDG DESIGN

The AASHTOWare Pavement ME software was utilized to evaluate the potential impact of the new concrete inputs determined as part of this work on the design and predicted performance of North Carolina concrete pavements. To facilitate this analysis, the new inputs were utilized in place of the inputs previously used for design of several representative pavement sections, and the analysis was re-run using the Pavement ME Design software. Four concrete pavement projects designed by NCDOT prior to the project were identified and used to facilitate the evaluation. Holding other parameters constant, performance predictions using the new concrete inputs were compared to the performance predictions using the actual design inputs. A second analysis was performed using the new suggested inputs to determine if the thickness could be reduced due to the improved predicted distresses of the pavement sections.

5.1 Selected Concrete Pavement Projects in North Carolina

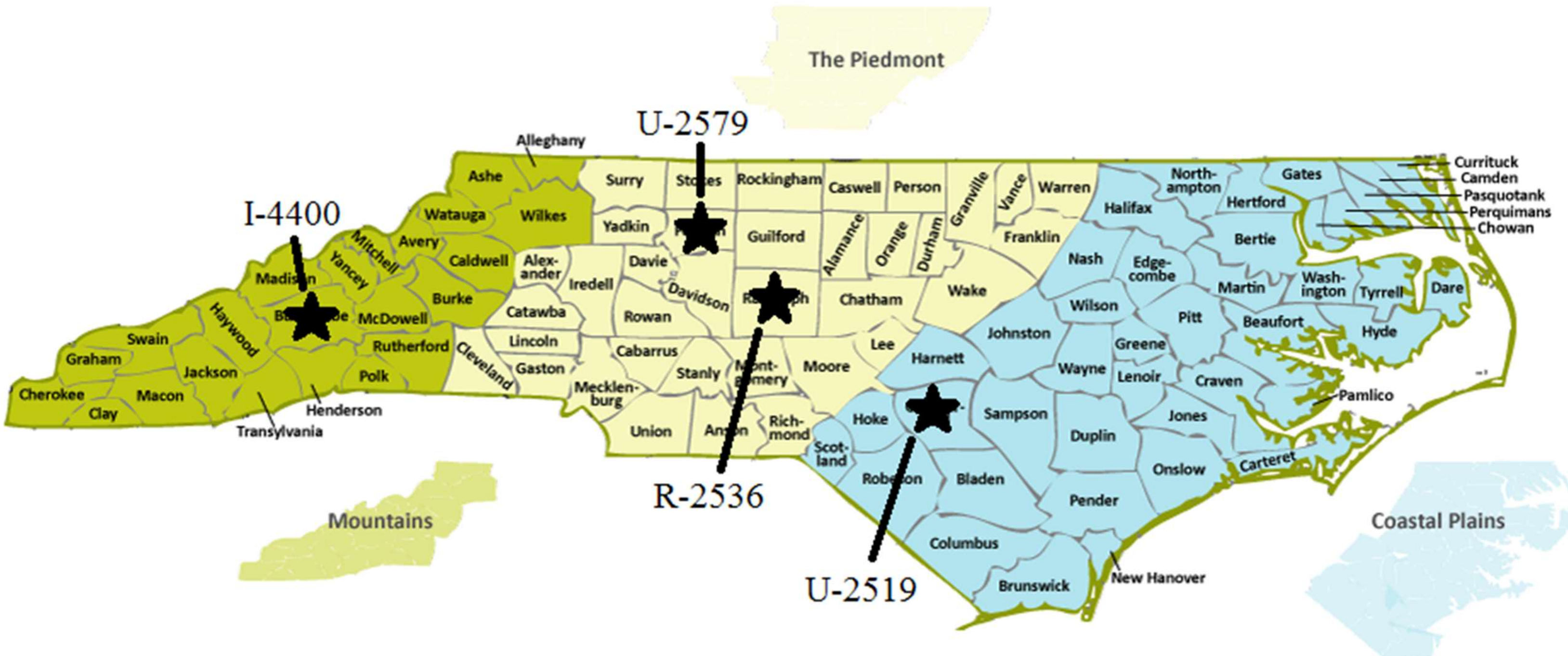
Four recent concrete pavement projects were identified by NCDOT, and the ME Pavement Design input files were provided to the researchers. These four projects were selected as representative examples of roadways recently designed for construction in several regions of North Carolina. The first project (I-4400 Concrete), in the Mountain region, is a four lane (in one direction) interstate in Buncombe County, NC. The second project (R-2536 Concrete FATC-17) is in the Piedmont region, and is a two lane (in one direction) bypass around Asheboro in Randolph County, NC. The third project (U-2579C

Update Concrete), also in the Piedmont region, is a three lane (in one direction) northern beltway in Forsyth County, NC. The fourth project (U-2519 CA Concrete) is a two lane (in one direction) outer loop around Fayetteville in Cumberland County, NC. For the purpose of this research, it is considered a Coastal project as well as a Piedmont project due to its location in an area near the generally accepted boundary of these two regions. Technically Cumberland County is in the Coastal region, but the area receives the majority of its aggregates from the nearby quarries in the Piedmont region. These selected projects are summarized in Table 5.1, below. A North Carolina map indicating the location of each project is shown in Figure 5.1.

Table 5.1: NCDOT selected project summaries

Project ID	Region	County and State	ADT	Number of lanes	Layer description and thickness
I-4400	Mountain	Buncombe Co., NC	13,400	4	10.5" of PCC
U-2579	Piedmont	Forsyth Co., NC	11,064	3	11" of PCC
R-2536	Piedmont	Randolph Co., NC	1,573	2	9.5" of PCC
U-2519	Coastal & Piedmont	Cumberland Co., NC	4,550	2	10" of PCC

Figure 5.1: NC region map with project locations



For most of these designs, NCDOT used target values of 185.00 in/mile for terminal IRI, 0.12 inches for mean joint faulting, and 10.00 % for JPCP transverse cracking, with reliability for each of these criteria set to 90 %. It is noted that for the R-2536 project, a target value of 172.00 in/mile for terminal IRI, 0.12 inches for mean joint faulting, and 15.00 % for JPCP transverse cracking, with reliability for each of these criteria set at 90 %. It should be noted that the tables in the following sections compare the NCDOT currently used PCC inputs to each individual cement in order to show that the cement does not have much influence.

5.1.1 Mountain Projects

Project I-4400 Concrete was selected to evaluate the inputs for the concrete representing Mountain region mixtures, due to its location in Buncombe County, NC which is located in the heart of the Mountain region.

5.1.1.1 Design and Predicted Performance

In Table 5.2, design details as well as the results for the comparison using the new suggested inputs determined through laboratory testing using Mountain coarse aggregates are provided. The concrete inputs for project I-4400 that were utilized by NCDOT in design of this pavement are listed in the yellow column in Table 5.2. As discussed previously, the input values for the PCC pavement utilized by NCDOT in design were typically the default values for the software, with the exception of NCDOT's decision to use a CTE value of 6.0 x 10^{-6} in/in°F and a Poisson's ratio of 0.17. In the design of project I-4400, NCDOT determined a 10.5 inch JPCP thickness was suitable for this roadway using the selected subgrade and pavement design inputs, along with the previously utilized concrete material inputs. Using these input values, the predicted performances were as follows: terminal IRI of 162.48 in/mile with a 96.88% reliability, mean joint faulting of 0.11 inches with a 94.30% reliability, and JPCP transverse cracking of 8.59 % with a 93.88% reliability.

Table 5.2: Analysis of project I-4400 using previous inputs and new inputs for concrete in the Mountain region

<table>
<tr><th colspan="3"></th><th>NCDOT Project R-2536 Randolph Co.</th><th>NCDOT 2MS Manufactured Sand with A Cement</th><th>NCDOT 2MS Manufactured Sand with B Cement</th><th>NCDOT 2MS Manufactured Sand with BL Cement</th></tr>
<tr><td rowspan="11">Layer 1: PCC</td><td colspan="2">Pavement Thickness (in)</td><td>10.5</td><td>10.5</td><td>10.5</td><td>10.5</td></tr>
<tr><td colspan="2">Cementitious Material Content (lb/yd^3)</td><td>600</td><td>550</td><td>550</td><td>550</td></tr>
<tr><td colspan="2">Water to cement ratio</td><td>0.42</td><td>0.48</td><td>0.48</td><td>0.48</td></tr>
<tr><td colspan="2">Unit Weight (PCF)</td><td>150</td><td>145</td><td>145</td><td>145</td></tr>
<tr><td colspan="2">28 Day Compressive Strength (psi)</td><td></td><td>5,030</td><td>5,100</td><td>4,790</td></tr>
<tr><td colspan="2">28 Day Modulus of Rupture (psi)</td><td>650</td><td>570</td><td>641</td><td>606</td></tr>
<tr><td colspan="2">28 Day Modulus of Elasticity (psi)</td><td>4,200,000</td><td>2,540,000</td><td>2,760,000</td><td>3,020,000</td></tr>
<tr><td colspan="2">Poisson's Ratio</td><td>0.17</td><td>0.18</td><td>0.20</td><td>0.20</td></tr>
<tr><td colspan="2">Coefficient of Thermal Expansion (x 10^{-6} in/in°F)</td><td>6.00</td><td>4.46</td><td>4.46</td><td>4.56</td></tr>
<tr><td colspan="2">Heat Capacity (Btu/lb-°F)</td><td>0.28</td><td>0.20</td><td>0.21</td><td>0.20</td></tr>
<tr><td colspan="2">Thermal Conductivity (Btu/(ft)(hr)(°F))</td><td>1.25</td><td>0.87</td><td>0.95</td><td>0.91</td></tr>
<tr><td colspan="3">Layer 2:</td><td colspan="4">4 inches of Flexible Pavement</td></tr>
<tr><td colspan="3">Layer 3:</td><td colspan="4">8 inches of Lime Stabilized</td></tr>
<tr><td colspan="3">Layer 4:</td><td colspan="4">12 inches of A-5 Subgrade</td></tr>
<tr><td colspan="3">Layer 5:</td><td colspan="4">Semi-infinite layer of A-5 Subgrade</td></tr>
<tr><td colspan="3">Climate Data</td><td colspan="4">Asheville, NC</td></tr>
<tr><td rowspan="3">Distress</td><td>Terminal IRI (in/mile)</td><td>185.00 (Target)</td><td>162.48</td><td>142.84</td><td>141.24</td><td>146.00</td></tr>
<tr><td>Mean Joint Faulting (in)</td><td>0.12 (Target)</td><td>0.11</td><td>0.07</td><td>0.08</td><td>0.08</td></tr>
<tr><td>JPCP Transverse Cracking (percent slabs)</td><td>10.00 (Target)</td><td>8.59</td><td>4.25</td><td>3.83</td><td>4.25</td></tr>
<tr><td rowspan="3">Reliability</td><td colspan="2">Terminal IRI (in/mile)</td><td>96.88</td><td>99.42</td><td>99.51</td><td>99.18</td></tr>
<tr><td colspan="2">Mean Joint Faulting (in)</td><td>94.30</td><td>99.78</td><td>99.69</td><td>99.40</td></tr>
<tr><td colspan="2">JPCP Transverse Cracking (percent slabs)</td><td>93.88</td><td>99.87</td><td>99.96</td><td>99.87</td></tr>
</table>

5.1.1.2 Predicted Performance Using New Inputs

To evaluate the effect of the new suggested inputs for PCC using coarse aggregates available in the Mountain region on the predicted performance of the project, the analysis was rerun. It is noted that due to limitations in the size of the testing program for this research study, the Mountain coarse aggregate was paired with a Piedmont manufactured sand. Although optimally a manufactured sand from the Mountain region would be utilized, based on the similar characteristics of the granitic Mountain and Piedmont coarse aggregate, it is possible that a manufactured sand sourced from a Mountain quarry would perform similarly to the Piedmont-sourced manufactured sand used in this study.

The target values set by NCDOT were kept constant at 185.00 in/mile for terminal IRI, 0.12 inches for mean joint faulting, and 10.00 % for JPCP transverse cracking. The pavement thickness, layer properties for layer two though five, and the climate data were also kept constant. The remaining PCC input values listed in Table 5.2 were modified to the new input values for concrete mixtures produced in the Mountain Region, as determined through laboratory testing of locally available materials as part of this research study.

The PCC pavement property inputs used for the Mountain coarse aggregate, manufactured sand, and cement A are listed in the column two of Table 5.2. Using these input values, the predicted performances were as follows: terminal IRI of 142.84 in/mile with a 99.42% reliability, mean joint faulting of 0.07 inches with a 99.78% reliability, and JPCP transverse cracking of 4.25 % with a 99.87% reliability. It should be noted that cement A was only used for analysis in the Mountain region due to its location in Tennessee and being the primary cement used throughout that region. The PCC pavement property inputs used for the Mountain coarse aggregate, manufactured sand, and cement B are listed in the third column of Table 5.2. Using these input values, the predicted performances were as follows: terminal IRI of 141.24 in/mile with a 99.51% reliability, mean joint faulting of 0.08 inches with a 99.69% reliability, and JPCP transverse cracking of 3.83 % with a 99.96% reliability. The PCC pavement property inputs used for the Mountain aggregate, manufactured sand, and cement BL are listed in the last column of Table 5.2. Using these input values, the predicted performances were as follows: terminal IRI of 146.00 in/mile with a 99.18% reliability, mean joint faulting of 0.08 inches with a 99.40% reliability, and JPCP transverse cracking of 4.25 % with a 99.87% reliability.

As can be seen, using concrete inputs based on locally available coarse aggregates, this pavement is predicted to have improved performance at the designed (and to-be-constructed) thickness of 10.5 inches. The most significant improvement is that the predicted IRI values drop by approximately 15 inches/mile (approximately 10 %). Improvements in joint faulting (approximately 0.03 in, or 27 %) and slab cracking (approximately 4.3 % slabs, or 50 % improvement) are also observed. Inputs determined using the PLC were similar to those of the OPC. As can be seen in Table 5.2, the predicted performance of the pavement designed with both sets of inputs (cement B and BL) is similar. Ultimately, this analysis indicates that this pavement is predicted to have improved performance using the new inputs. Conversely, it could be viewed that the performance predictions of the design using the original inputs are conservative.

5.1.2 Piedmont Projects

Projects U-2579C Update Concrete, R-2536 Concrete FATC-17, and U-2519 CA Concrete were selected to evaluate the impact of use of the new inputs for concrete made with locally available materials from the Piedmont region.

5.1.2.1 Design and Predicted Performance

In Table 5.3, NCDOT's design details as well as the results for the analysis using the new suggested inputs determined through laboratory testing using materials available in the Piedmont are provided. The concrete inputs for project U-2579C that were utilized by NCDOT in the design of the project are listed in the yellow column in Table 5.3. As discussed previously, the input values for the PCC utilized for design are the default values, with the exception of NCDOT's decision to use a CTE value of 6.0 x 10^{-6} in/in°F. In designing this pavement, NCDOT determined that an 11 inch JPCP thickness was suitable for this roadway using the selected subgrade and pavement design inputs, along with the previously utilized concrete materials inputs. Using these input values, the predicted performances were as follows: terminal IRI of 131.90 in/mile with a 99.83% reliability, mean joint faulting of 0.08 inches with a 99.34% reliability, and JPCP transverse cracking of 4.39 % with a 99.83% reliability.

Table 5.3: Analysis of project U-2579 using previous inputs and new inputs for concrete in the Piedmont region

<table>
<tr><th colspan="3"></th><th>NCDOT Project U-2579C Forsyth Co.</th><th>NCDOT 2MS Manufactured Sand with B Cement</th><th>NCDOT 2MS Manufactured Sand with BL Cement</th><th>C-33 Natural Sand with B Cement</th><th>C-33 Natural Sand with BL Cement</th></tr>
<tr><td rowspan="11">Layer 1: PCC</td><td colspan="2">Pavement Thickness (in)</td><td>11</td><td>11</td><td>11</td><td>11</td><td>11</td></tr>
<tr><td colspan="2">Cementitious Material Content (lb/yd^3)</td><td>600</td><td>550</td><td>550</td><td>550</td><td>550</td></tr>
<tr><td colspan="2">Water to cement ratio</td><td>0.42</td><td>0.48</td><td>0.48</td><td>0.48</td><td>0.48</td></tr>
<tr><td colspan="2">Unit Weight (PCF)</td><td>150</td><td>143</td><td>144</td><td>142</td><td>141</td></tr>
<tr><td colspan="2">28 Day Compressive Strength (psi)</td><td></td><td>4,850</td><td>5,020</td><td>4,390</td><td>5,190</td></tr>
<tr><td colspan="2">28 Day Modulus of Rupture (psi)</td><td>690</td><td>670</td><td>655</td><td>715</td><td>753</td></tr>
<tr><td colspan="2">28 Day Modulus of Elasticity (psi)</td><td>4,200,000</td><td>3,340,000</td><td>2,430,000</td><td>3,510,000</td><td>3,040,000</td></tr>
<tr><td colspan="2">Poisson's Ratio</td><td>0.20</td><td>0.20</td><td>0.18</td><td>0.19</td><td>0.15</td></tr>
<tr><td colspan="2">Coefficient of Thermal Expansion (x 10^{-6} in/in°F)</td><td>6.00</td><td>4.63</td><td>4.54</td><td>5.31</td><td>5.32</td></tr>
<tr><td colspan="2">Heat Capacity (Btu/lb-°F)</td><td>0.28</td><td>0.20</td><td>0.20</td><td>0.20</td><td>0.20</td></tr>
<tr><td colspan="2">Thermal Conductivity (Btu/(ft)(hr)(°F))</td><td>1.25</td><td>0.95</td><td>0.80</td><td>1.12</td><td>1.18</td></tr>
<tr><td colspan="3">Layer 2:</td><td colspan="5">4.25 inches of Flexible Pavement</td></tr>
<tr><td colspan="3">Layer 3:</td><td colspan="5">8 inches of Lime Stabilized</td></tr>
<tr><td colspan="3">Layer 4:</td><td colspan="5">12 inches of A-2-5 Subgrade</td></tr>
<tr><td colspan="3">Layer 5:</td><td colspan="5">Semi-infinite layer of A-2-5 Subgrade</td></tr>
<tr><td colspan="3">Climate Data</td><td colspan="5">Winston Salem, NC</td></tr>
<tr><td rowspan="3">Distress</td><td>Terminal IRI (in/mile)</td><td>185.00 (Target)</td><td>131.90</td><td>117.80</td><td>112.06</td><td>126.66</td><td>121.81</td></tr>
<tr><td>Mean Joint Faulting (in)</td><td>0.12 (Target)</td><td>0.08</td><td>0.06</td><td>0.05</td><td>0.07</td><td>0.07</td></tr>
<tr><td>JPCP Transverse Cracking (percent slabs)</td><td>10.00 (Target)</td><td>4.39</td><td>3.83</td><td>3.83</td><td>3.83</td><td>3.83</td></tr>
<tr><td rowspan="3">Reliability</td><td colspan="2">Terminal IRI (in/mile)</td><td>99.83</td><td>99.99</td><td>100.00</td><td>99.93</td><td>99.97</td></tr>
<tr><td colspan="2">Mean Joint Faulting (in)</td><td>99.34</td><td>99.98</td><td>100.00</td><td>99.76</td><td>99.92</td></tr>
<tr><td colspan="2">JPCP Transverse Cracking (percent slabs)</td><td>99.83</td><td>99.96</td><td>99.96</td><td>99.96</td><td>99.96</td></tr>
</table>

In Table 5.4, design details as well as the results for the comparison using the new suggested inputs determined through laboratory testing using Piedmont coarse aggregate are provided. The PCC pavement property inputs for project R-2536 that were utilized by NCDOT for design of the project are listed in the yellow column in Table 5.4. It was found that NCDOT determined a 9.5 inch JPCP thickness was suitable for this roadway using the selected subgrade and pavement design inputs, along with the previously utilized concrete materials inputs. Using these input values, the predicted performances were as follows:

terminal IRI of 136.13 in/mile with a 99.13% reliability, mean joint faulting of 0.07 inches with a 99.81% reliability, and JPCP transverse cracking of 7.98 % with a 99.59% reliability.

Table 5.4: Analysis of project R-2536 using previous inputs and new inputs for concrete in the Piedmont region

			NCDOT Project R-2536 Randolph Co.	NCDOT 2MS Manufactured Sand with B Cement	NCDOT 2MS Manufactured Sand with BL Cement	C-33 Natural Sand with B Cement	C-33 Natural Sand with BL Cement
Layer 1: PCC	Pavement Thickness (in)		9.5	9.5	9.5	9.5	9.5
	Cementitious Material Content (lb/yd^3)		600	550	550	550	550
	Water to cement ratio		0.42	0.48	0.48	0.48	0.48
	Unit Weight (PCF)		150	143	144	142	141
	28 Day Compressive Strength (psi)			4,850	5,020	4,390	5,190
	28 Day Modulus of Rupture (psi)		650	670	655	715	753
	28 Day Modulus of Elasticity (psi)		4,200,000	3,340,000	2,430,000	3,510,000	3,040,000
	Poisson's Ratio		0.17	0.20	0.18	0.19	0.15
	Coefficient of Thermal Expansion (x 10^{-6} in/in°F)		6.00	4.63	4.54	5.31	5.32
	Heat Capacity (Btu/lb-°F)		0.28	0.20	0.20	0.20	0.20
	Thermal Conductivity (Btu/(ft)(hr)(°F))		1.25	0.95	0.80	1.12	1.18
Layer 2:			6 inches of Sandwich Granular				
Layer 3:			12 Inches of A-5 Subgrade				
Layer 4:			Semi-infinite layer of A-5 Subgrade				
Layer 5:			N.A.				
Climate Data			Chapel Hill, NC				
Distress	Terminal IRI (in/mile)	172.00 (Target)	136.13	125.90	120.02	133.47	129.02
	Mean Joint Faulting (in)	0.12 (Target)	0.07	0.06	0.05	0.07	0.07
	JPCP Transverse Cracking (percent slabs)	15.00 (Target)	7.98	4.65	3.83	4.49	3.83
Reliability	Terminal IRI (in/mile)		99.13	99.76	99.91	99.35	99.62
	Mean Joint Faulting (in)		99.81	99.98	100.00	99.80	99.92
	JPCP Transverse Cracking (percent slabs)		99.59	100.00	100.00	100.00	100.00

Table 5.5 provides design details as well as the results for the comparison using the new suggested inputs determined through laboratory testing using Piedmont coarse aggregate. The PCC pavement property inputs for project U-2519 that were provided by

NCDOT are listed in the yellow column in Table 5.5. NCDOT used target values of 185.00 in/mile for terminal IRI, 0.12 inches for mean joint faulting, and 10.00 % for JPCP transverse cracking. It was found that NCDOT selected a 10 inch pavement thickness for this roadway, which was analyzed using a CTE value of 6.0 x 10^{-6} in/in°F. The remaining input values for the PCC pavement are considered default for this study. Using these input values, the predicted performances were as follows: terminal IRI of 144.74 in/mile with a 99.24% reliability, mean joint faulting of 0.10 inches with a 97.31% reliability, and JPCP transverse cracking of 4.25 % with a 99.87% reliability.

Table 5.5: Analysis of project U-2519 using previous inputs and new inputs for concrete in the Piedmont region

			NCDOT Project U-2519 Cumberland Co.	NCDOT 2MS Manufactured Sand with B Cement	NCDOT 2MS Manufactured Sand with BL Cement	C-33 Natural Sand with B Cement	C-33 Natural Sand with BL Cement
Layer 1: PCC	Pavement Thickness (in)		10	10	10	10	10
	Cementitious Material Content (lb/yd^3)		600	550	550	550	550
	Water to cement ratio		0.42	0.48	0.48	0.48	0.48
	Unit Weight (PCF)		150	143	144	142	141
	28 Day Compressive Strength (psi)			4,850	5,020	4,390	5,190
	28 Day Modulus of Rupture (psi)		690	670	655	715	753
	28 Day Modulus of Elasticity (psi)		4,200,000	3,340,000	2,430,000	3,510,000	3,040,000
	Poisson's Ratio		0.20	0.20	0.18	0.19	0.15
	Coefficient of Thermal Expansion (x 10^{-6} in/in°F)		6.00	4.63	4.54	5.31	5.32
	Heat Capacity (Btu/lb-°F)		0.28	0.20	0.20	0.20	0.20
	Thermal Conductivity (Btu/(ft)(hr)(°F))		1.25	0.95	0.80	1.12	1.18
Layer 2:			4.25 inches of Flexible Pavement				
Layer 3:			8 inches of Lime Stabilized				
Layer 4:			12 inches of A-6 Subgrade				
Layer 5:			Semi-infinite layer of A-6 Subgrade				
Climate Data			Fayetteville, NC				
Distress	Terminal IRI (in/mile)	185.00 (Target)	144.74	129.42	123.15	139.76	134.12
	Mean Joint Faulting (in)	0.12 (Target)	0.10	0.07	0.06	0.09	0.08
	JPCP Transverse Cracking (percent slabs)	10.00 (Target)	4.25	3.83	3.83	3.83	3.83
Reliability	Terminal IRI (in/mile)		99.24	99.89	99.96	99.56	99.78
	Mean Joint Faulting (in)		97.31	99.82	99.97	98.60	99.40
	JPCP Transverse Cracking (percent slabs)		99.87	99.96	99.96	99.96	99.96

5.1.2.2 Predicted Performance Using New Inputs

To evaluate project U-2579 previously used inputs to the new suggested inputs for PCC using local materials available in the Piedmont region on the predicted performance, the analysis was rerun. The target values set by NCDOT were kept constant at 185.00 in/mile for terminal IRI, 0.12 inches for mean joint faulting, and 10.00 % for JPCP transverse cracking. The pavement thickness, layer properties for layer two though five, and the climate data were also kept constant. The remaining PCC input values listed in

Table 5.3 were modified to meet the input values determined through laboratory testing of locally available materials as part of this research study.

The PCC pavement property inputs used for the Piedmont aggregate, manufactured sand, and cement B are listed in the second column of Table 5.3. Using these input values, the predicted performances were as follows: terminal IRI of 125.90 in/mile with a 99.76% reliability, mean joint faulting of 0.06 inches with a 99.98% reliability, and JPCP transverse cracking of 4.65 % with a 100.00% reliability. The PCC pavement property inputs used for the Piedmont aggregate, manufactured sand, and cement BL listed in the third column of Table 5.3. Using these input values, the predicted performances were as follows: terminal IRI of 120.02 in/mile with a 99.91% reliability, mean joint faulting of 0.05 inches with a 100.00% reliability, and JPCP transverse cracking of 3.83 % with a 100.00% reliability. The PCC pavement property inputs used for the Piedmont aggregate, natural sand, and cement B are listed in the fourth column of Table 5.3. Using these input values, the predicted performances were as follows: terminal IRI of 133.47 in/mile with a 99.35% reliability, mean joint faulting of 0.07 inches with a 99.80% reliability, and JPCP transverse cracking of 4.49 % with a 100.00% reliability. The PCC pavement property inputs used for the Piedmont aggregate, natural sand, and cement BL are listed in the last column of Table 5.3. Using these input values, the predicted performances were as follows: terminal IRI of 129.02 in/mile with a 99.62% reliability, mean joint faulting of 0.07 inches with a 99.92% reliability, and JPCP transverse cracking of 3.83 % with a 100.00% reliability.

As can be seen, using concrete inputs based on locally available materials with manufactured sand, this pavement is predicted to have improved performance at a thickness of 11 inches. The most significant improvement is that the predicted IRI values drop by approximately 14 inches/mile (approximately 11 %). Improvements in joint faulting (approximately 0.02 in, or 25 %) and slab cracking (approximately 0.5 % slabs, or 13 % improvement) are also predicted.

The same analysis using inputs for concrete with locally available coarse aggregates, cement, and natural sand (instead of manufactured sand) reveals predicted performance improvements as well, but on a smaller scale. The most significant improvement is that the predicted IRI values drop by approximately 6 inches/mile (approximately 4 %). Improvements in joint faulting (approximately 0.01 in, or 13 %) and slab cracking (approximately 0.5 % slabs, or 13 % improvement) are also observed. As expected based on the similar performance of the PLC and OPC concrete mixtures in the laboratory testing associated with the MEPDG inputs, use of the inputs for PLC concrete in the analysis resulted in similar predicted performance to the OPC concrete pavement. Overall, it was determined that the type of sand (manufactured vs. natural) used had a larger impact on the predicted performance of this pavement than the cement type (OPC or PLC).

Following the approach described above for project U-2579, a similar analysis was performed on project R-2536. The target values set by NCDOT were kept constant at 172.00 in/mile for terminal IRI, 0.12 inches for mean joint faulting, and 15.00 % for JPCP transverse cracking. The pavement thickness, layer properties for layer two though five, and the climate data were also kept constant. The remaining PCC input values listed in Table 5.4 were modified to meet the input values determined through laboratory testing of locally available materials as part of this research study.

The concrete inputs determined for concrete mixtures produced with the Piedmont aggregate, manufactured sand, and cement B are listed in the second column of Table 5.4.

Using these input values, the predicted performances were as follows: terminal IRI of 125.90 in/mile with a 99.76% reliability, mean joint faulting of 0.06 inches with a 99.98% reliability, and JPCP transverse cracking of 4.65 % with a 100.00% reliability. The concrete inputs used for the Piedmont aggregate, manufactured sand, and cement BL listed in the third column of Table 5.4. Using these input values, the predicted performances were as follows: terminal IRI of 120.02 in/mile with a 99.91% reliability, mean joint faulting of 0.05 inches with a 100.00% reliability, and JPCP transverse cracking of 3.83 % with a 100.00% reliability. The concrete inputs used for the Piedmont aggregate, natural sand, and cement B are listed in the fourth column of Table 5.4. Using these input values, the predicted performances were as follows: terminal IRI of 133.47 in/mile with a 99.35% reliability, mean joint faulting of 0.07 inches with a 99.80% reliability, and JPCP transverse cracking of 4.49 % with a 100.00% reliability. The concrete inputs used for the concrete mixtures produced with the Piedmont aggregate, natural sand, and cement BL are listed in the last column of Table 5.4. Using these input values, the predicted performances were as follows: terminal IRI of 129.02 in/mile with a 99.62% reliability, mean joint faulting of 0.07 inches with a 99.92% reliability, and JPCP transverse cracking of 3.83 % with a 100.00% reliability.

As can be seen, using concrete inputs based on locally available materials with manufactured sand, this pavement is predicted to have improved performance at the as-designed (and to-be-constructed) thickness of 9.5 inches. The most significant predicted improvement is that the predicted IRI values drop by approximately 10 inches/mile (approximately 8 %). Improvements in predicted joint faulting (approximately 0.01 in, or 14 %) and slab cracking (approximately 3.3 % slabs, or 42 % improvement) are also observed. The same analysis using the inputs for concrete mixtures produced with locally available materials (cement, coarse aggregate) and natural sand shows improvements as well but on a smaller scale. The most significant improvement is that the predicted IRI values drop by approximately 3 inches/mile (approximately 2 %). No improvements in joint faulting were noted. However, improvements to slab cracking (approximately 3.5 % slabs, or 44 % improvement) are also observed and comparable to the predicted performance of concrete pavement using mixtures with the manufactured sand. As anticipated based on similar laboratory test results, use of the inputs for concrete made with PLC had performance predicted to be similar to the OPC concrete pavement. However, similar predicted performance of the PLC, paired with the sustainability benefits associated with its use, provide incentive for use. It was noted that the type of sand used had a larger impact on the predicted performance than the cement type.

To evaluate project U-2519 the approach described above for project U-2579 and project R-2536 was repeated. The target values set by NCDOT were kept constant at 185.00 in/mile for terminal IRI, 0.12 inches for mean joint faulting, and 10.00 % for JPCP transverse cracking. The pavement thickness, layer properties for layer two though five, and the climate data were also kept constant. The remaining PCC input values listed in Table 5.5 were modified to evaluate the impact of the new inputs determined as part of this research study.

The PCC pavement property inputs used for concrete mixtures produced with the Piedmont aggregate, manufactured sand, and cement B are listed in the second column of Table 5.5. Using these input values, the predicted performances were as follows: terminal IRI of 129.42 in/mile with a 99.89% reliability, mean joint faulting of 0.07 inches with a

99.82% reliability, and JPCP transverse cracking of 3.83 % with a 99.96% reliability. The PCC pavement property inputs used for concrete mixtures produced with the Piedmont aggregate, manufactured sand, and cement BL listed in the third column of Table 5.5. Using these input values, the predicted performances were as follows: terminal IRI of 123.15 in/mile with a 99.96% reliability, mean joint faulting of 0.06 inches with a 99.97% reliability, and JPCP transverse cracking of 3.83 % with a 99.96% reliability. The PCC pavement property inputs used for concrete mixtures produced with the Piedmont aggregate, natural sand, and cement B are listed in the fourth column of Table 5.5. Using these input values, the predicted performances were as follows: terminal IRI of 139.76 in/mile with a 99.56% reliability, mean joint faulting of 0.09 inches with a 98.60% reliability, and JPCP transverse cracking of 3.83 % with a 99.96% reliability. The PCC pavement property inputs used for concrete mixtures produced with the Piedmont aggregate, natural sand, and cement BL are listed in the last column of Table 5.5. Using these input values, the predicted performances were as follows: terminal IRI of 134.12 in/mile with a 99.78% reliability, mean joint faulting of 0.08 inches with a 99.40% reliability, and JPCP transverse cracking of 3.83 % with a 99.96% reliability.

As can be seen in Table 5.5, using concrete inputs based on laboratory testing of concrete mixtures produced with locally available materials with manufactured sand, this pavement is predicted to have improved performance at a thickness of 10 inches. The most significant improvement is that the predicted IRI values drop by approximately 15 inches/mile (approximately 11 %). Improvements in joint faulting (approximately 0.03 in, or 30 %) and slab cracking (approximately 0.4 % slabs, or 10 % improvement) are also observed. The same analysis using inputs for concrete mixtures produced with locally available materials and natural sand shows improvements as well, but on a smaller scale. The most significant improvement is that the predicted IRI values drop by approximately 5 inches/mile (approximately 3 %). Improvements in joint faulting (approximately 0.01 in, or 10 %) and slab cracking (approximately 0.4 % slabs, or 10 %) are also observed.

Again, as anticipated based on the similar laboratory test results for PLC and OPC concrete, the predicted performance of the PLC pavement is similar to the performance of the OPC pavement. It was noted that the type of sand used had a larger impact on the predicted performance than the cement type.

5.1.3 Coastal Projects

Currently, fewer concrete pavements are being designed and constructed in the Coastal region of North Carolina. However, more concrete pavement projects are predicted in the Coastal region in the future. The design for a recent concrete pavement project near the Coastal region, Project U-2519 CA Concrete, was identified by NCDOT and provided to the researchers for this study. The location of this project (Fayetteville, NC) is in the Coastal region, but on the border with the Piedmont region from which coarse aggregate for this metropolitan area is typically provided. For the purpose of this analysis, however, U-2519 was treated as a Coastal region project to evaluate the impact of the new input values for Coastal concrete.

5.1.3.1 Design and Predicted Performance

In Table 5.6, design details as well as the results for the comparison using the new suggested inputs determined through laboratory testing using Coastal coarse aggregate are provided. The concrete inputs for project U-2519 that were provided by NCDOT are listed in the yellow column in Table 5.6. As discussed previously, the input values for the PCC utilized for design are the default values, with the exception of NCDOT's decision to use a CTE value of 6.0×10^{-6} in/in°F. In design of this project, NCDOT determined that a 10 inch JPCP thickness was suitable for this roadway using the selected subgrade and pavement design inputs, along with the previously utilized concrete inputs. Using these input values, the predicted performances were as follows: terminal IRI of 144.74 in/mile with a 99.24% reliability, mean joint faulting of 0.10 inches with a 97.31% reliability, and JPCP transverse cracking of 4.25 % with a 99.87% reliability.

Table 5.6: Analysis of project U-2519 using previous inputs and new inputs for concrete in the Coastal region

				NCDOT Project U-2519 Cumberland Co.	NCDOT 2MS Manufactured Sand with B Cement	NCDOT 2MS Manufactured Sand with BL Cement
Layer 1: PCC	Pavement Thickness (in)			10	10	10
	Cementitious Material Content (lb/yd^3)			600	550	550
	Water to cement ratio			0.42	0.48	0.48
	Unit Weight (PCF)			150	139	139
	28 Day Compressive Strength (psi)				5,960	5,610
	28 Day Modulus of Rupture (psi)			690	750	676
	28 Day Modulus of Elasticity (psi)			4,200,000	3,490,000	3,690,000
	Poisson's Ratio			0.20	0.21	0.22
	Coefficient of Thermal Expansion (x 10^{-6} in/in°F)			6.00	4.28	4.30
	Heat Capacity (Btu/lb-°F)			0.28	0.22	0.20
	Thermal Conductivity (Btu/(ft)(hr)(°F))			1.25	0.89	0.87
Layer 2:				4.25 inches of Flexible Pavement		
Layer 3:				8 inches of Lime Stabilized		
Layer 4:				12 inches of A-6 Subgrade		
Layer 5:				Semi-infinite layer of A-6 Subgrade		
Climate Data				Fayetteville, NC		
Distress	Terminal IRI (in/mile)	185.00	(Target)	144.74	125.61	129.47
	Mean Joint Faulting (in)	0.12	(Target)	0.10	0.07	0.07
	JPCP Transverse Cracking (percent slabs)	10.00	(Target)	4.25	3.83	3.83
Reliability	Terminal IRI (in/mile)			99.24	99.94	99.89
	Mean Joint Faulting (in)			97.31	99.90	99.81
	JPCP Transverse Cracking (percent slabs)			99.87	99.96	99.96

5.1.3.2 Predicted Performance Using New Inputs

To evaluate the effect of the new suggested inputs for PCC using local materials available in the Coastal region on the predicted performance of the project, the analysis was rerun. The target values set by NCDOT were kept constant at 185.00 in/mile for terminal IRI, 0.12 inches for mean joint faulting, and 10.00 % for JPCP transverse cracking.

The pavement thickness, layer properties for layer two though five, and the climate data were also kept constant. The remaining PCC input values listed in Table 5.1 were modified to the input values determined through laboratory testing of locally available materials as part of this research study.

The PCC pavement property inputs used for the Coastal aggregate, manufactured sand, and cement B are listed in the middle column of Table 5.6. Using these input values, the predicted performances are as follows: terminal IRI of 125.61 in/mile with a 99.94% reliability, mean joint faulting of 0.07 inches with a 99.90% reliability, and JPCP transverse cracking of 3.83 % with a 99.96% reliability. The PCC pavement property inputs used for the Coastal aggregate, manufactured sand, and cement BL are listed in the last column of Table 5.6. Using these input values, the predicted performances were as follows: terminal IRI of 129.47 in/mile with a 99.89% reliability, mean joint faulting of 0.07 inches with a 99.81% reliability, and JPCP transverse cracking of 3.83 % with a 99.96% reliability.

As can be seen in Table 5.6, using concrete inputs based on locally available materials, this pavement is predicted to have improved performance at a thickness of 10 inches. The most significant improvement is that the predicted IRI values drop by approximately 15 inches/mile (approximately 10 %). Improvements in joint faulting (approximately 0.03 in, or 30 %) and slab cracking (approximately 0.4 % slabs, or 10 % improvement) are also observed. As anticipated based on similar laboratory test results, use of the inputs for concrete made with PLC had performance predicted to be similar to the OPC concrete pavement. However, sustainability benefits associated with the use of PLC along with improvements in smoothness, cracking, and joint faulting would offer additional potential advantages (Chimmula 2016).

5.2 Evaluation of Potential Impact of New Inputs on Concrete Pavement Design Thickness

In this section, the potential impact of the new concrete inputs on the required (or design) thickness of PCC pavements is evaluated. Using the four NCDOT projects previously described in Section 5.1 along with the catalog of suggested concrete inputs presented in Table 4.5 analyses with AASHTOWare Pavement ME software were performed to identify the reduction in thickness of PCC that could be obtained. For each of the following analyses, the size of the dowel bars was modified per NCDOT specifications presented in Standard Drawing 700.01 (shown in Table 5.7). In some cases, the dowel bar sizes selected in the design provided by NCDOT did not meet the standard drawing and in that case the base comparisons were left unmodified.

Table 5.7: Table I – Dowel bars, as found in NCDOT standard drawing 700.01

TABLE I - DOWEL BARS		
SLAB THICKNESS	DOWEL BAR "D"	DOWEL LENGTH "L"
8" OR LESS	1"	14"
8 1/2" TO 9 1/2"	1 1/8"	16"
10" TO 10 1/2"	1 1/4"	18"
11" AND ABOVE	1 1/2"	18"

The first project, shown in Table 5.8, provides design details as well as the results for the thickness optimization for project I-4400 using new suggested inputs determined through laboratory testing of concrete mixtures produced using Mountain aggregates and manufactured sand. The target ranges for this project are as follows: terminal IRI of 185.00 in/mile, mean joint faulting of 0.12 inches, and transverse cracking of 10.00 %. The input values and predicted performance of the original project are listed in the yellow column. The next column has the same design parameters as the original project with the Mountain coarse aggregate concrete inputs. The two left columns have the Mountain coarse aggregate concrete inputs with modified pavement thickness in half inch increments and dowel bar diameters to match based on the NCDOT Standard Drawing 700.01. The predicted performances of this analysis indicate that using the new PCC input values, project I-4400 could not be reduced in thickness (due to failing at mean joint faulting predicted performance, shown in red in Table 5.8) while remaining within the target ranges specified in AASHTOWare Pavement ME by NCDOT.

Table 5.8: Project evaluation of potential PCC thickness reduction using locally calibrated inputs for Mountain region concrete

			NCDOT Project R-2536 Randolph Co.	NCDOT 2MS Manufactured Sand 10.5 inch	NCDOT 2MS Manufactured Sand 10 inch	NCDOT 2MS Manufactured Sand 9.5 inch
Layer 1: PCC	Pavement Thickness (in)		10.5	10.5	10	9.5
	Dowel Diameter (in)		1.5	1.5	1.25	1.125
	Cementitious Material Content (lb/yd^3)		600	550	550	550
	Water to cement ratio		0.42	0.48	0.48	0.48
	Unit Weight (PCF)		150	146	146	146
	28 Day Modulus of Rupture (psi)		650	660	660	660
	28 Day Modulus of Elasticity (psi)		4,200,000	3,000,000	3,000,000	3,000,000
	Poisson's Ratio		0.17	0.19	0.19	0.19
	Coefficient of Thermal Expansion (x 10^{-6} in/in°F)		6.00	4.56	4.56	4.56
	Heat Capacity (Btu/lb-°F)		0.28	0.21	0.21	0.21
	Thermal Conductivity (Btu/(ft)(hr)(°F))		1.25	0.95	0.95	0.95
Layer 2:			4 inches of Flexible Pavement			
Layer 3:			8 inches of Lime Stabilized			
Layer 4:			12 inches of A-5 Subgrade			
Layer 5:			Semi-infinite layer of A-5 Subgrade			
Climate Data			Asheville, NC			
Distress	Terminal IRI (in/mile)	185.00 (Target)	162.48	141.56	173.21	210.32
	Mean Joint Faulting (in)	0.12 (Target)	0.11	0.08	0.13	0.19
	JPCP Transverse Cracking (percent slabs)	10.00 (Target)	8.59	3.83	3.83	4.39
Reliability	Terminal IRI (in/mile)		96.88	99.49	94.17	77.50
	Mean Joint Faulting (in)		94.30	99.62	84.97	39.17
	JPCP Transverse Cracking (percent slabs)		93.88	99.96	99.96	99.83

Project U-2579, was analyzed using the new suggested Piedmont coarse aggregate with manufactured sand concrete inputs, as shown in Table 5.9. The target ranges for this project are as follows: terminal IRI of 185.00 in/mile, mean joint faulting of 0.12 inches, and transverse cracking of 10.00 %. The input values and predicted performance of the original project are listed in the yellow column. The next column has the same design parameters as the original project with the Piedmont coarse aggregate concrete inputs. The two left columns have the Piedmont coarse aggregate concrete inputs with modified pavement thickness in half inch increments and dowel bar diameters to match based on the NCDOT Standard Drawing 700.01. As can be seen in Table 5.9, the new concrete inputs for Piedmont concrete facilitate sufficient performance of a thinner PCC section. However, a significant decrease in predicted performance occurs between section thicknesses of 11 inches and 10.5 inches due to the change in the dowel bar size of 1.5 inches to 1.25 inches.

Table 5.9: Evaluation of potential PCC thickness reduction using locally calibrated inputs for Piedmont region concrete

			NCDOT Project U-2579C Forsyth Co.	NCDOT 2MS Manufactured Sand 11 inch	NCDOT 2MS Manufactured Sand 10.5 inch	NCDOT 2MS Manufactured Sand 10 inch
Layer 1: PCC	Pavement Thickness (in)		11	11	10.5	10
	Dowel Diameter (in)		1.5	1.5	1.25	1.25
	Cementitious Material Content (lb/yd^3)		600	550	550	550
	Water to cement ratio		0.42	0.48	0.48	0.48
	Unit Weight (PCF)		150	145	145	145
	28 Day Modulus of Rupture (psi)		690	660	660	660
	28 Day Modulus of Elasticity (psi)		4,200,000	3,000,000	3,000,000	3,000,000
	Poisson's Ratio		0.20	0.19	0.19	0.19
	Coefficient of Thermal Expansion (x 10^{-6} in/in°F)		6.00	4.63	4.63	4.63
	Heat Capacity (Btu/lb-°F)		0.28	0.20	0.20	0.20
	Thermal Conductivity (Btu/(ft)(hr)(°F))		1.25	0.95	0.95	0.95
Layer 2:			4.25 inches of Flexible Pavement			
Layer 3:			8 inches of Lime Stabilized			
Layer 4:			12 inches of A-2-5 Subgrade			
Layer 5:			Semi-infinite layer of A-2-5 Subgrade			
Climate Data			Winston Salem, NC			
Distress	Terminal IRI (in/mile)	185.00 (Target)	131.90	115.16	143.79	143.03
	Mean Joint Faulting (in)	0.12 (Target)	0.08	0.05	0.10	0.10
	JPCP Transverse Cracking (percent slabs)	10.00 (Target)	4.39	3.83	3.83	3.83
Reliability	Terminal IRI (in/mile)		99.83	99.99	99.31	99.36
	Mean Joint Faulting (in)		99.34	99.99	96.89	97.26
	JPCP Transverse Cracking (percent slabs)		99.83	99.96	99.96	99.96

Project U-2579, shown in Table 5.10, was analyzed using the new suggested Piedmont coarse aggregate with natural sand concrete inputs. The target ranges for this project are as follows: terminal IRI of 185.00 in/mile, mean joint faulting of 0.12 inches, and transverse cracking of 10.00 %. The input values and predicted performance of the original project are listed in the yellow column. The next column has the same design as the original project with the exception of the new suggested inputs for concrete mixtures produced with the Piedmont coarse aggregate and natural sand. The two left columns are the Piedmont coarse aggregate concrete inputs with modified pavement thickness in half inch increments and dowel bar diameters to match based on the NCDOT Standard Drawing 700.01. The predicted performances of this analysis indicate that using the new concrete input values, project U-2579 could not be reduced in thickness (due to failing at mean joint faulting predicted performance, shown in red in Table 5.10) while remaining within the target ranges specified in AASHTOWare Pavement ME by NCDOT.

Table 5.10: Evaluation of potential PCC thickness reduction using locally calibrated inputs for Piedmont region concrete

			NCDOT Project U-2579C Forsyth Co.	Natural Sand 11 inch	Natural Sand 10.5 inch	Natural Sand 10 inch
Layer 1: PCC	Pavement Thickness (in)		11	11	10.5	10
	Dowel Diameter (in)		1.5	1.5	1.25	1.25
	Cementitious Material Content (lb/yd^3)		600	550	550	550
	Water to cement ratio		0.42	0.48	0.48	0.48
	Unit Weight (PCF)		150	142	142	142
	28 Day Modulus of Rupture (psi)		690	740	740	740
	28 Day Modulus of Elasticity (psi)		4,200,000	3,300,000	3,300,000	3,300,000
	Poisson's Ratio		0.20	0.16	0.16	0.16
	Coefficient of Thermal Expansion (x 10^{-6} in/in°F)		6.00	5.40	5.40	5.40
	Heat Capacity (Btu/lb-°F)		0.28	0.20	0.20	0.20
	Thermal Conductivity (Btu/(ft)(hr)(°F))		1.25	1.20	1.20	1.20
Layer 2:			4.25 inches of Flexible Pavement			
Layer 3:			8 inches of Lime Stabilized			
Layer 4:			12 inches of A-2-5 Subgrade			
Layer 5:			Semi-infinite layer of A-2-5 Subgrade			
Climate Data			Winston Salem, NC			
Distress	Terminal IRI (in/mile)	185.00 (Target)	131.90	124.12	159.30	158.29
	Mean Joint Faulting (in)	0.12 (Target)	0.08	0.07	0.13	0.13
	JPCP Transverse Cracking (percent slabs)	10.00 (Target)	4.39	3.83	3.83	3.83
Reliability	Terminal IRI (in/mile)		99.83	99.95	97.43	97.60
	Mean Joint Faulting (in)		99.34	99.85	86.15	87.21
	JPCP Transverse Cracking (percent slabs)		99.83	99.96	99.96	99.96

Project R-2536, shown in Table 5.11, was analyzed using the new suggested Piedmont coarse aggregate with manufactured sand concrete inputs. The target ranges for this project are as follows: terminal IRI of 172.00 in/mile, mean joint faulting of 0.12 inches, and transverse cracking of 15.00 %. The input values and predicted performance of the original project are listed in the yellow column. The predicted performances of this analysis show that project R-2536 could potentially be reduced by up to an inch and still remain within the target ranges specified in AASHTOWare Pavement ME by NCDOT. The next column has the same design parameters as the original project with the Piedmont coarse aggregate concrete inputs. The two left columns have the Piedmont coarse aggregate concrete inputs with modified pavement thickness in half inch increments and dowel bar diameters to match based on the NCDOT Standard Drawing 700.01. As can be seen in Table 5.11, the new concrete inputs for Piedmont concrete facilitate sufficient performance of a thinner PCC section. However, a significant decrease in predicted performance occurs between section thicknesses of 9.5 inches and 9 inches due to the change in the dowel bar size of 1.25 inches to 1.125 inches.

Table 5.11: Evaluation of potential PCC thickness reduction using locally calibrated inputs for Piedmont region concrete

			NCDOT Project R-2536 Randolph Co.	NCDOT 2MS Manufactured Sand 9.5 inch	NCDOT 2MS Manufactured Sand 9 inch	NCDOT 2MS Manufactured Sand 8.5 inch
Layer 1: PCC	Pavement Thickness (in)		9.5	9.5	9	8.5
	Dowel Diameter (in)		1.25	1.25	1.125	1.125
	Cementitious Material Content (lb/yd^3)		600	550	550	550
	Water to cement ratio		0.42	0.48	0.48	0.48
	Unit Weight (PCF)		150	145	145	145
	28 Day Modulus of Rupture (psi)		650	660	660	660
	28 Day Modulus of Elasticity (psi)		4,200,000	3,000,000	3,000,000	3,000,000
	Poisson's Ratio		0.17	0.19	0.19	0.19
	Coefficient of Thermal Expansion (x 10^{-6} in/in°F)		6.00	4.63	4.63	4.63
	Heat Capacity (Btu/lb-°F)		0.28	0.20	0.20	0.20
	Thermal Conductivity (Btu/(ft)(hr)(°F))		1.25	0.95	0.95	0.95
Layer 2:			6 inches of Sandwich Granular			
Layer 3:			12 Inches of A-5 Subgrade			
Layer 4:			Semi-infinite layer of A-5 Subgrade			
Layer 5:			N.A.			
Climate Data			Chapel Hill, NC			
Distress	Terminal IRI (in/mile)	172.00 (Target)	136.13	123.79	142.48	142.82
	Mean Joint Faulting (in)	0.12 (Target)	0.07	0.05	0.08	0.08
	JPCP Transverse Cracking (percent slabs)	15.00 (Target)	7.98	4.39	5.38	7.35
Reliability	Terminal IRI (in/mile)		99.13	99.82	98.40	98.35
	Mean Joint Faulting (in)		99.81	99.99	99.23	99.37
	JPCP Transverse Cracking (percent slabs)		99.59	100.00	99.99	99.77

Project R-2536, shown in Table 5.12, was analyzed using the new suggested Piedmont coarse aggregate with natural sand concrete inputs. The target ranges for this project are as follows: terminal IRI of 172.00 in/mile, mean joint faulting of 0.12 inches, and transverse cracking of 15.00 %. The predicted performances of this analysis show that project R-2536 could potentially be reduced by up to an inch and still remain within the target ranges specified in AASHTOWare Pavement ME by NCDOT. The input values and predicted performance of the original project are listed in the yellow column. The next column has the same design parameters as the original project with the Piedmont coarse aggregate concrete inputs. The two left columns have the Piedmont coarse aggregate concrete inputs with modified pavement thickness in half inch increments and dowel bar diameters to match based on the NCDOT Standard Drawing 700.01. As can be seen in Table 5.12, the new concrete inputs for Piedmont concrete facilitate sufficient performance of a thinner PCC section. However, a significant decrease in predicted performance occurs between section thicknesses of 9.5 inches and 9 inches due to the change in the dowel bar size of 1.25 inches to 1.125 inches.

Table 5.12: Evaluation of potential PCC thickness reduction using locally calibrated inputs for Piedmont region concrete

			NCDOT Project R-2536 Randolph Co.	Natural Sand 9.5 inch	Natural Sand 9 inch	Natural Sand 8.5 inch
Layer 1: PCC	Pavement Thickness (in)		9.5	9.5	9	8.5
	Dowel Diameter (in)		1.25	1.25	1.125	1.125
	Cementitious Material Content (lb/yd^3)		600	550	550	550
	Water to cement ratio		0.42	0.48	0.48	0.48
	Unit Weight (PCF)		150	142	142	142
	28 Day Modulus of Rupture (psi)		650	740	740	740
	28 Day Modulus of Elasticity (psi)		4,200,000	3,300,000	3,300,000	3,300,000
	Poisson's Ratio		0.17	0.16	0.16	0.16
	Coefficient of Thermal Expansion (x 10^{-6} in/in°F)		6.00	5.40	5.40	5.40
	Heat Capacity (Btu/lb-°F)		0.28	0.20	0.20	0.20
	Thermal Conductivity (Btu/(ft)(hr)(°F))		1.25	1.20	1.20	1.20
Layer 2:			6 inches of Sandwich Granular			
Layer 3:			12 Inches of A-5 Subgrade			
Layer 4:			Semi-infinite layer of A-5 Subgrade			
Layer 5:			N.A.			
Climate Data			Chapel Hill, NC			
Distress	Terminal IRI (in/mile)	172.00 (Target)	136.13	130.92	154.54	153.21
	Mean Joint Faulting (in)	0.12 (Target)	0.07	0.07	0.11	0.10
	JPCP Transverse Cracking (percent slabs)	15.00 (Target)	7.98	3.83	4.57	5.23
Reliability	Terminal IRI (in/mile)		99.13	99.52	96.00	96.33
	Mean Joint Faulting (in)		99.81	99.88	94.85	95.72
	JPCP Transverse Cracking (percent slabs)		99.59	100.00	100.00	99.99

Project U-2519, shown in Table 5.13, was analyzed using the new suggested Piedmont coarse aggregate with manufactured sand concrete inputs. The predicted performances of this analysis show that project U-2519 could potentially be reduced by up to an inch and still remain within NCDOT's desired target ranges specified in AASHTOWare Pavement ME (terminal IRI of 185.00 in/mile, mean joint faulting of 0.12 inches, and transverse cracking of 10.00 %). The input values and predicted performance of the original project are listed in the yellow column. The next column has the same design as the original project with the exception of the new suggested inputs for the Piedmont coarse aggregate concrete inputs. The two left columns are the Piedmont coarse aggregate concrete inputs with modified pavement thickness in half inch increments and dowel bar diameters to match based on the NCDOT Standard Drawing 700.01. As can be seen in Table 5.13, the new concrete inputs for Piedmont concrete facilitate sufficient performance of a thinner PCC section. However, a significant decrease in predicted performance occurs between section thicknesses of 10 inches and 9.5 inches due to the change in the dowel bar size of 1.25 inches to 1.125 inches.

Table 5.13: Evaluation of potential PCC thickness reduction using locally calibrated inputs for Piedmont region concrete

			NCDOT Project U-2519 Cumberland Co.	NCDOT 2MS Manufactured Sand 10 inch	NCDOT 2MS Manufactured Sand 9.5 inch	NCDOT 2MS Manufactured Sand 9 inch
Layer 1: PCC	Pavement Thickness (in)		10	10	9.5	9
	Dowel Diameter (in)		1.25	1.25	1.125	1.125
	Cementitious Material Content (lb/yd^3)		600	550	550	550
	Water to cement ratio		0.42	0.48	0.48	0.48
	Unit Weight (PCF)		150	145	145	145
	28 Day Modulus of Rupture (psi)		690	660	660	660
	28 Day Modulus of Elasticity (psi)		4,200,000	3,000,000	3,000,000	3,000,000
	Poisson's Ratio		0.20	0.19	0.19	0.19
	Coefficient of Thermal Expansion (x 10^{-6} in/in°F)		6.00	4.63	4.63	4.63
	Heat Capacity (Btu/lb-°F)		0.28	0.20	0.20	0.20
	Thermal Conductivity (Btu/(ft)(hr)(°F))		1.25	0.95	0.95	0.95
Layer 2:			4.25 inches of Flexible Pavement			
Layer 3:			8 inches of Lime Stabilized			
Layer 4:			12 inches of A-6 Subgrade			
Layer 5:			Semi-infinite layer of A-6 Subgrade			
Climate Data			Fayetteville, NC			
Distress	Terminal IRI (in/mile)	185.00 (Target)	144.74	126.28	153.72	148.27
	Mean Joint Faulting (in)	0.12 (Target)	0.10	0.07	0.11	0.10
	JPCP Transverse Cracking (percent slabs)	10.00 (Target)	4.25	3.83	3.83	4.39
Reliability	Terminal IRI (in/mile)		99.24	99.94	98.32	98.94
	Mean Joint Faulting (in)		97.31	99.92	93.75	96.58
	JPCP Transverse Cracking (percent slabs)		99.87	99.96	99.96	99.83

Project U-2519, shown in Table 5.14, was analyzed using the new suggested Piedmont coarse aggregate with natural sand concrete inputs. The target ranges for this project are as follows: terminal IRI of 185.00 in/mile, mean joint faulting of 0.12 inches, and transverse cracking of 10.00 %. The input values and predicted performance of the original project are listed in the yellow column. The next column has the same design as the original project with the exception of the new suggested inputs for the Piedmont coarse aggregate concrete inputs with natural sand. The two left columns are the Piedmont coarse aggregate concrete inputs with modified pavement thickness in half inch increments and dowel bar diameters to match based on the NCDOT Standard Drawing 700.01. The predicted performances of this analysis indicate that using the new PCC input values, project U-2519 could not be reduced in thickness (due to failing at mean joint faulting predicted performance, shown in red in Table 5.14) while remaining within the target ranges specified in AASHTOWare Pavement ME by NCDOT.

Table 5.14: Evaluation of potential PCC thickness reduction using locally calibrated inputs for Piedmont region concrete

			NCDOT Project U-2519 Cumberland Co.	Natural Sand 10 inch	Natural Sand 9.5 inch	Natural Sand 9 inch
Layer 1: PCC	Pavement Thickness (in)		10	10	9.5	9
	Dowel Diameter (in)		1.25	1.25	1.125	1.125
	Cementitious Material Content (lb/yd^3)		600	550	550	550
	Water to cement ratio		0.42	0.48	0.48	0.48
	Unit Weight (PCF)		150	142	142	142
	28 Day Modulus of Rupture (psi)		690	740	740	740
	28 Day Modulus of Elasticity (psi)		4,200,000	3,300,000	3,300,000	3,300,000
	Poisson's Ratio		0.20	0.16	0.16	0.16
	Coefficient of Thermal Expansion (x 10^{-6} in/in°F)		6.00	5.40	5.40	5.40
	Heat Capacity (Btu/lb-°F)		0.28	0.20	0.20	0.20
	Thermal Conductivity (Btu/(ft)(hr)(°F))		1.25	1.20	1.20	1.20
Layer 2:			4.25 inches of Flexible Pavement			
Layer 3:			8 inches of Lime Stabilized			
Layer 4:			12 inches of A-6 Subgrade			
Layer 5:			Semi-infinite layer of A-6 Subgrade			
Climate Data			Fayetteville, NC			
Distress	Terminal IRI (in/mile)	185.00 (Target)	144.74	136.67	169.50	163.53
	Mean Joint Faulting (in)	0.12 (Target)	0.10	0.09	0.14	0.13
	JPCP Transverse Cracking (percent slabs)	10.00 (Target)	4.25	3.83	3.83	3.83
Reliability	Terminal IRI (in/mile)		99.24	99.69	95.18	96.62
	Mean Joint Faulting (in)		97.31	99.09	78.79	85.25
	JPCP Transverse Cracking (percent slabs)		99.87	99.96	99.96	99.96

Project U-2519, shown in Table 5.15, was analyzed using the new suggested Coastal coarse aggregate with manufactured sand concrete inputs. The predicted performances of this analysis show that the concrete pavement thickness for this Coastal project could potentially be reduced by up to an inch and still remain within the target ranges specified in AASHTOWare Pavement ME by NCDOT. The target ranges for this project are as follows: terminal IRI of 185.00 in/mile, mean joint faulting of 0.12 inches, and transverse cracking of 10.00 %. The input values and predicted performance of the original project are listed in the yellow column. The next column shows use of the same design parameters with the Coastal coarse aggregate concrete inputs. The two left columns show the use of the Coastal coarse aggregate concrete inputs with modified pavement thickness in half inch increments and dowel bar diameters to match based on the NCDOT Standard Drawing 700.01. As can be seen in Table 5.15, the new concrete inputs for Coastal concrete facilitate sufficient performance of a thinner PCC section. However, a significant decrease in predicted performance occurs between section thicknesses of 10 inches and 9.5 inches due to the change in the dowel bar size of 1.25 inches to 1.125 inches.

Table 5.15: Evaluation of potential PCC thickness reduction using locally calibrated inputs for Coastal region concrete

			NCDOT Project U-2519 Cumberland Co.	NCDOT 2MS Manufactured Sand 10 inch	NCDOT 2MS Manufactured Sand 9.5 inch	NCDOT 2MS Manufactured Sand 9 inch
Layer 1: PCC	Pavement Thickness (in)		10	10	9.5	9
	Dowel Diameter (in)		1.25	1.25	1.125	1.125
	Cementitious Material Content (lb/yd^3)		600	550	550	550
	Water to cement ratio		0.42	0.48	0.48	0.48
	Unit Weight (PCF)		150	139	139	139
	28 Day Modulus of Rupture (psi)		690	660	660	660
	28 Day Modulus of Elasticity (psi)		4,200,000	3,000,000	3,000,000	3,000,000
	Poisson's Ratio		0.20	0.19	0.19	0.19
	Coefficient of Thermal Expansion (x 10^{-6} in/in°F)		6.00	4.30	4.30	4.30
	Heat Capacity (Btu/lb-°F)		0.28	0.22	0.22	0.22
	Thermal Conductivity (Btu/(ft)(hr)(°F))		1.25	0.90	0.90	0.90
Layer 2:			4.25 inches of Flexible Pavement			
Layer 3:			8 inches of Lime Stabilized			
Layer 4:			12 inches of A-6 Subgrade			
Layer 5:			Semi-infinite layer of A-6 Subgrade			
Climate Data			Fayetteville, NC			
Distress	Terminal IRI (in/mile)	185.00 (Target)	144.74	125.39	152.61	147.49
	Mean Joint Faulting (in)	0.12 (Target)	0.10	0.06	0.11	0.10
	JPCP Transverse Cracking (percent slabs)	10.00 (Target)	4.25	3.83	3.83	4.25
Reliability	Terminal IRI (in/mile)		99.24	99.94	98.46	99.02
	Mean Joint Faulting (in)		97.31	99.94	94.38	96.86
	JPCP Transverse Cracking (percent slabs)		99.87	99.96	99.96	99.87

5.3 Summary of Findings

The AASHTOWare Pavement ME software facilitates rapid analysis of pavement sections using numerous changes in environment, service conditions, and materials characteristics. As illustrated in the previous sections, the predicted performance of a pavement changes with every change to the input values. In some cases, such as a change to the heat capacity input, the predicted changes in pavement performance are very small.

However, changes in other inputs (such as MOR, MOE, Poisson's ratio, CTE, and thermal conductivity) caused significant changes in the predicted performance of a pavement section. Ultimately, if NCDOT elects to utilize the new input values determined as part of this study, it will be important for NCDOT to gain a level of comfort in the impact of these new input values on the design (and predicted performance) of concrete pavements.

As outlined in the previous sections, it was consistently found that the predicted performances of pavement sections re-analyzed using the new suggested input values found through laboratory testing of concrete with locally available materials outperform those sections as designed using the input values for PCC currently utilized by NCDOT. This offers insight into the potentially longer service life of concrete pavements designed and constructed in the past by NCDOT. Additionally, use of the new PCC input values may also result in the design of slightly thinner concrete pavements in the future. Thinner pavements will reduce the amount of materials used in pavement construction, resulting in lower costs and lower environmental impact of concrete pavement.

CHAPTER 6: SENSITIVITY ANALYSIS

In this chapter, the results of a sensitivity analysis performed to evaluate the relative impact of changes in input values on predicted distresses for representative pavements are presented. This sensitivity analysis was performed using several projects selected by NCDOT (as detailed in Chapter 5), the AASHTOWare Pavement ME software, and the new concrete inputs identified through laboratory testing of concrete mixtures for North Carolina rigid pavements (as detailed in Chapter 4).

6.1 Objective of Sensitivity Analysis

To evaluate the impact that the new suggested MEPDG inputs for North Carolina concrete pavement mixtures will have on predicted performance of concrete pavements, and to compare the relative sensitivity of each input on predicted distress measures, a sensitivity analysis was conducted within the AASTHTOWare Pavement ME Design software. The concrete materials inputs determined through the laboratory testing program included in this study were utilized along with other inputs not the focus of this study (such as subgrade, base course, slab thickness, dowel placement, etc.) in pavement designs that are typical of each type of roadway (interstate, US and state routes, and local pavements) in the state of North Carolina. Data on traffic was selected to represent a typical traffic condition that could be reasonably expected on these types of pavements. Climate data for representative locations in each region was also utilized, as provided in Pavement ME's climate model database. Climate data for Greensboro, NC was selected for all of the analysis since Greensboro is a central location in the state in order to minimize fluctuations in the models to only the input that is being varied.

Design parameters for inputs not the focus of this study (e.g. all inputs except the concrete inputs) were held constant in each sensitivity analysis, while the PCC input values obtained from the testing performed in this research were varied one at a time. As the inputs were varied across the desired range (to be discussed subsequently in Section 6.2), the predicted distresses for each pavement section were compared. This approach facilitated evaluation of the impact that each of the different concrete mixture inputs (and hence the effect of the locally available materials) has on the predicted concrete pavement performances (failure modes, useful life, and deterioration). This approach also allowed identification of the concrete materials inputs that are most sensitive for North Carolina MEPDG pavement design.

6.2 Sensitivity Analysis Parameters

NCDOT provided four different pavement sections for this research (I-4400, U-2579, R-2536, and U-2519), and details of these four different pavement sections are provided in Chapter 5. The NCDOT design inputs for those four projects were used to develop the pavement design as can be seen in Figure 6.1, as well as the input parameters that were used as constants in the sensitivity analysis. As can be seen in Figure 6.1, the pavement design parameters maintained constant in this sensitivity analysis are as follows: layer 1 is a 10 inch JPCP, layer 2 is 8 inches of lime stabilized base, layer 3 is 10 inches of crushed gravel, and layer 4 is a semi-infinite A-6 soil subgrade. Input parameters held constant for the sensitivity analysis are typical of those utilized in the pavement sections provided by NCDOT, and are provided in Table 6.1 and Table 6.2.

Figure 6.1: Pavement design selected for sensitivity analysis

Table 6.1: Performance, traffic, and climate inputs held constant for sensitivity analysis

	Input parameter	Constant Value
	Design Life (years)	30
Performance Criteria	Initial IRI (in/mi)	63
	Terminal IRI (in/mi)	185
	Transverse cracking (% slabs cracked)	10
	Mean joint faulting (in)	0.12
Traffic Data for Analysis	Two-way AADT	6000
	Number of lanes in design direction	2
	Percent of trucks in design direction	50
	Percent of trucks in design lane	90
	Operational speed (mph)	65
	Average axle width (ft)	8.5
	Dual tire spacing (in)	12
	Tire Pressure (psi)	120
	Tandem axle spacing (in)	51.6
	Tridem axle spacing (in)	49.2
	Quad axle spacing (in)	49.2
	Mean wheel location (in)	18
	Traffic wander standard deviation (in)	10
	Design lane width (ft)	12
	Average axle spacing (short, medium, long) (ft)	12, 15, 18
	Percent of trucks (short, medium, long) (%)	17, 22, 61
	Climate location	Greensboro, NC

Table 6.2: Layer inputs held constant for sensitivity analysis

	Input parameter	Constant Value
JPCP Design Properties	Permanent curl/warp effective temperature difference (°F)	-10
	Joint spacing (ft)	15
	Sealant type	Preformed
	Dowel diameter (in)	1.25
	Dowel spacing (in)	12
	Widened slab	Not widened
	Tied shoulders	Tied
	Load transfer efficiency (%)	50
	Erodibility index	Erosion resistant (3)
	PCC-base contact friction	Full friction
	Friction loss (months)	240
	Surface shortwave absorptivity	0.85
Layer 1 PCC:	Layer thickness (in)	10
	Cementitious material content (pcy)	550
	Water/cement ratio	0.48
	Ultimate shrinkage (calculated) (microstrain)	Computed per input values
	Reversible shrinkage (% of ultimate shrinkage)	50
	Time to develop 50% of ultimate shrinkage (days)	35
	Curing method	Curing compound
Layer 2:	Layer 2:	Lime stabilized
	Thickness (in)	8
	Unit weight (pcf)	150
	Poisson's ratio	0.2
	Elastic/resilient modulus (psi)	45000
	Thermal conductivity (BTU/hr-ft-°F)	1.25
	Heat capacity (BTU/lb-°F)	0.28
Layer 3:	Layer 3:	Crushed gravel (A-1-a)
	Thickness (in)	10
	Poisson's ratio	0.35
	Coefficient of lateral earth pressure (k0)	0.5
	Elastic/resilient modulus (psi)	25000
Layer 4:	Layer 4:	A-6
	Thickness (in)	Semi-infinite
	Poisson's ratio	0.35
	Coefficient of lateral earth pressure (k0)	0.5
	Elastic/resilient modulus (psi)	14000

To determine the range of values to be utilized for each varied input, the data obtained as part of laboratory testing was used to identify a lower, median, and upper value, as well as upper and lower quartile values. Based on laboratory testing (results presented in Chapter 4) for the seven concrete inputs, values range as follows:

- unit weight ranges from 138 pcf to 150 pcf
- MOR ranges from 540 to 750 psi
- MOE ranges from 2,430,000 to 4,200,000 psi
- Poisson's ratio ranges from 0.15 to 0.23
- CTE ranges from 4.23 x 10^{-6} to 5.50 x 10^{-6} in/in°F
- thermal conductivity ranges from 0.80 to 1.25 Btu/(ft)(hr)(°F)
- heat capacity ranges from 0.20 to 0.28 Btu/lb-°F.

Each input that was modified for the sensitivity analysis is provided in Table 6.3, along with the minimum and maximum value obtained from laboratory testing, and the computed median and quartile values.

Table 6.3: Inputs that are varied for sensitivity analysis

Input Parameter	Sensitivity Analysis Variability Range				
	Lower	Lower Quartile	Median	Upper Quartile	Upper
Unit weight (pcf)	138	141	144	147	150
28-day PCC modulus of rupture (psi)	540	593	645	698	750
28-day PCC modulus of elasticity (psi)	2,430,000	2,872,500	3,315,000	3,757,500	4,200,000
Poisson's ratio	0.15	0.17	0.19	0.21	0.23
Coefficient of thermal expansion (x 10^{-6} in/in/°F)	4.23	4.55	4.87	5.18	5.50
Thermal conductivity (BTU/hr-ft-°F)	0.80	0.91	1.03	1.14	1.25
Heat Capacity (BTU/lb-°F)	0.20	0.22	0.24	0.26	0.28

To perform the sensitivity analysis, each PCC input shown in Table 6.3 was run individually using the batch processing capabilities of the AASHTOWare Pavement ME software. To accomplish this one-at-a-time sensitivity analyses, a total of 35 software simulations were run utilizing the AASHTOWare Pavement ME software. Each time the simulation was run, a series of graphs and results was produced. The results were tabulated and summarized in the following sections.

6.3 Results of Sensitivity Analysis

The results of the sensitivity analysis were rated a scale of "Very Sensitive," "Sensitive," and "Neutral" as shown in Table 6.4. To determine these sensitivity level thresholds, the maximum and minimum values of average rate of change were computed. From these values, reasonable ranges for "Very Sensitive," "Sensitive," and Neutral" were selected. "Very sensitive" is utilized to describe an input that, when varied, exhibits great influence on the individual predicted distress. "Sensitive" is utilized to describe an input that, when varied, results in a moderate change to the predicted distress. "Neutral" was utilized for inputs that, when varied over the specified range, had minimal to no impact on the predictive distresses.

Table 6.4: Sensitivity results overview

	Average Change in Distress		
	Terminal IRI (in/mile)	Mean Joint Faulting (in)	Transverse Cracking (% slabs cracked)
Very Sensitive	3.0 and up	0.01 and up	1.0 and up
Sensitive	2.99 to 1.00	0.009 to 0.001	0.99 to 0.10
Neutral	0.99 to 0.00	0.00	0.09 to 0.00

The following sections provide detailed results for the PCC inputs in AASHTOWare Pavement ME.

6.3.1 Unit Weight

Data from the sensitivity analysis for unit weight are provided in Table 6.5. In Figure 6.2, Figure 6.3, and Figure 6.4, graphical representations of the terminal IRI, mean joint faulting, and transverse cracking compared to each of the target values for this sensitivity analysis respectively, are provided. From Table 6.5, as well as Figures 6.2, 6.3, and 6.4, it can be seen that terminal IRI is "Very Sensitive" to unit weight, mean joint faulting is "Sensitive" to unit weight, and transverse cracking does not appear to have much sensitivity to unit weight ("Neutral"). It should be noted that as the input value for unit weight increases, the terminal IRI and mean joint faulting predicted distress decrease, indicating improved predicted performance with higher unit weight concrete.

Table 6.5: Unit weight sensitivity analysis results

Unit weight (pcf)	Distress		
	Terminal IRI (in/mile)	Mean Joint Faulting (in)	Transverse Cracking (% slabs cracked)
Target Value:	185.00	0.12	10.00
138	169.49	0.13	4.65
141	165.29	0.12	4.65
144	161.21	0.12	4.65
147	157.53	0.11	4.65
150	153.90	0.11	4.65
Average Change	3.90	0.005	0.00

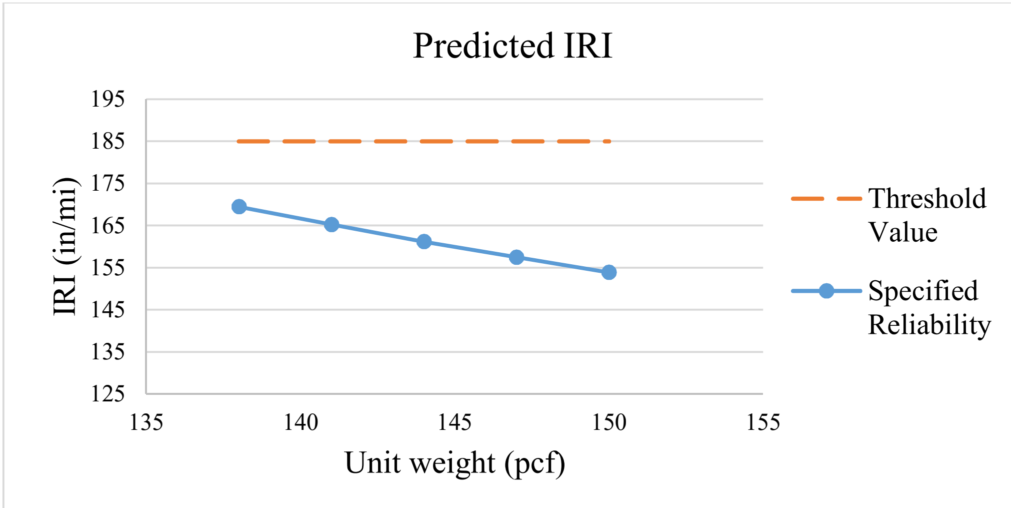


Figure 6.2: Unit weight predicted IRI

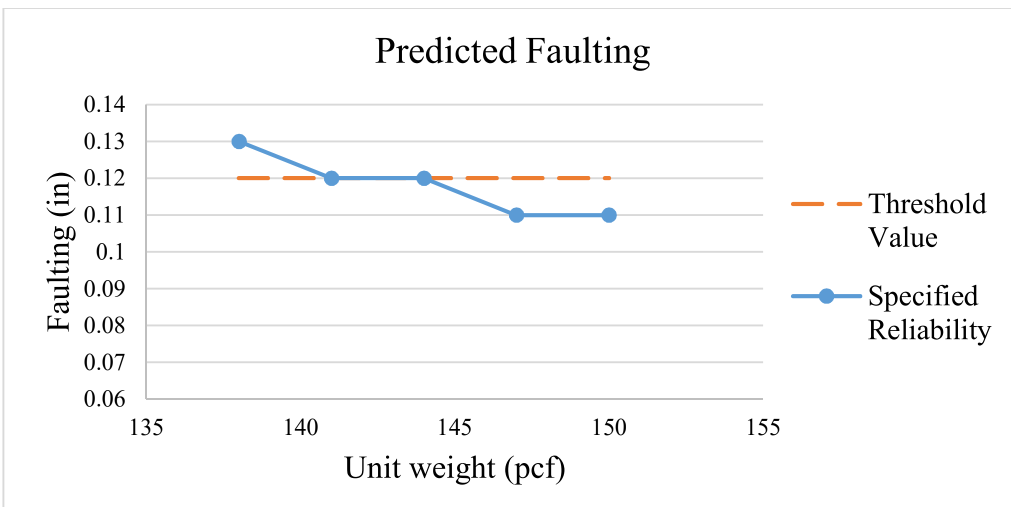


Figure 6.3: Unit weight predicted faulting

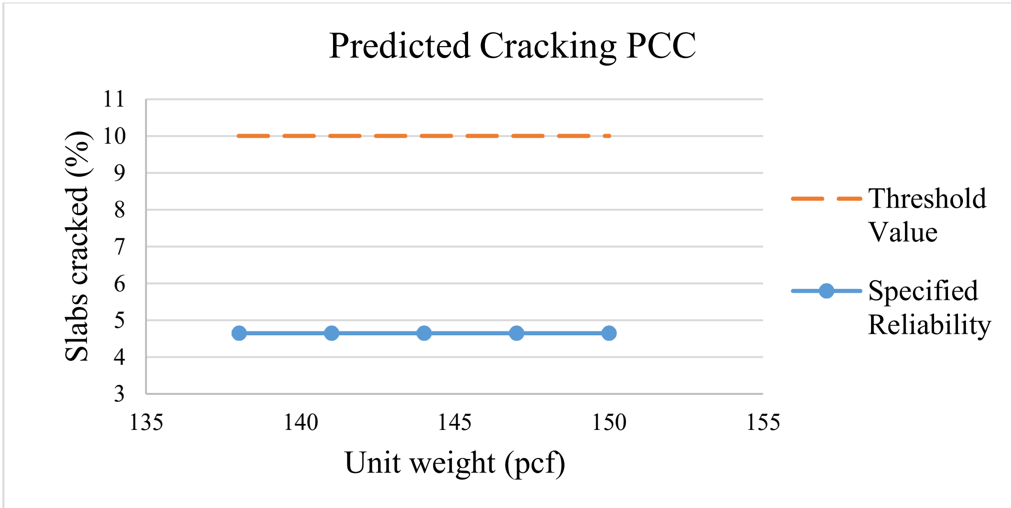


Figure 6.4: Unit weight predicted cracking PCC

6.3.2 Modulus of Rupture

Data from the sensitivity analysis for MOR are provided in Table 6.6. In Figure 6.5, Figure 6.6, and Figure 6.7, graphical representations of the terminal IRI, mean joint faulting, and transverse cracking compared to each of the target values for this sensitivity analysis respectively, are provided. From Table 6.6, as well as Figures 6.5, 6.6, and 6.7, it can be seen that terminal IRI and transverse cracking are "Very Sensitive" to MOR, whereas mean joint faulting does not appear to have much sensitivity to MOR ("Neutral"). It should be noted that as the input value for MOR increases, the terminal IRI and transverse cracking predicted distress decrease, indicating improved predicted performance with higher MOR.

It should also be noted that once the MOR input reaches into the NCDOT specified minimum strength (650 psi) the influence of this input is much less. The terminal IRI distress at that point falls into the "Sensitive" range, transverse cracking remains in the "Very Sensitive" range. When MOR is changed, the ultimate shrinkage input is also recalculated in the software (the values are coupled in the software, linked with a default algorithm). This influence is reflected in the predicted transverse cracking distresses being far less at MOR greater than 645 psi due to the reduced shrinkage calculated by the software.

Table 6.6: Modulus of rupture sensitivity analysis results

28-day PCC modulus of rupture (psi)	Distress		
	Terminal IRI (in/mile)	Mean Joint Faulting (in)	Transverse Cracking (% slabs cracked)
Target Value:	185.00	0.12	10.00
540	225.94	0.11	78.89
593	165.91	0.11	16.65
645	155.98	0.11	6.18
698	153.65	0.11	4.49
750	152.28	0.11	3.83
Average Change	18.42	0.000	18.77

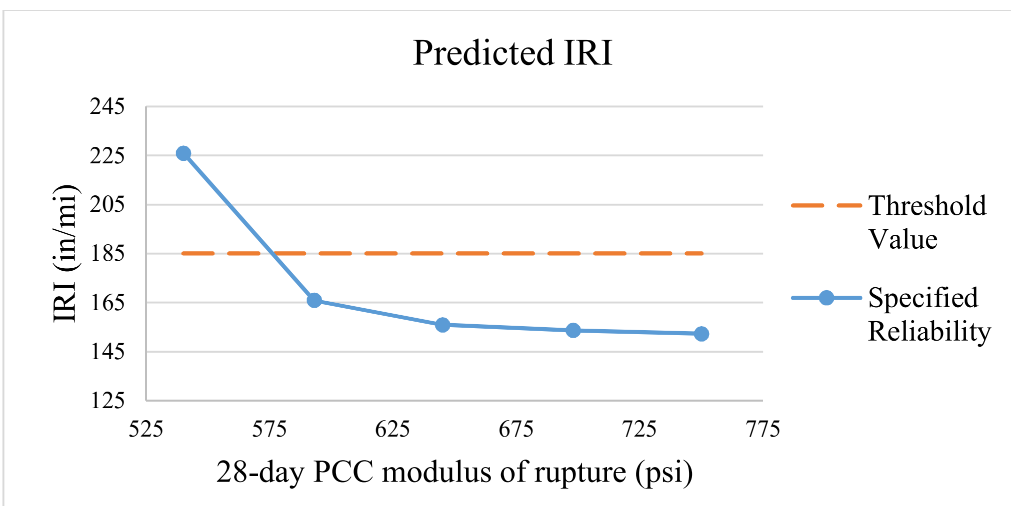


Figure 6.5: Modulus of rupture predicted IRI

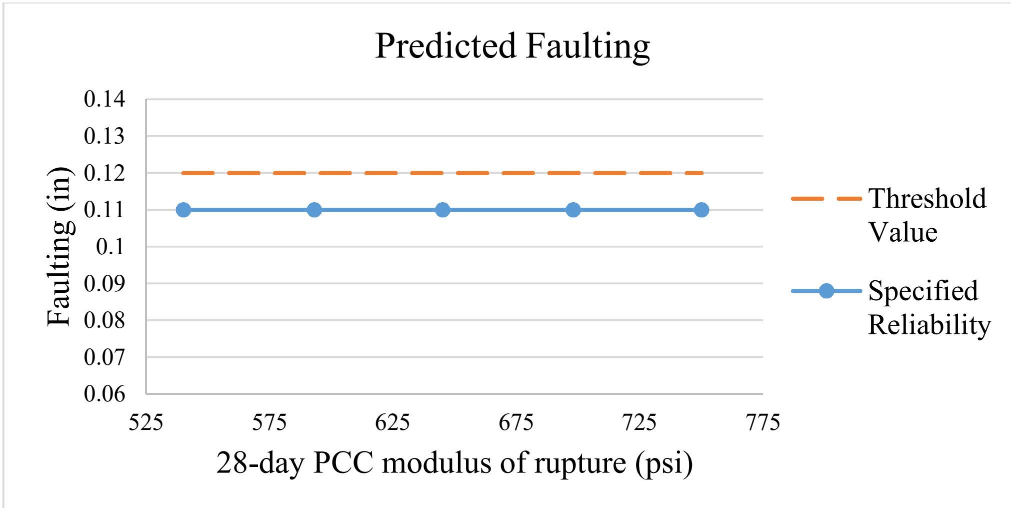


Figure 6.6: Modulus of rupture predicted faulting

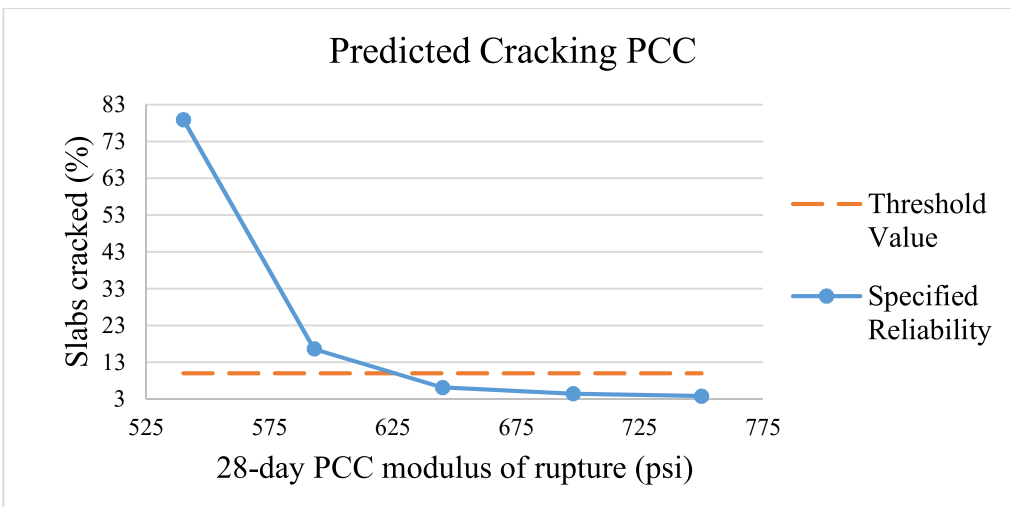


Figure 6.7: Modulus of rupture predicted cracking PCC

6.3.3 Modulus of Elasticity

Data from the sensitivity analysis for MOE are provided in Table 6.7. In Figure 6.8, Figure 6.9, and Figure 6.10, graphical representations of the terminal IRI, mean joint faulting, and transverse cracking compared to each of the target values for this sensitivity analysis respectively, are provided. From Table 6.7, as well as Figures 6.8, 6.9, and 6.10, it can be seen that terminal IRI, mean joint faulting, and transverse cracking are each "Sensitive" to MOE. It should be noted that as the input value for MOE increases, the terminal IRI, mean joint faulting, and transverse cracking predicted distress increase, indicating reduced predicted performance as the MOE increases.

Table 6.7: Modulus of elasticity sensitivity analysis results

28-day PCC modulus of elasticity (psi)	Distress		
	Terminal IRI (in/mile)	Mean Joint Faulting (in)	Transverse Cracking (% slabs cracked)
Target Value:	185.00	0.12	10.00
2,430,000	145.31	0.09	3.83
2,872,500	148.79	0.10	3.83
3,315,000	151.12	0.10	3.83
3,757,500	152.80	0.10	4.25
4,200,000	153.90	0.11	4.65
Average Change	2.15	0.005	0.21

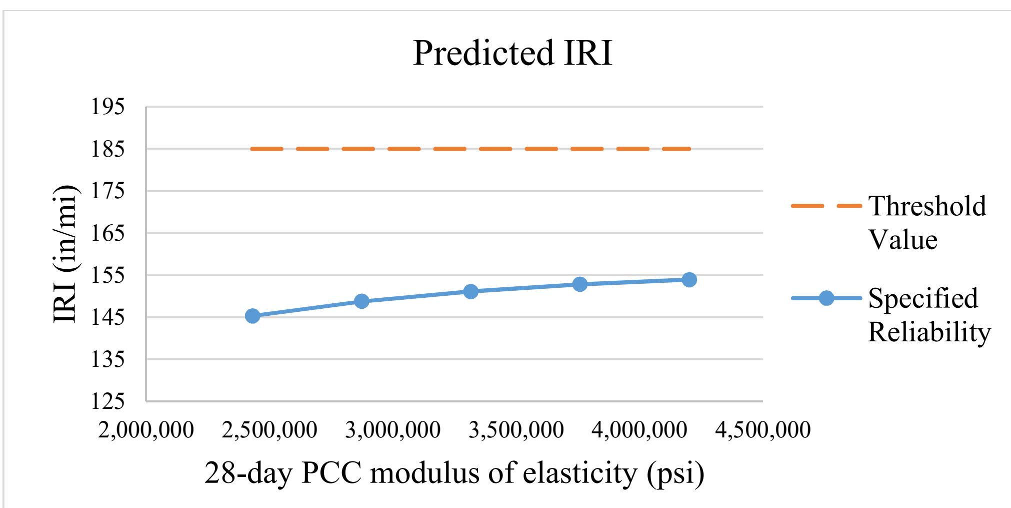


Figure 6.8: Modulus of elasticity predicted IRI

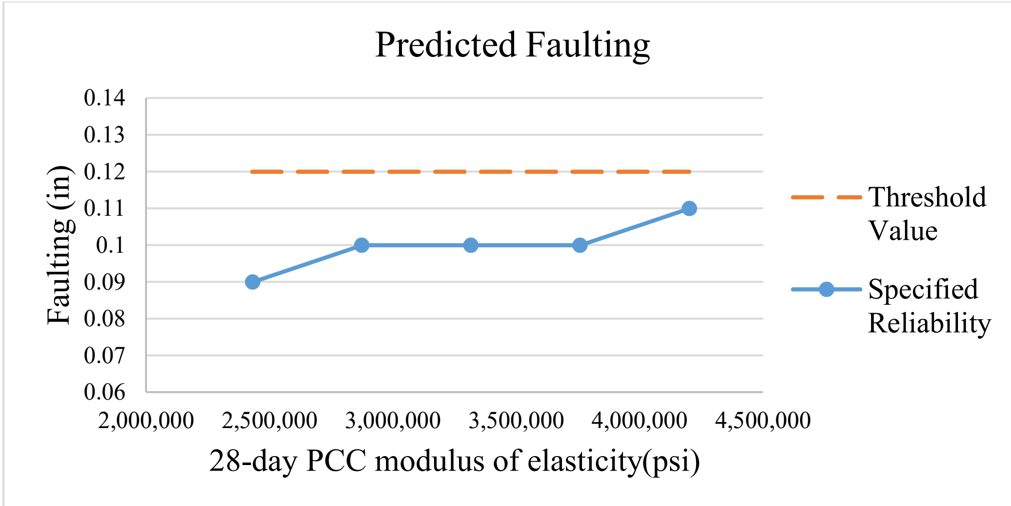


Figure 6.9: Modulus of elasticity predicted faulting

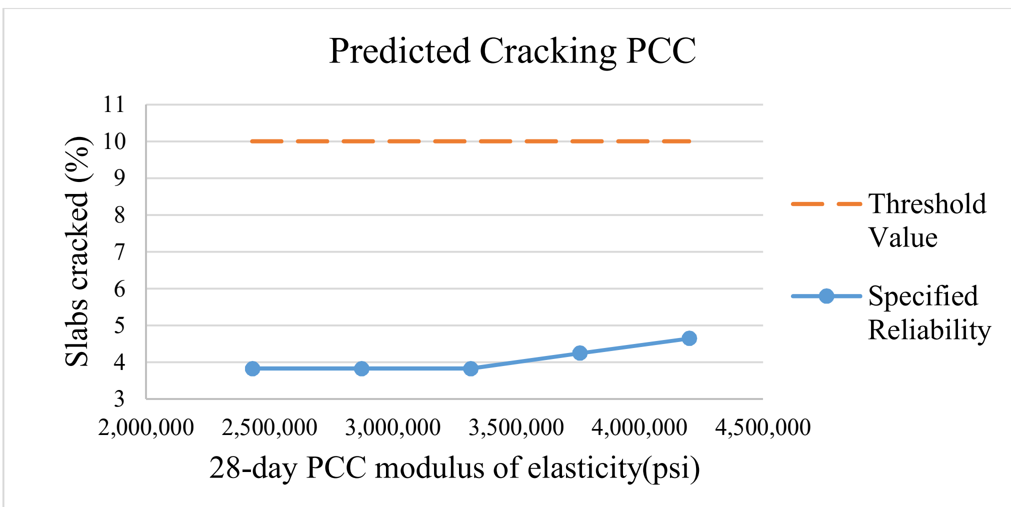


Figure 6.10: Modulus of elasticity predicted cracking PCC

6.3.4 Poisson's Ratio

Data from the sensitivity analysis for Poisson's ratio are provided in Table 6.8. In Figure 6.11, Figure 6.12, and Figure 6.13, graphical representations of the terminal IRI, mean joint faulting, and transverse cracking compared to each of the target values for this sensitivity analysis respectively, are provided. From Table 6.8, as well as Figures 6.11, 6.12, and 6.13, it can be seen that terminal IRI, mean joint faulting, and transverse cracking are each “Sensitive” to Poisson’s ratio. It should be noted that as the input value for Poisson’s ratio increases, the terminal IRI, mean joint faulting, and transverse cracking predicted distress increase, indicating reduced predicted performance as Poisson’s ratio is increased.

Table 6.8: Poisson’s ratio sensitivity analysis results

Poisson's ratio	Distress		
	Terminal IRI (in/mile)	Mean Joint Faulting (in)	Transverse Cracking (% slabs cracked)
Target Value:	185.00	0.12	10.00
0.15	149.37	0.10	4.39
0.17	151.19	0.10	4.49
0.19	153.00	0.10	4.57
0.21	154.88	0.11	4.71
0.23	156.78	0.11	4.88
Average Change	1.85	0.003	0.12

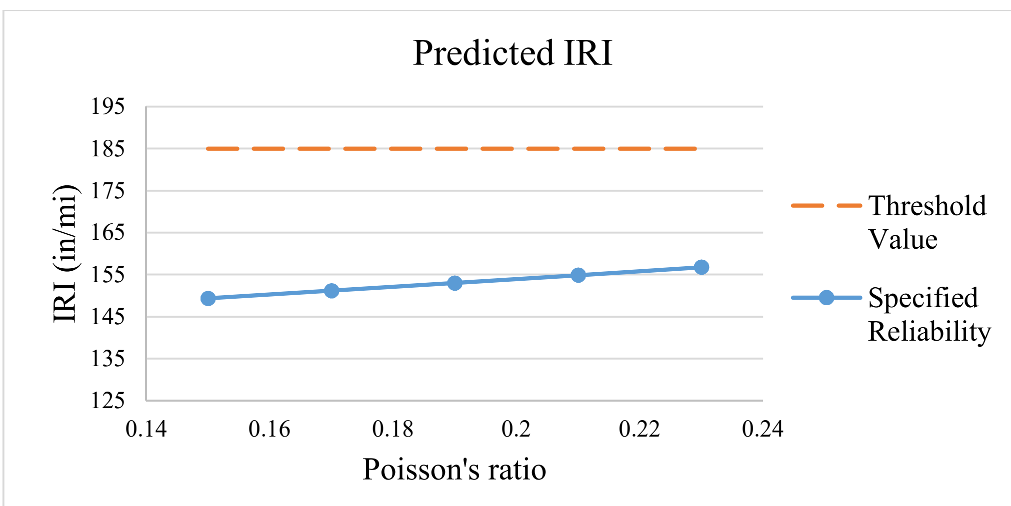


Figure 6.11: Poisson’s ratio predicted IRI

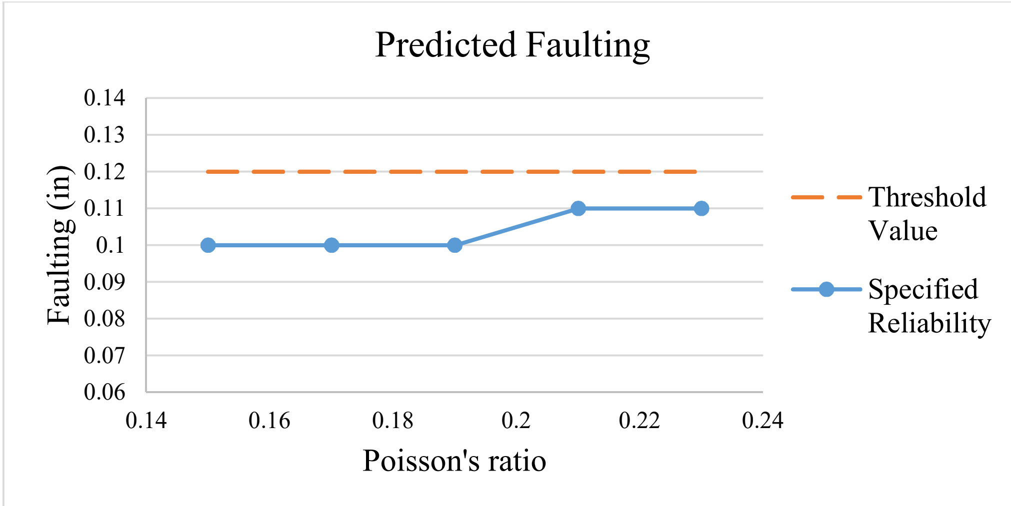


Figure 6.12: Poisson's ratio predicted faulting

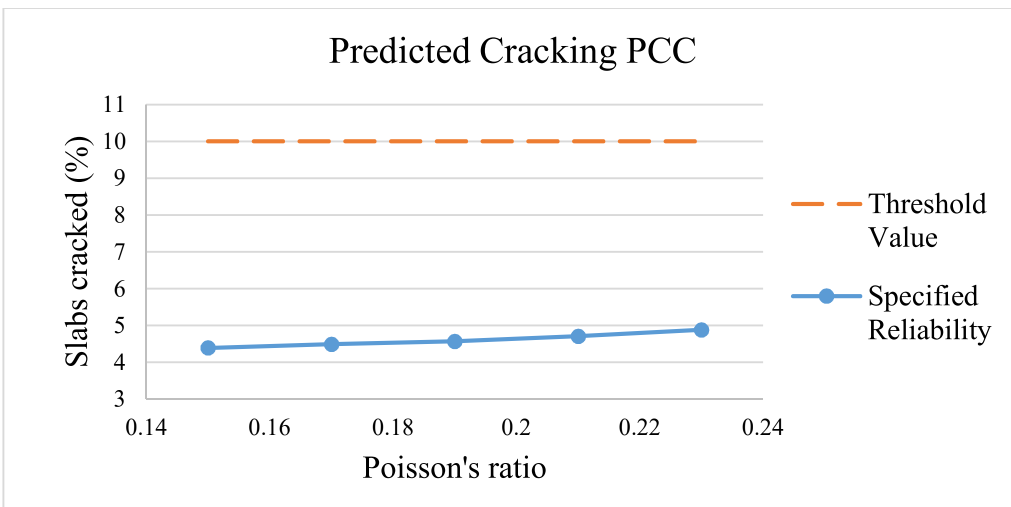


Figure 6.13: Poisson's ratio predicted cracking PCC

6.3.5 Coefficient of Thermal Expansion

Data from the sensitivity analysis for CTE are provided in Table 6.9. In Figure 6.14, Figure 6.15, and Figure 6.16, graphical representations of the terminal IRI, mean joint faulting, and transverse cracking compared to each of the target values for this sensitivity analysis respectively, are provided. From Table 6.9, as well as Figures 6.14, 6.15, and

6.16, it can be seen that terminal IRI and mean joint faulting are “Very Sensitive” to CTE, and transverse cracking is “Sensitive” to CTE. It should be noted that as the input value for CTE increases, the terminal IRI, mean joint faulting, and transverse cracking predicted distress increase, indicating reduced predicted performance.

Table 6.9: CTE sensitivity analysis results

Coefficient of thermal expansion (x 10^{-6} in/in/°F)	Distress		
	Terminal IRI (in/mile)	Mean Joint Faulting (in)	Transverse Cracking (% slabs cracked)
Target Value:	185.00	0.12	10.00
4.23	130.57	0.07	3.83
4.55	135.96	0.08	4.25
4.87	141.61	0.09	4.25
5.18	147.49	0.10	4.39
5.50	153.90	0.11	4.65
Average Change	5.83	0.010	0.21

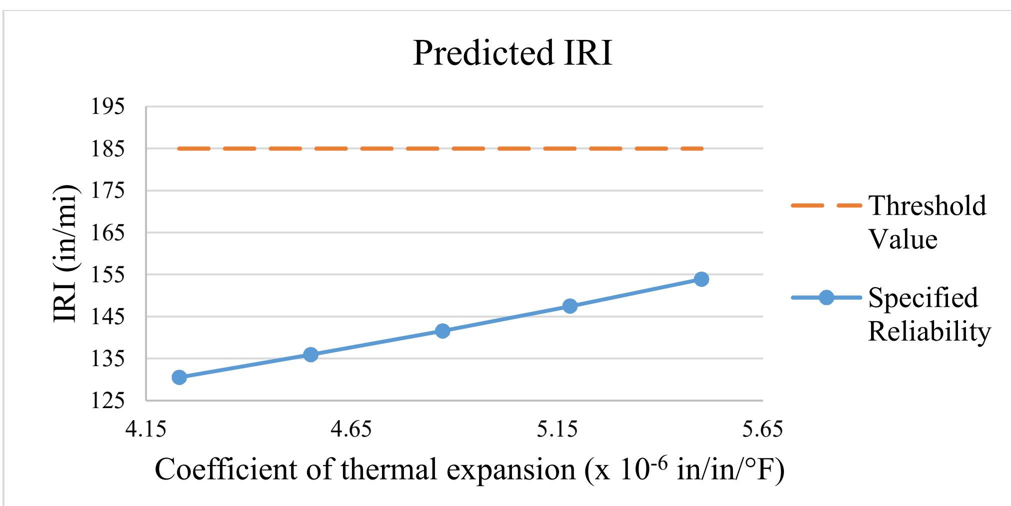


Figure 6.14: CTE predicted IRI

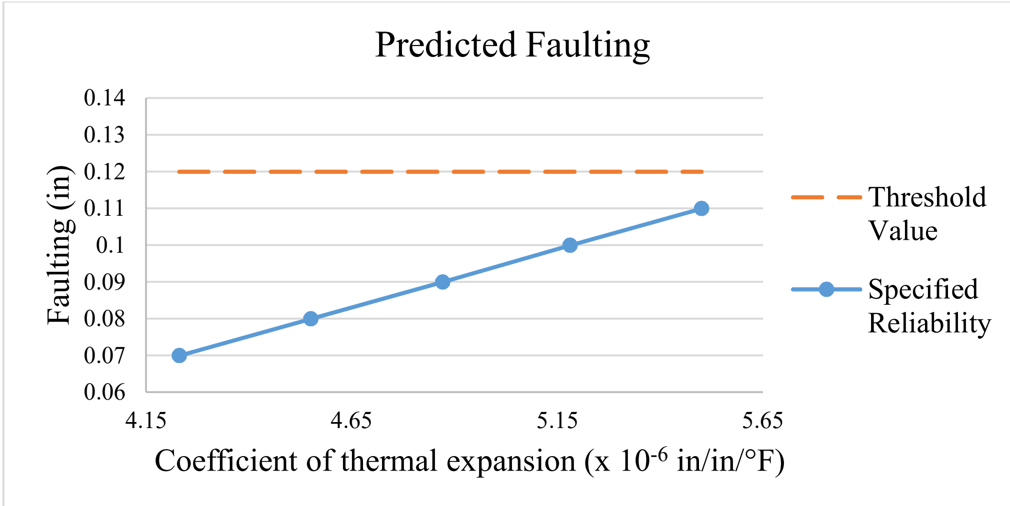


Figure 6.15: CTE predicted faulting

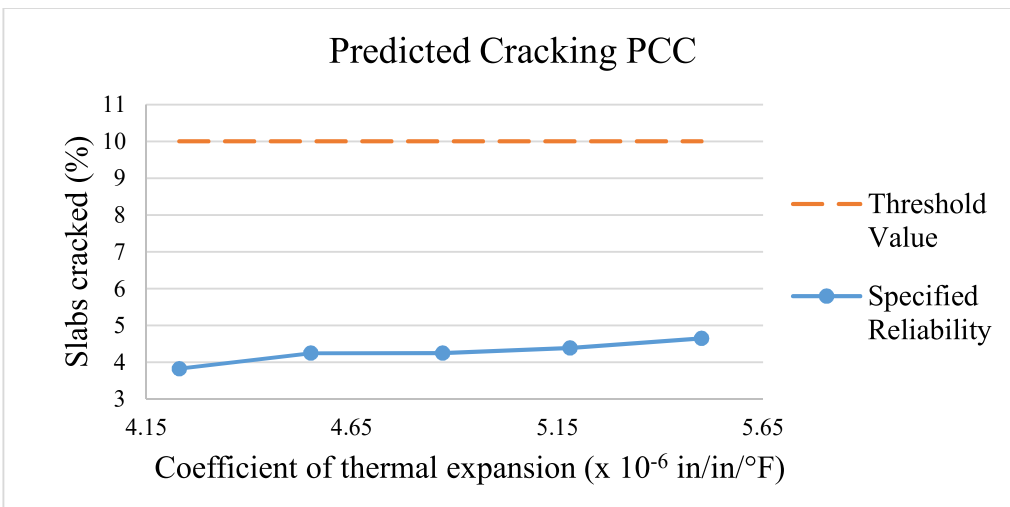


Figure 6.16: CTE predicted cracking PCC

6.3.6 Thermal Conductivity

Data from the sensitivity analysis for thermal conductivity are provided in Table 6.10. In Figure 6.17, Figure 6.18, and Figure 6.19, graphical representations of the terminal IRI, mean joint faulting, and transverse cracking compared to each of the target values for this sensitivity analysis respectively, are provided. From Table 6.10, as well as Figures

6.17, 6.18, and 6.19, it can be seen that terminal IRI does not appear to have much sensitivity to thermal conductivity (“Neutral”), mean joint faulting is “Sensitive” to thermal conductivity, and transverse cracking is “Very Sensitive” to thermal conductivity. It should be noted that as the input value for thermal conductivity increases, the mean joint faulting predicted distress increases and transverse cracking predicted distress decreases. However, as the input for thermal conductivity is increased, the terminal IRI predicted distress increases, then decreases, indicating an optimum range of inputs for optimum predicted performance exists for this input.

Table 6.10: Thermal conductivity sensitivity analysis results

Thermal conductivity (BTU/hr-ft-°F)	Distress		
	Terminal IRI (in/mile)	Mean Joint Faulting (in)	Transverse Cracking (% slabs cracked)
Target Value:	185.00	0.12	10.00
0.80	150.57	0.10	9.02
0.91	155.51	0.11	6.52
1.03	155.38	0.11	5.38
1.14	154.85	0.11	4.88
1.25	153.90	0.11	4.65
Average Change	0.83	0.003	1.09

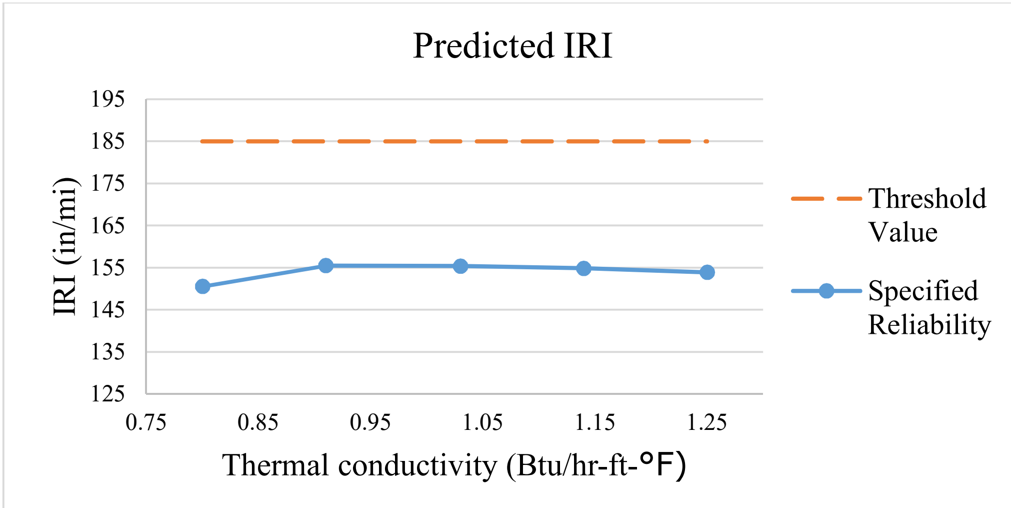


Figure 6.17: Thermal conductivity predicted IRI

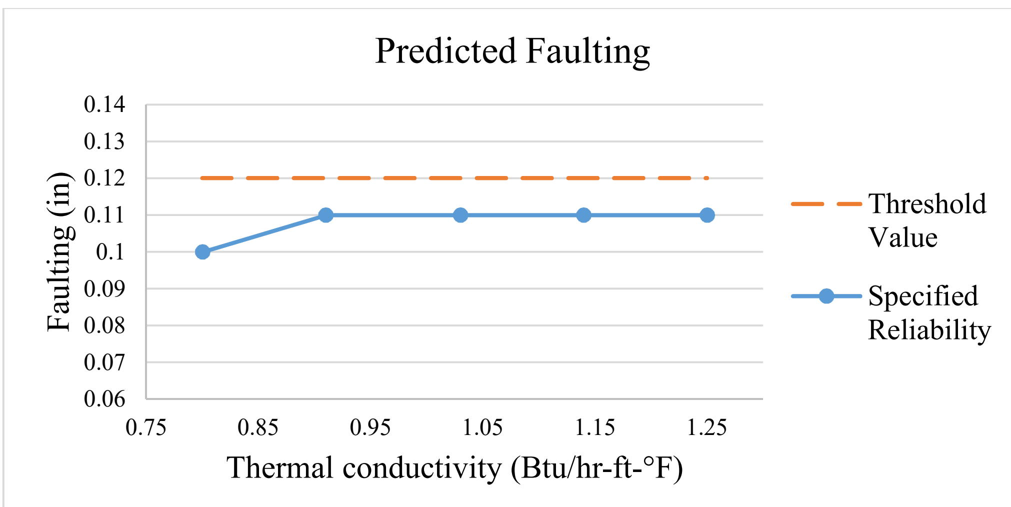


Figure 6.18: Thermal conductivity predicted faulting

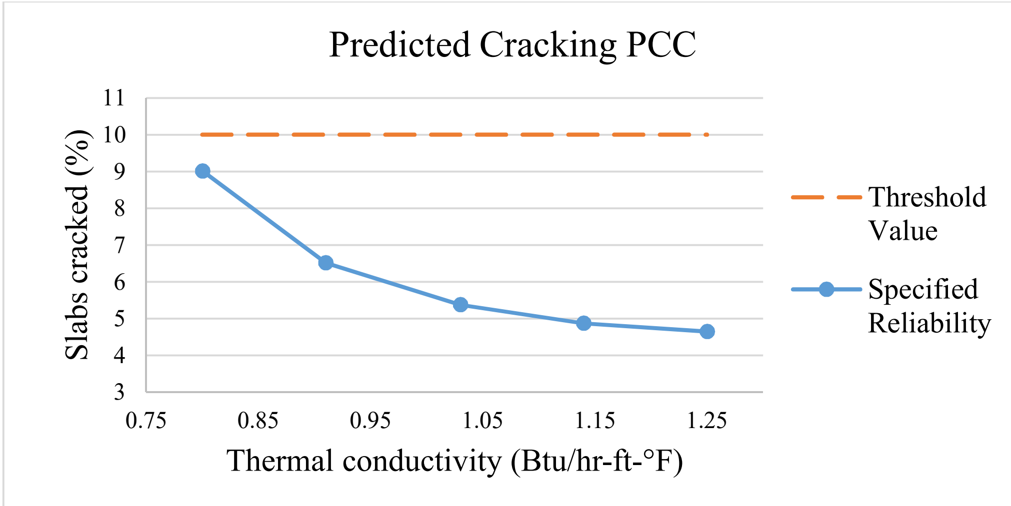


Figure 6.19: Thermal conductivity predicted cracking PCC

6.3.7 Heat Capacity

Data from the sensitivity analysis for heat capacity are provided in Table 6.11. In Figure 6.20, Figure 6.21, and Figure 6.22, graphical representations of the terminal IRI, mean joint faulting, and transverse cracking compared to each of the target values for this sensitivity analysis respectively, are provided. From Table 6.11, as well as Figures 6.20, 6.21, and 6.22, it can be seen that terminal IRI and mean joint faulting do not appear to have much sensitivity to heat capacity ("Neutral") and transverse cracking is "Sensitive" to heat capacity. It should be noted that as the input value for heat capacity increases, the terminal IRI and transverse cracking predicted distress decreases, indicating improved predicted performance with an increase in heat capacity.

Table 6.11: Heat capacity sensitivity analysis results

Heat Capacity (BTU/lb-°F)	Distress		
	Terminal IRI (in/mile)	Mean Joint Faulting (in)	Transverse Cracking (% slabs cracked)
Target Value:	185.00	0.12	10.00
0.20	157.01	0.11	5.79
0.22	156.28	0.11	5.19
0.24	155.37	0.11	4.88
0.26	154.81	0.11	4.71
0.28	153.90	0.11	4.65
Average Change	0.78	0.000	0.29

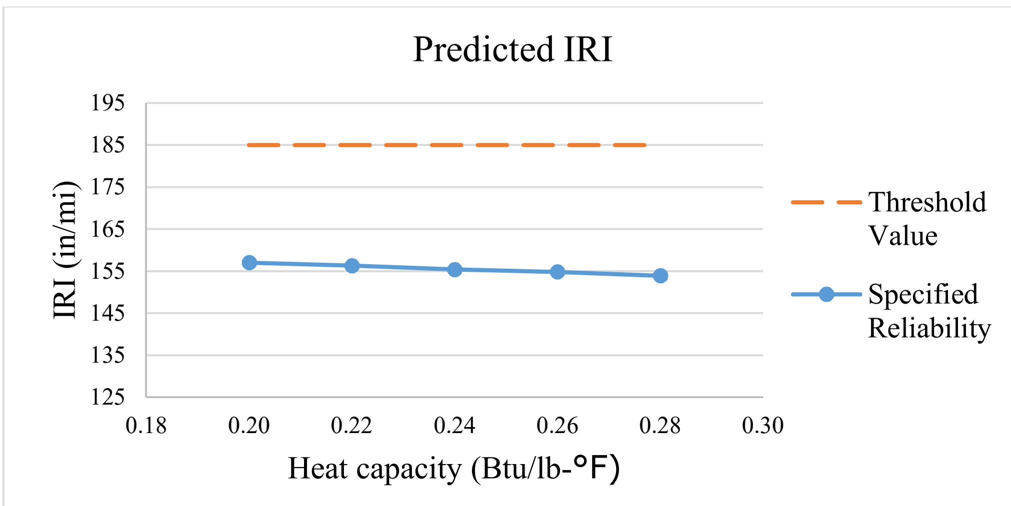


Figure 6.20: Heat capacity predicted IRI

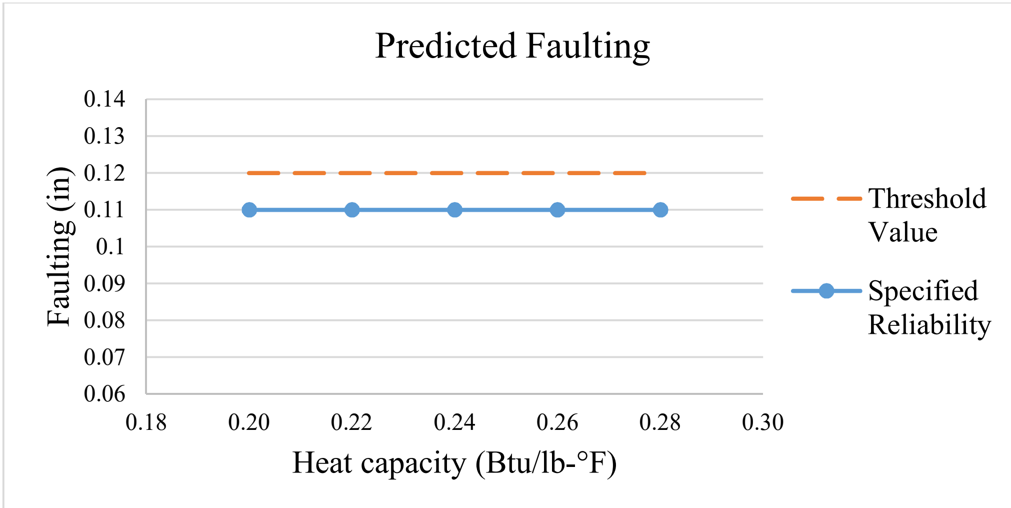


Figure 6.21: Heat capacity predicted faulting

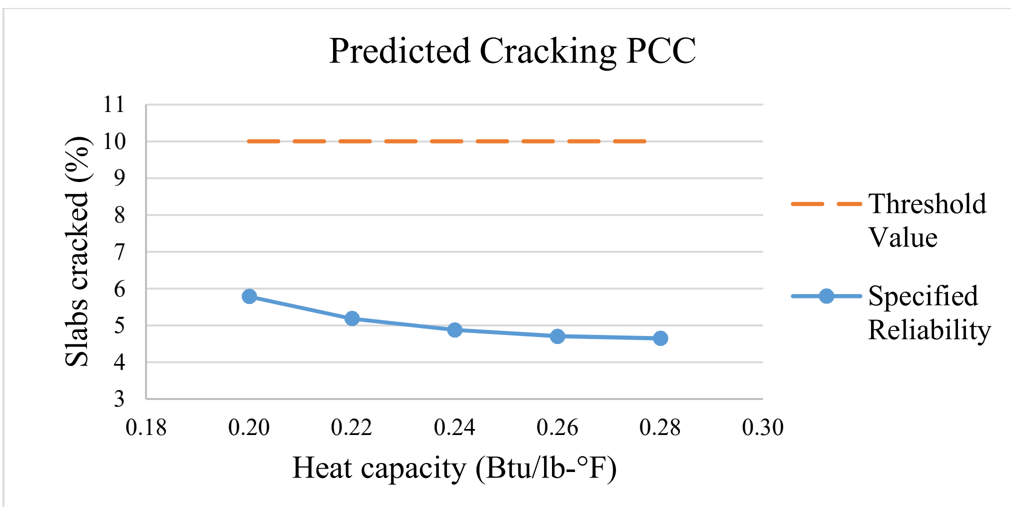


Figure 6.22: Heat capacity predicted cracking PCC

6.4 Summary of Findings

It is noted that based on this sensitivity analysis, overall, CTE was determined to be “Very Sensitive” for North Carolina concrete pavements for all modes of predicted distress, which is consistent with the findings of other researchers (McCarthy et al. 2014, Tanesi et al. 2007, Tran et al. 2008). Unit weight, MOR, MOE, Poisson’s ratio, thermal conductivity, and heat capacity were each determined to be “Sensitive” inputs for one or more distress modes. In a few cases, such as unit weight, MOR, CTE, and thermal conductivity, some distresses were found to be “Very Sensitive” to one or more inputs. A summary of the sensitivity of the pavement section’s predicted performance to changes in input values is provided in Table 6.12. In this table, indication of the relative impact (increase or decrease) observed for each of the predicted distresses when the input value in column 1 is increased (denoted by the up arrow). When a distress is decreased, it is providing an improved predicted performance.

Table 6.12: The effect on predicted distress by increasing each input value

Input	Terminal IRI (in/mile)	Mean Joint Faulting (in)	Transverse Cracking (% slabs cracked)
Unit weight ↑	Decrease	Decrease	Decrease
Modulus of rupture ↑	Decrease	Neutral	Decrease
Modulus of elasticity ↑	Increase	Increase	Increase
Poisson's ratio ↑	Increase	Increase	Increase
CTE ↑	Increase	Increase	Increase
Thermal conductivity ↑	Increase, then decrease	Increase	Decrease
Heat Capacity ↑	Decrease	Neutral	Decrease

Overall, the results of this sensitivity analysis can be utilized to aid in identification of concrete properties or characteristics that could be modified to achieve performance goals. This information could also be helpful in specification of concrete characteristics that could potentially provide improved concrete pavement performance.

CHAPTER 7: CONCLUSIONS AND RECOMMENDATIONS

This study included the batching and laboratory testing of eighteen different concrete mixtures produced using a variety of materials local to North Carolina to provide data to support identification of new, Level 1 PCC inputs for use in MEPDG pavement design using AASHTOWare Pavement ME software. Laboratory test results for the matrix of mixtures were analyzed, and a proposed catalog of inputs was proposed for consideration for use by NCDOT. This catalog of inputs, along with the suite of data from supporting laboratory test results, provide confidence to North Carolina Pavement designers about inputs for use in design of PCC pavements. Key findings from the laboratory testing are:

- The cement type (OPC or PLC) used in the concrete mixture does not highly influence the laboratory test results for the suite of tests used to determine the MEPDG PCC inputs.
- The fine aggregate utilized in the concrete mixture (manufactured sand versus natural sand) had significant influence on the workability of the concrete as well as the thermal properties used for MEPDG PCC inputs.
- Although coarse aggregates vary greatly across North Carolina, the type of coarse aggregate utilized in this study did not highly influence the laboratory test results for the suite of tests used to determine the MEPDG PCC inputs.

- Use of fly ash in concrete pavement mixtures may make it unsuitable to utilize the 28-day strength as a PCC input in MEPDG due to the delayed strength gain.

Ultimately, if NCDOT elects to utilize the new input values determined as part of this study, it will be important for NCDOT to gain a level of comfort in the impact of these new input values on the design (and predicted performance) of concrete pavements. The AASHTOWare Pavement ME software facilitates rapid analysis of pavement sections using numerous changes in environment, service conditions, and materials characteristics, and changes in input values result in changes in the predicted performance of a pavement. In some cases, such as a change to the heat capacity input, the predicted changes in pavement performance are very small. However, changes in other inputs (such as MOR, MOE, Poisson's ratio, CTE, and thermal conductivity) caused significant changes in the predicted performance of a pavement section.

In Chapter 5, a variety of typical North Carolina concrete pavements was analyzed using previous and newly suggested PCC inputs using the original design constraints. Through this analysis, it was consistently found that the predicted performances of pavement sections re-analyzed using the new suggested input values found through laboratory testing of concrete with locally available materials outperform those sections as designed using the input values for PCC currently utilized by NCDOT. This offers insight into the potentially longer service life of concrete pavements designed and constructed in the past by NCDOT. Additionally, use of the new PCC input values may also result in the design of slightly thinner concrete pavements in the future. Thinner pavements will reduce the amount of materials used in pavement construction, resulting in lower costs and environmental impact of concrete pavement.

In Chapter 6, a sensitivity analysis was performed using AASHTOWare Pavement ME to compare the relative sensitivity of each input on predicted pavement distresses overall, CTE was determined to be "Very Sensitive" for North Carolina concrete pavements for all modes of predicted distress, which is consistent with the findings of other researchers (McCarthy et al. 2014, Tanesi et al. 2007, Tran et al. 2008). Unit weight, MOR, MOE, Poisson's ratio, thermal conductivity, and heat capacity were each determined to be "Sensitive" inputs for one or more distress modes. In a few cases, such as unit weight, MOR, CTE, and thermal conductivity, some distresses were found to be "Very Sensitive" to one or more inputs. The results of this sensitivity analysis could be utilized to aid in identification of concrete properties or characteristics that could be modified to achieve performance goals. This information could also be helpful in specification of concrete characteristics that could potentially provide improved concrete pavement performance.

Recommendations for future work include similar laboratory testing of concrete produced using natural sand with the Coastal and Mountain coarse aggregates in order to see how the natural sand changes the new suggested PCC inputs for the AASHTOWare Pavement ME software. Based on feedback from industry received during the course of the project, another recommendation is that a similar laboratory testing program using concrete mixtures batched with a blend of natural sand and manufactured sand (paired with the Mountain, Piedmont, and Coastal coarse aggregate).

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APPENDIX A: SUPPLEMENTAL INFORMATION FOR CHAPTER 3

CEMENT MILL TEST REPORT

Cement Identified as: **Type I LA, Type II LA** Date: **10/1/2014**

Plant:

Location:

Production Dates:

Beginning: **10/1/2014**

Ending: **Silos: 14**

CHEMICAL REQUIREMENTS (ASTM C 114)	ASTM C 150 & AASHTO M85 SPEC'S	TYPE I (ASTM, AASHTO)	TYPE II (ASTM, AASHTO)	TYPE I LA (ASTM, AASHTO)	TEST RESULTS
Silicon Dioxide (SiO2), %	Minimum	----	----	----	20.3
Aluminum Oxide (Al2O3), %	Maximum	----	6.0	----	4.7
Ferric Oxide (Fe2O3), %	Maximum	----	6.0	----	3.3
Calcium Oxide (CaO), %		----	----	----	64.1
Magnesium Oxide (MgO), %	Maximum	6.0	6.0	6.0	1.2
Sulfur Trioxide (SO3), % **	Maximum	3.5	3.0	3.5	3.0
Loss on Ignition (LOI), %	Maximum	3.0	3.0	3.0	1.6
Insoluble Residue, %	Maximum	0.75	0.75	0.75	0.30
Alkalies (Na2O equivalent), %	Maximum			0.60	0.54
Tricalcium Silicate (C3S), %	Maximum	----	----	----	58
Tricalcium Aluminate (C3A), %	Maximum	----	8	----	7
C3S + 4.75(C3A), %	Maximum	----	100	----	92
PHYSICAL REQUIREMENTS					
(ASTM C 204) Blaine Fineness, M2/Kg	Minimum	280	280	280	4074
(ASTM C 191) Time of Setting (Vicat)					
Initial Set, minutes	Minimum	45	45	45	115
Final Set, minutes	Maximum	375	375	375	210
(ASTM C 451) False Set, %	Minimum	50	50	50	85
(ASTM C 185) Air Content, %	Maximum	12	12	12	6
(ASTM C 151) Autoclave Expansion, %	Maximum	0.80	0.80	0.80	-0.01
(ASTM C 1038) Expansion in Water, %at 3.6 SO3	Maximum	0.02	0.02	0.02	0.001
(ASTM C186) 7 day Heat of Hydration, (cal/g)					73
(ASTM C 109) Compressive Strength, psi (MPa)					
1 Day		----	----	----	2530 (17.4)
3 Day	Minimum	1740(12.0)	1450(10.0)	1740(12.0)	3560(24.5)
7 Day	Minimum	2760(19.0)	2470(17.0)	2760(19.0)	4530 (31.2)
*28 Day	Minimum	----	----	----	6370 (43.9)

*** The performance of Type I/II has proven to be improved with sulfur trioxide levels in excess of the 3.0% limit for Type II. Note D in ASTM C-150 allows for additional sulfate, provided expansion as measured by ASTM C-1038 does not exceed 0.020%.*

Satisfies the requirements of VDOT Standard Road & Bridge specification section 214

(*) Tests results for this period not available. Most recent test results provided

hereby certifies that this cement meets or exceeds the chemical and physical Specifications of:

Physical testing completed by:
Chemical testing completed by:

x ASTM C-150 for Type I
x ASTM C-150 for Type II
x ASTM C-150 for Type II M.H.
x ASTM C-150 for Type I L.A.
x AASHTO M85 for SCDOT Type I LA
x AASHTO M85 for Type I
x AASHTO M85 for Type II
x ASTM C-1157 for Type GU

By
Quality Control Manager

is not responsible for the improper use or workmanship associated with the use of this cement.

Figure A.1: Mill report for OPC1

Samples for UNC Charlotte

	UNCC	UNCC
Sample Type	I-II	IL
Sample ID		
Date Tested at HH	1/20/2015	1/13/2015
% Limestone	3.4	10.2
Blaine	406	530
SiO2	20.33	19.83
Al2O3	4.93	4.29
Fe2O3	3.46	3.45
CaO	64.46	64.32
MgO	1.56	1.38
SO3	3.29	3.46
Na2O	0.18	0.15
K2O	0.59	0.47
NaEq	0.57	0.46
C3S	60.5	
C2S	12.7	
C3A	7.2	
C4AF	10.5	
1 Day psi	2580	2690
3 Day psi	4340	4520
7 Day psi	5250	5610
28 Day psi	6400	6590

Please Note: The Bogue phase calculations are not corrected for Limestone addition.

Figure A.2: Mill report for OPC2 and PLC

Figure A.3: Fly ash A report

Date: February 10, 2016
I.D.:
Lab No.:

REPORT OF FLY ASH TESTS			
Date Sampled: DS 11/23-12/11; Manufacturer: Roxboro	Start Date: November 23, 2015; End Date: December 11, 2015; Date Received: December 16, 2015		
	Results	Specification (Class F)	
Chemical Analysis**	(wt%)	ASTM C618-15	AASHTO M295-11
Silicon Dioxide (SiO_2)	53.8	----	----
Aluminum Oxide (Al_2O_3)	27.5	---	----
Iron Oxide (Fe_2O_3)	8.05	----	----
Sum of Silicon Dioxide, Iron Oxide & Aluminum Oxide (SiO_2+Al_2O_3+Fe_2O_3)	89.3	70 % min.	70 % min.
Calcium Oxide (CaO)	2.3	----	----
Magnesium Oxide (MgO)	1.0	----	----
Sodium Oxide (Na_2O)	0.45	----	----
Potassium Oxide (K_2O)	2.44	---	----
"Sodium Oxide Equivalent (Na_2O+0.658K_2O)"	2.05	----	----
Sulfur Trioxide (SO_3)	0.62	5 % max.	5 % max.
Loss on Ignition	2.1	6 % max.	5 % max.
Moisture Content	0.18	3 % max.	3 % max.
Available Alkalies**			
Sodium Oxide (Na_2O) as Available Alkalies	0.16	----	----
Potassium Oxide (K_2O) as Available Alkalies	0.71	----	----
Available Alkalies as "Sodium Oxide Equivalent (Na_2O+0.658K_2O)"	0.63	----	1.5 % max.*
Physical Analysis			
Fineness (Amount Retained on #325 Sieve)	21.9%	34 % max.	34 % max.
Strength Activity Index with Portland Cement			
At 7 Days: Control Average, psi: 4820; Test Average, psi: 3780	78%	75 % min.† (of control)	75 % min.† (of control)
At 28 Days: Control Average, psi: 6100; Test Average, psi: 5190	85%	75 % min.† (of control)	75 % min.† (of control)
Water Requirements (Test H_2O/Control H_2O): Control, mls: 242; Test, mls: 236	98%	105 % max. (of control)	105 % max. (of control)
Autoclave Expansion:	-0.03%	± 0.8 % max.	± 0.8 % max.
Specific Gravity:	2.21	----	----

† Meeting the 7 day or 28 day strength activity index will indicate specification compliance
* Optional
**Chemical Analysis performed by

Figure A.4: Fly ash B report

Date: January 30, 2015
Project No:
Laboratory No:

REPORT OF FLY ASH TESTS

Date Sampled: DS 12/11-12/16
Manufacturer: Belews Creek

Start Date: December 11, 2014
End Date: December 16, 2014
Date Received: December 22, 2014

Chemical Analysis**	Results	Specification (Class F) ASTM C618-12a	Specification (Class F) AASHTO M295-11
Silicon Dioxide	**53.21**	----	----
Aluminum Oxide	**28.74**	----	----
Iron Oxide	**7.64**	----	----
Sum of Silicon Dioxide, Iron Oxide & Aluminum Oxide	**89.59**	70 % min.	70 % min.
Calcium Oxide	**1.74**	----	----
Magnesium Oxide	**0.92**		
Sulfur Trioxide	**0.38**	5 % max.	5 % max.
Loss on Ignition	**2.61**	6 % max.	5 % max.
Moisture Content	**0.10**	3 % max.	3 % max.
Available Alkalies as Na_2O	**0.42**	----	1.5 % max.*
Sodium Oxide	**0.11**	----	----
Potassium Oxide	**0.47**	----	----
Physical Analysis			
Fineness (Amount Retained on #325 Sieve)	**13.3%**	34 % max.	34 % max.
Strength Activity Index with Portland Cement			
At 7 Days: Control Average, psi: 4930; Test Average, psi: 3840	**78%**	75 % min.† (of control)	75 % min.† (of control)
At 28 Days: Control Average, psi: 6150; Test Average, psi: 5540	**90%**	75 % min.† (of control)	75 % min.† (of control)
Water Requirements (Test H_2O/Control H_2O): Control, mls: 242; Test, mls: 236	**98%**	105 % max. (of control)	105 % max. (of control)
Autoclave Expansion	**0.03%**	± 0.8 % max.	± 0.8 % max.
Specific Gravity:	**2.29**	----	----

† Meeting the 7 day or 28 day strength activity index will indicate specification compliance
* Optional Requirement
**Chemical Analysis performed by

Table A.1: Mountain coarse aggregate sieve analysis

Sieve Size	Percent Passing	ASTM C 33 Specification, % Passing
1"	98.8%	100
3/4"	81.8%	90-100
1/2"	27.9%	---
3/8"	11.9%	20-55
No.4	3.5%	0-10
No.8	0.8%	0-5
No.200 Decant, %:	0.4%	1.0/1.5[1]

Table A.2: Piedmont coarse aggregate sieve analysis

Sieve Size	Percent Passing	ASTM C 33 Specification, % Passing
1"	100	100
3/4"	96	90-100
1/2"	55	---
3/8"	33	20-55
No.4	5	0-10
No.8	2	0-5
No.200 Decant, %:	0.3	1.0/1.5[1]

Table A.3: Coastal coarse aggregate sieve analysis

Sieve Size	Percent Passing	ASTM C 33 Specification, % Passing
1"	97.8%	100
3/4"	76.9%	90-100
1/2"	38.3%	---
3/8"	24.0%	20-55
No.4	5.6%	0-10
No.8	1.4%	0-5
No.200 Decant, %:	0.3%	1.0/1.5[1]

Table A.4: Manufactured sand sieve analysis

Sieve Size	Percent Passing	NCDOT 2MS Specification Percent Passing (%)
3/8	100.0%	100.0%
No. 4	100.0%	95-100%
No. 8	85.0%	80-100%
No. 16	64.0%	45-95%
No. 30	47.0%	25-75%
No. 50	30.0%	5-35%
No. 100	14.0%	0-20%
No. 200	5.2%	0-10%

Table A.5: Natural sand sieve analysis

Sieve Size	Percent Passing	ASTM C 33 Specification, % Passing
3/8	100.0%	100.0%
No. 4	99.9%	95-100%
No. 8	98.8%	80-100%
No. 16	79.5%	50-85%
No. 30	34.9%	25-60%
No. 50	5.6%	5-30%
No. 100	0.9%	0-10%
No. 200	0.3%	0-3%

APPENDIX B: SUPPLEMENTAL INFORMATION FOR CHAPTER 4

Table B.1: Compiled results of slump values for each batch

Designation	Individual Batch Slump (in)				Average Slump (in)
	1	2	3	4	
P.A.N.M	1.5	1.25	1.25	1.75	1.4
P.B.N.M	2	2	1.75	2	1.9
P.BL.N.M	2	2	2.25	2.5	2.2
C.A.N.M	-	0.75	1	1.5	1.1
C.B.N.M	-	1	1.5	1.75	1.4
C.BL.N.M	-	1	1	1.25	1.1
M.A.N.M	2.75	-	1.75	1.5	2.0
M.B.N.M	3.25	-	2.25	1.75	2.4
M.BL.N.M	2.25	-	2.5	2	2.3
P.A.A.M	2.5	3.5	2.25	2.5	2.7
P.B.A.M	2.5	2.5	2.25	1.75	2.3
P.BL.A.M	2.5	3.25	2	2.25	2.5
P.A.B.M	-	3	2.25	2	2.4
P.B.B.M	-	2.75	2	2	2.3
P.BL.B.M	-	2.755	2.25	2	2.3
P.A.N.N	-	1.5	2	2.25	1.9
P.B.N.N	-	2.5	3.75	3.75	3.3
P.BL.N.N	-	2.75	3	2.75	2.8

Table B.2: Compiled results of air content values for each batch

Designation	Individual Batch Air Content (%)				Average Air Content (%)
	1	2	3	4	
P.A.N.M	5.6	5.5	5.1	5.5	5.4
P.B.N.M	5.9	6.0	6.0	6.0	6.0
P.BL.N.M	5.0	5.8	5.6	6.0	5.6
C.A.N.M	-	5.7	5.8	6.0	5.8
C.B.N.M	-	5.4	5.7	5.8	5.6
C.BL.N.M	-	5.6	5.0	6.0	5.5
M.A.N.M	5.4	-	5.4	5.2	5.3
M.B.N.M	5.7	-	5.2	5.2	5.4
M.BL.N.M	5.0	-	5.4	5.0	5.1
P.A.A.M	5.5	5.9	5.6	5.6	5.7
P.B.A.M	5.1	5.3	5.3	5.0	5.2
P.BL.A.M	5.1	5.3	5.1	5.3	5.2
P.A.B.M	-	5.4	5.6	5.8	5.6
P.B.B.M	-	6.0	5.6	5.6	5.7
P.BL.B.M	-	5.9	5.6	5.3	5.6
P.A.N.N	-	5.0	5.3	5.5	5.3
P.B.N.N	-	5.1	5.6	5.6	5.4
P.BL.N.N	-	5.9	5.3	5.4	5.5

Table B.3: Compiled results of unit weight values for each batch

Designation	Individual Batch Unit Weight (pcf)				Average Unit Weight (pcf)
	1	2	3	4	
P.A.N.M	144	144	146	145	145
P.B.N.M	143	143	143	143	143
P.BL.N.M	146	143	144	142	144
C.A.N.M	-	138	138	137	138
C.B.N.M	-	138	139	138	139
C.BL.N.M	-	137	139	139	139
M.A.N.M	145	-	145	146	145
M.B.N.M	143	-	145	145	144
M.BL.N.M	146	-	144	146	145
P.A.A.M	141	139	142	142	141
P.B.A.M	142	142	142	143	142
P.BL.A.M	143	141	142	142	142
P.A.B.M	-	141	142	142	142
P.B.B.M	-	139	141	142	141
P.BL.B.M	-	140	141	142	141
P.A.N.N	-	144	142	142	143
P.B.N.N	-	143	142	142	142
P.BL.N.N	-	147	142	141	143

Table B.4: Compiled 28-day compressive strength results

Designation	28-day Compressive Strength (psi)			Average Compressive Strength (psi)	Standard Deviation
	1	2	3		
P.A.N.M	5,130	5,207	5,338	5,220	105
P.B.N.M	4,899	4,783	4,856	4,850	59
P.BL.N.M	4,781	5,011	5,264	5,020	242
C.A.N.M	5,432	5,405	5,233	5,360	108
C.B.N.M	5,743	6,272	5,856	5,960	279
C.BL.N.M	5,405	5,295	5,969	5,560	362
M.A.N.M	5,060	5,151	4,882	5,030	137
M.B.N.M	4,941	5,271	5,077	5,100	166
M.BL.N.M	4,727	5,008	4,636	4,790	194
P.A.A.M	4,445	4,026	4,352	4,270	220
P.B.A.M	4,295	4,115	3,745	4,050	280
P.BL.A.M	3,693	3,915	3,635	3,750	148
P.A.B.M	3,911	3,732	3,702	3,780	113
P.B.B.M	3,138	3,222	3,045	3,140	89
P.BL.B.M	3,616	3,211	4,501	3,780	660
P.A.N.N	5,245	5,584	5,378	5,400	171
P.B.N.N	4,220	4,458	4,484	4,390	145
P.BL.N.N	5,196	5,352	5,024	5,190	164

Table B.5: Compiled 28-day modulus of rupture results

Designation	28-day Modulus of Rupture (psi)		Average Modulus of Rupture (psi)	Standard Deviation
	1	2		
P.A.N.M	674	685	680	8
P.B.N.M	721	620	670	71
P.BL.N.M	635	676	660	29
C.A.N.M	738	721	730	12
C.B.N.M	704	795	750	64
C.BL.N.M	686	665	680	15
M.A.N.M	583	565	570	13
M.B.N.M	632	650	640	13
M.BL.N.M	598	614	610	11
P.A.A.M	610	680	650	49
P.B.A.M	458	613	540	110
P.BL.A.M	675	621	650	38
P.A.B.M	562	573	570	8
P.B.B.M	609	622	620	9
P.BL.B.M	579	537	560	30
P.A.N.N	717	754	740	26
P.B.N.N	738	695	720	30
P.BL.N.N	728	777	750	35

Table B.6: Compiled 28-day modulus of elasticity results

Designation	28-day Modulus of Elasticity (psi)		Average Modulus of Elasticity (psi)	Standard Deviation
	1	2		
P.A.N.M	2,713,049	3,123,108	2,920,000	289,955
P.B.N.M	3,184,042	3,490,374	3,340,000	216,609
P.BL.N.M	2,659,514	2,203,131	2,430,000	322,712
C.A.N.M	4,085,851	3,382,608	3,730,000	497,268
C.B.N.M	3,620,150	3,366,678	3,490,000	179,232
C.BL.N.M	3,805,354	3,578,321	3,690,000	160,537
M.A.N.M	2,484,757	2,604,384	2,540,000	84,589
M.B.N.M	2,710,181	2,808,936	2,760,000	69,830
M.BL.N.M	2,923,484	3,122,951	3,020,000	141,044
P.A.A.M	3,257,485	3,190,631	3,220,000	47,273
P.B.A.M	2,205,106	3,200,277	2,700,000	703,692
P.BL.A.M	2,486,174	2,895,681	2,690,000	289,565
P.A.B.M	2,776,134	2,896,999	2,840,000	85,464
P.B.B.M	2,436,815	2,574,383	2,510,000	97,275
P.BL.B.M	2,671,917	2,773,204	2,720,000	71,621
P.A.N.N	3,620,851	3,176,120	3,400,000	314,472
P.B.N.N	2,919,808	4,109,804	3,510,000	841,454
P.BL.N.N	2,925,107	3,150,812	3,040,000	159,598

Table B.7: Compiled Poisson's ratio results

Designation	28-day Poisson's Ratio		Average Poisson's Ratio	Standard Deviation
	1	2		
P.A.N.M	0.19	0.20	0.20	0.01
P.B.N.M	0.18	0.21	0.20	0.02
P.BL.N.M	0.18	0.18	0.18	0.00
C.A.N.M	0.22	0.23	0.22	0.00
C.B.N.M	0.21	0.20	0.21	0.01
C.BL.N.M	0.22	0.23	0.22	0.00
M.A.N.M	0.16	0.19	0.18	0.02
M.B.N.M	0.19	0.20	0.20	0.01
M.BL.N.M	0.19	0.20	0.20	0.01
P.A.A.M	0.24	0.23	0.23	0.01
P.B.A.M	0.20	0.21	0.21	0.01
P.BL.A.M	0.16	0.17	0.16	0.00
P.A.B.M	0.24	0.19	0.22	0.04
P.B.B.M	0.16	0.21	0.18	0.03
P.BL.B.M	0.19	0.20	0.19	0.00
P.A.N.N	0.17	0.13	0.15	0.03
P.B.N.N	0.18	0.20	0.19	0.01
P.BL.N.N	0.16	0.15	0.15	0.00

Table B.8: Compiled CTE results

Designation	28-day CTE (x 10^{-6} in/in°F)			Average CTE (x 10^{-6} in/in°F)	Standard Deviation
	1	2	3		
P.A.N.M	4.51	4.52	4.68	4.57	0.10
P.B.N.M	4.56	4.56	4.78	4.63	0.13
P.BL.N.M	4.47	4.53	4.62	4.54	0.08
C.A.N.M	4.25	4.09	4.36	4.23	0.14
C.B.N.M	4.24	4.11	4.48	4.28	0.19
C.BL.N.M	4.16	4.21	4.53	4.30	0.20
M.A.N.M	4.36	4.43	4.59	4.46	0.12
M.B.N.M	4.49	4.49	4.72	4.57	0.13
M.BL.N.M	4.48	4.43	4.77	4.56	0.18
P.A.A.M	4.39	4.44	4.43	4.42	0.02
P.B.A.M	4.47	4.46	4.45	4.46	0.01
P.BL.A.M	4.55	4.58	4.56	4.57	0.01
P.A.B.M	4.42	4.45	4.43	4.43	0.02
P.B.B.M	4.45	4.59	4.51	4.52	0.07
P.BL.B.M	4.63	4.54	4.51	4.56	0.06
P.A.N.N	5.42	5.40	5.38	5.40	0.02
P.B.N.N	5.31	5.29	5.33	5.31	0.02
P.BL.N.N	5.22	5.54	5.20	5.32	0.19

Table B.9: Compiled thermal conductivity results

Designation	Thermal Conductivity (Btu/(ft)(hr)(°F))			Average Thermal Conductivity (Btu/(ft)(hr)(°F))	Standard Deviation
	1	2	3		
P.A.N.M	0.97	0.90	0.90	0.92	0.04
P.B.N.M	0.88	0.97	0.98	0.95	0.06
P.BL.N.M	0.78	0.94	0.69	0.80	0.13
C.A.N.M	0.84	0.71	0.89	0.81	0.09
C.B.N.M	0.81	0.95	0.93	0.89	0.07
C.BL.N.M	0.86	0.88	0.86	0.87	0.02
M.A.N.M	0.85	0.88	0.88	0.87	0.02
M.B.N.M	0.93	0.93	0.99	0.95	0.03
M.BL.N.M	0.89	0.87	0.96	0.91	0.04
P.A.A.M	0.91	0.90	0.88	0.90	0.02
P.B.A.M	0.84	1.02	0.84	0.90	0.10
P.BL.A.M	0.89	0.88	0.88	0.88	0.01
P.A.B.M	0.94	0.90	0.83	0.89	0.06
P.B.B.M	0.90	0.93	0.88	0.90	0.02
P.BL.B.M	0.85	0.88	0.98	0.90	0.06
P.A.N.N	1.25	1.21	1.28	1.25	0.03
P.B.N.N	1.14	1.22	0.99	1.12	0.11
P.BL.N.N	1.14	1.15	1.25	1.18	0.06

Table B.10: Compiled heat capacity results

Designation	Heat Capacity (Btu/(lb-°F))			Average Heat Capacity (Btu/(lb-°F))	Standard Deviation
	1	2	3		
P.A.N.M	0.20	0.20	0.20	0.20	0.00
P.B.N.M	0.21	0.20	0.20	0.20	0.00
P.BL.N.M	0.21	0.21	0.19	0.20	0.02
C.A.N.M	0.22	0.21	0.22	0.22	0.01
C.B.N.M	0.21	0.22	0.22	0.22	0.01
C.BL.N.M	0.20	0.20	0.20	0.20	0.00
M.A.N.M	0.20	0.20	0.20	0.20	0.00
M.B.N.M	0.21	0.21	0.21	0.21	0.00
M.BL.N.M	0.20	0.20	0.20	0.20	0.00
P.A.A.M	0.20	0.20	0.20	0.20	0.00
P.B.A.M	0.20	0.20	0.21	0.20	0.00
P.BL.A.M	0.20	0.19	0.20	0.20	0.00
P.A.B.M	0.20	0.20	0.20	0.20	0.00
P.B.B.M	0.20	0.20	0.20	0.20	0.00
P.BL.B.M	0.21	0.20	0.20	0.20	0.01
P.A.N.N	0.20	0.20	0.20	0.20	0.00
P.B.N.N	0.20	0.20	0.20	0.20	0.00
P.BL.N.N	0.20	0.19	0.19	0.20	0.00

Table B.11: Complete table of laboratory testing results

Designation	Fresh Properties					Compressive Strength (psi)				MOE (psi)	Poisson's Ratio	MOR (psi)	CTE (in/in °F)	Heat Capacity (Btu/lb-°F)	Thermal Conductivity (Btu/(ft)(hr)(°F))
	Slump (in)	Air Content (%)	SAM (%)	SAM Number	Unit Weight (PCF)	3 Day	7 Day	28 Day	90 Day						
P.A.N.M	1.4	5.4	6.2	0.19	145	3,370	4,020	5,020	5,230	2,920,000	0.20	680	4.57 × 10-6	0.20	0.92
P.B.N.M	1.9	6.0	6.9	0.23	143	3,660	3,960	4,850	5,500	3,340,000	0.20	670	4.63 × 10-6	0.20	0.95
P.BL.N.M	2.2	5.6	6.6	0.28	144	3,720	4,340	5,020	6,170	2,430,000	0.18	660	4.54 × 10-6	0.20	0.80
C.A.N.M	1.1	5.8	7.3	0.80	138	3,650	4,890	5,360	6,010	3,730,000	0.22	730	4.23 × 10-6	0.22	0.81
C.B.N.M	1.4	5.6	7.0	0.35	139	4,340	4,770	5,960	5,690	3,490,000	0.21	750	4.28 × 10-6	0.22	0.89
C.BL.N.M	1.1	5.5	7.4	0.19	139	4,290	4,850	5,560	5,610	3,690,000	0.22	680	4.30 × 10-6	0.20	0.87
M.A.N.M	2.0	5.3	-	-	145	3,060	3,930	5,030	5,530	2,540,000	0.18	570	4.46 × 10-6	0.20	0.87
M.B.N.M	2.4	5.4	-	-	144	3,800	4,130	5,100	5,390	2,760,000	0.20	640	4.57 × 10-6	0.21	0.95
M.BL.N.M	2.3	5.1	-	-	145	3,670	4,130	4,790	5,530	3,020,000	0.20	610	4.56 × 10-6	0.20	0.91
P.A.A.M	2.7	5.7	7.2	0.88	141	2,620	3,550	4,270	5,560	3,220,000	0.23	650	4.42 × 10-6	0.20	0.90
P.B.A.M	2.3	5.2	6.1	0.42	142	2,460	3,050	4,050	4,380	2,700,000	0.21	540	4.46 × 10-6	0.20	0.90
P.BL.A.M	2.5	5.2	6.4	0.29	142	2,210	2,960	3,750	4,620	2,690,000	0.16	650	4.57 × 10-6	0.20	0.88
P.A.B.M	2.4	5.6	6.4	0.29	142	2,130	2,390	3,780	5,490	2,840,000	0.22	570	4.43 × 10-6	0.20	0.89
P.B.B.M	2.3	5.7	7.2	0.22	141	2,040	2,410	3,140	4,340	2,510,000	0.18	620	4.52 × 10-6	0.20	0.90
P.BL.B.M	2.3	5.6	7.3	0.19	141	2,330	2,500	3,780	4,370	2,720,000	0.19	560	4.56 × 10-6	0.20	0.90
P.A.N.N	1.9	5.3	5.6	0.10	143	2,720	4,080	5,400	6,060	3,400,000	0.15	740	5.40 × 10-6	0.20	1.25
P.B.N.N	3.3	5.4	5.3	0.27	142	3,010	3,420	4,390	5,450	3,510,000	0.19	720	5.31 × 10-6	0.20	1.12
P.BL.N.N	2.8	5.5	7.2	0.19	143	3,270	3,930	5,190	5,800	3,040,000	0.15	750	5.32 × 10-6	0.20	1.18

APPENDIX C: SUPPLEMENTAL INFORMATION FOR CHAPTER 5

PCC	
Thickness (in.)	**10.5**
Unit weight (pcf)	**150**
Poisson's ratio	**0.17**
Thermal	
PCC coefficient of thermal expansion (in./in./deg F x 10^-6)	**6**
PCC thermal conductivity (BTU/hr-ft-deg F)	**1.25**
PCC heat capacity (BTU/lb-deg F)	**0.28**
Mix	
Cement type	**Type I (1)**
Cementitious material content (lb/yd^3)	**600**
Water to cement ratio	**0.42**
Aggregate type	**Dolomite (2)**
PCC zero-stress temperature (deg F)	**Calculated**
Ultimate shrinkage (microstrain)	**642.8 (calculated)**
Reversible shrinkage (%)	**50**
Time to develop 50% of ultimate shrinkage (days)	**35**
Curing method	**Curing Compound**
Strength	
PCC strength and modulus	**Level:3 Rupture(650) Modulus(4200000)**
Identifiers	
Display name/identifier	**JPCP Default**

Figure C.1: Example of PCC input values in the base MEPDG file

PCC	
Thickness (in.)	**10.5**
Unit weight (pcf)	**145**
Poisson's ratio	**0.2**
Thermal	
PCC coefficient of thermal expansion (in./in./deg F x 10^-6)	**4.46**
PCC thermal conductivity (BTU/hr-ft-deg F)	**0.95**
PCC heat capacity (BTU/lb-deg F)	**0.2**
Mix	
Cement type	**Type I (1)**
Cementitious material content (lb/yd^3)	**550**
Water to cement ratio	**0.48**
Aggregate type	**Dolomite (2)**
PCC zero-stress temperature (deg F)	**Calculated**
Ultimate shrinkage (microstrain)	**678.3 (calculated)**
Reversible shrinkage (%)	**50**
Time to develop 50% of ultimate shrinkage (days)	**35**
Curing method	**Curing Compound**
Strength	
PCC strength and modulus	**Level:3 Rupture(641) Modulus(2760000)**
Identifiers	
Display name/identifier	**JPCP Default**

Figure C.2: Example of new PCC input values MEPDG file

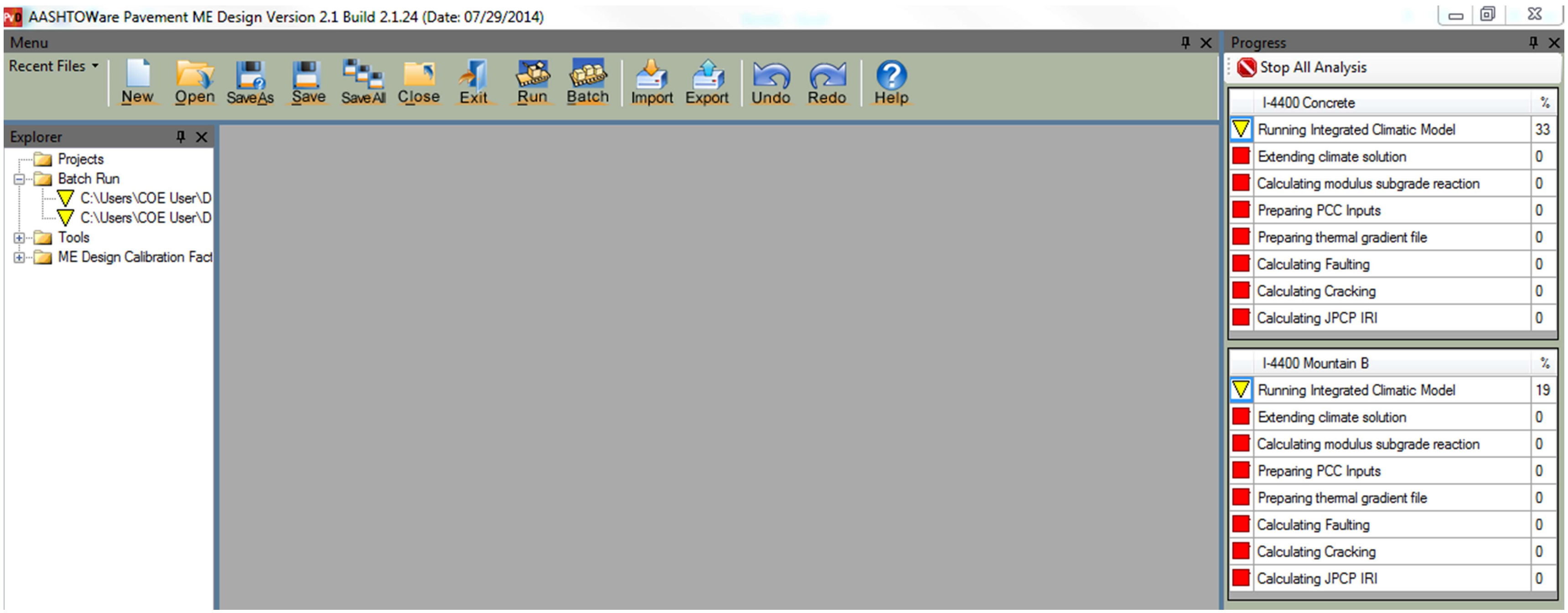
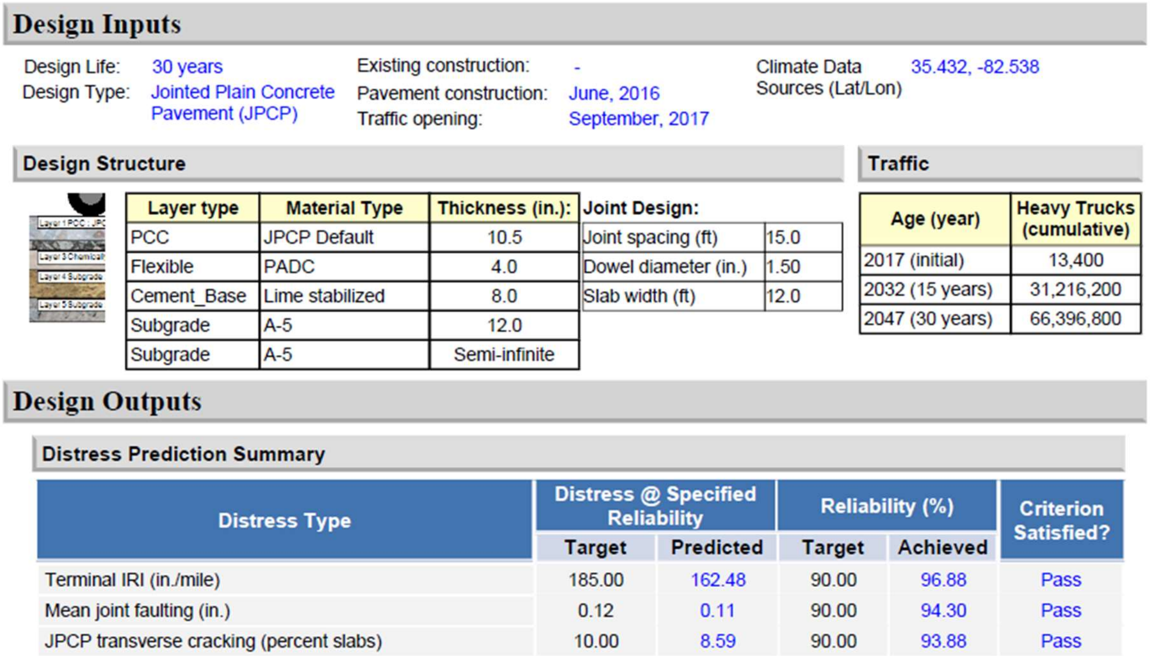


Figure C.3: Performing a batch



Design Inputs

Design Life: 30 years
Design Type: Jointed Plain Concrete Pavement (JPCP)
Existing construction: -
Pavement construction: June, 2016
Traffic opening: September, 2017
Climate Data Sources (Lat/Lon): 35.432, -82.538

Design Structure

Layer type	Material Type	Thickness (in.):
PCC	JPCP Default	10.5
Flexible	PADC	4.0
Cement_Base	Lime stabilized	8.0
Subgrade	A-5	12.0
Subgrade	A-5	Semi-infinite

Joint Design:	
Joint spacing (ft)	15.0
Dowel diameter (in.)	1.50
Slab width (ft)	12.0

Traffic

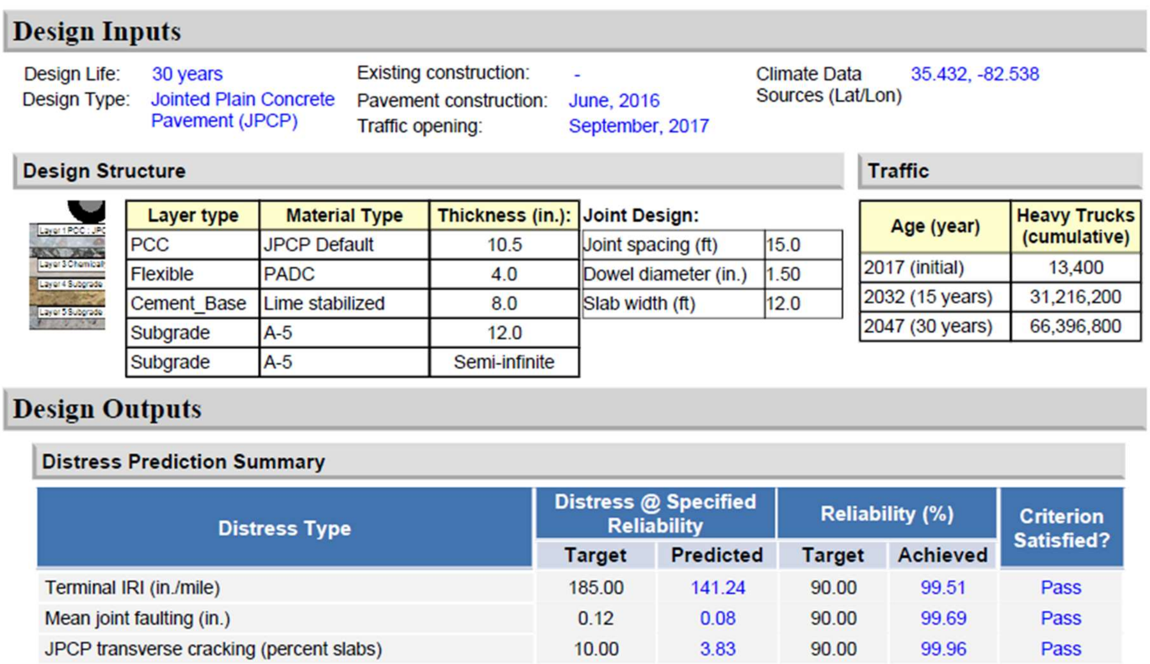
Age (year)	Heavy Trucks (cumulative)
2017 (initial)	13,400
2032 (15 years)	31,216,200
2047 (30 years)	66,396,800

Design Outputs

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mile)	185.00	162.48	90.00	96.88	Pass
Mean joint faulting (in.)	0.12	0.11	90.00	94.30	Pass
JPCP transverse cracking (percent slabs)	10.00	8.59	90.00	93.88	Pass

Figure C.4: Example of base MEPDG file results



Design Inputs

Design Life: 30 years
Design Type: Jointed Plain Concrete Pavement (JPCP)
Existing construction: -
Pavement construction: June, 2016
Traffic opening: September, 2017
Climate Data Sources (Lat/Lon): 35.432, -82.538

Design Structure

Layer type	Material Type	Thickness (in.):
PCC	JPCP Default	10.5
Flexible	PADC	4.0
Cement_Base	Lime stabilized	8.0
Subgrade	A-5	12.0
Subgrade	A-5	Semi-infinite

Joint Design:	
Joint spacing (ft)	15.0
Dowel diameter (in.)	1.50
Slab width (ft)	12.0

Traffic

Age (year)	Heavy Trucks (cumulative)
2017 (initial)	13,400
2032 (15 years)	31,216,200
2047 (30 years)	66,396,800

Design Outputs

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mile)	185.00	141.24	90.00	99.51	Pass
Mean joint faulting (in.)	0.12	0.08	90.00	99.69	Pass
JPCP transverse cracking (percent slabs)	10.00	3.83	90.00	99.96	Pass

Figure C.5: Example of new PCC inputs MEPDG file results

APPENDIX D: SUPPLEMENTAL INFORMATION FOR CHAPTER 6

Design Inputs

Design Life:	30 years	Existing construction:	-	Climate Data Sources (Lat/Lon)	36.098, -79.944
Design Type:	Jointed Plain Concrete Pavement (JPCP)	Pavement construction:	June, 2018		
		Traffic opening:	September, 2018		

Design Structure

Layer type	Material Type	Thickness (in.):
PCC	JPCP Default	10.0
Cement_Base	Lime stabilized	8.0
NonStabilized	Crushed gravel	10.0
Subgrade	A-6	Semi-infinite

Joint Design:	
Joint spacing (ft)	15.0
Dowel diameter (in.)	1.25
Slab width (ft)	12.0

Traffic

Age (year)	Heavy Trucks (cumulative)
2018 (initial)	6,000
2033 (15 years)	17,899,100
2048 (30 years)	42,454,800

Design Outputs

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mile)	185.00	169.49	90.00	95.20	Pass
Mean joint faulting (in.)	0.12	0.13	90.00	83.53	Fail
JPCP transverse cracking (percent slabs)	10.00	4.65	90.00	99.72	Pass

Distress Charts

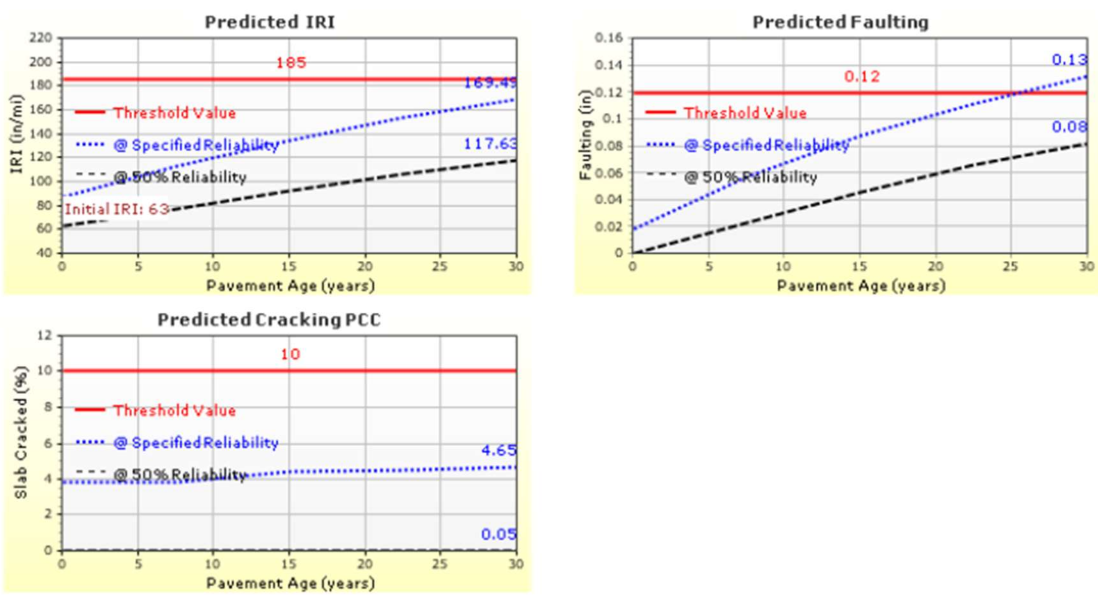


Figure D.1: Example result of unit weight of 138 pcf sensitivity analysis

Design Inputs

Design Life:	30 years	Existing construction:	-	Climate Data Sources (Lat/Lon)	36.098, -79.944
Design Type:	Jointed Plain Concrete Pavement (JPCP)	Pavement construction:	June, 2018		
		Traffic opening:	September, 2018		

Design Structure

Layer type	Material Type	Thickness (in.):
PCC	JPCP Default	10.0
Cement_Base	Lime stabilized	8.0
NonStabilized	Crushed gravel	10.0
Subgrade	A-6	Semi-infinite

Joint Design:	
Joint spacing (ft)	15.0
Dowel diameter (in.)	1.25
Slab width (ft)	12.0

Traffic

Age (year)	Heavy Trucks (cumulative)
2018 (initial)	6,000
2033 (15 years)	17,899,100
2048 (30 years)	42,454,800

Design Outputs

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mile)	185.00	165.29	90.00	96.24	Pass
Mean joint faulting (in.)	0.12	0.12	90.00	87.54	Fail
JPCP transverse cracking (percent slabs)	10.00	4.65	90.00	99.72	Pass

Distress Charts

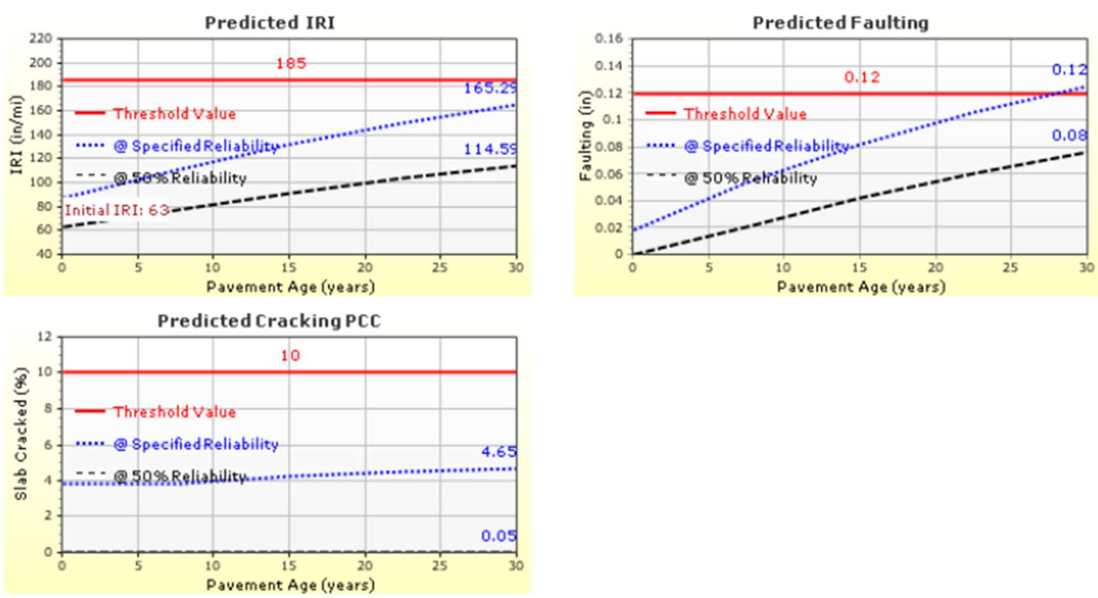


Figure D.2: Example result of unit weight of 141 pcf sensitivity analysis

Design Inputs

Design Life: 30 years
Design Type: Jointed Plain Concrete Pavement (JPCP)

Existing construction: -
Pavement construction: June, 2018
Traffic opening: September, 2018

Climate Data Sources (Lat/Lon): 36.098, -79.944

Design Structure

Layer type	Material Type	Thickness (in.):
PCC	JPCP Default	10.0
Cement_Base	Lime stabilized	8.0
NonStabilized	Crushed gravel	10.0
Subgrade	A-6	Semi-infinite

Joint Design:	
Joint spacing (ft)	15.0
Dowel diameter (in.)	1.25
Slab width (ft)	12.0

Traffic

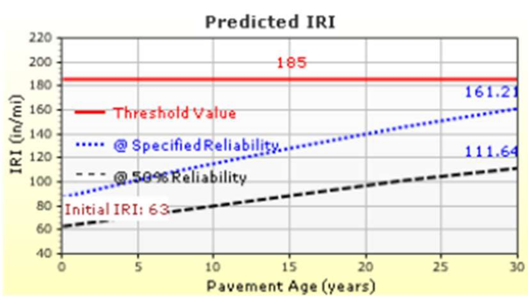
Age (year)	Heavy Trucks (cumulative)
2018 (initial)	6,000
2033 (15 years)	17,899,100
2048 (30 years)	42,454,800

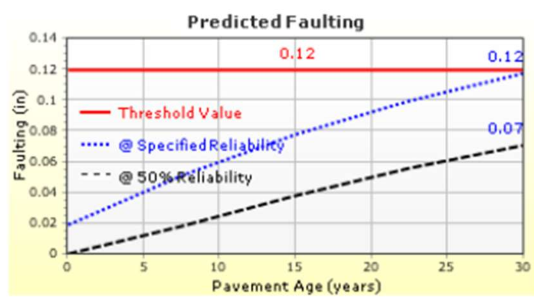
Design Outputs

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mile)	185.00	161.21	90.00	97.11	Pass
Mean joint faulting (in.)	0.12	0.12	90.00	90.86	Pass
JPCP transverse cracking (percent slabs)	10.00	4.65	90.00	99.72	Pass

Distress Charts





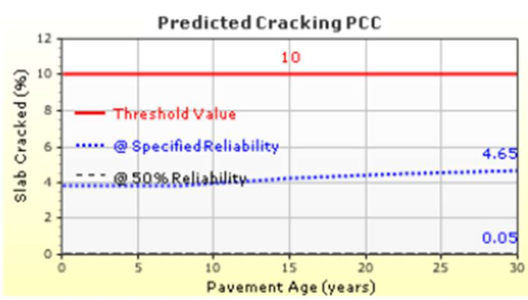


Figure D.3: Example result of unit weight of 144 pcf sensitivity analysis

Design Inputs

Design Life: 30 years
Design Type: Jointed Plain Concrete Pavement (JPCP)

Existing construction: -
Pavement construction: June, 2018
Traffic opening: September, 2018

Climate Data Sources (Lat/Lon): 36.098, -79.944

Design Structure

Layer type	Material Type	Thickness (in.):
PCC	JPCP Default	10.0
Cement_Base	Lime stabilized	8.0
NonStabilized	Crushed gravel	10.0
Subgrade	A-6	Semi-infinite

Joint Design:	
Joint spacing (ft)	15.0
Dowel diameter (in.)	1.25
Slab width (ft)	12.0

Traffic

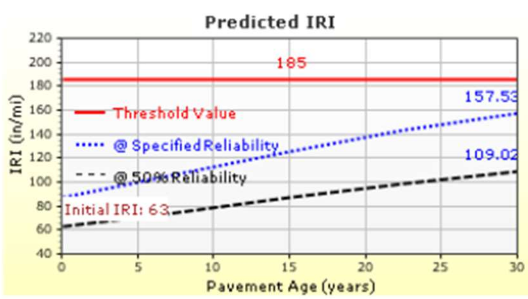
Age (year)	Heavy Trucks (cumulative)
2018 (initial)	6,000
2033 (15 years)	17,899,100
2048 (30 years)	42,454,800

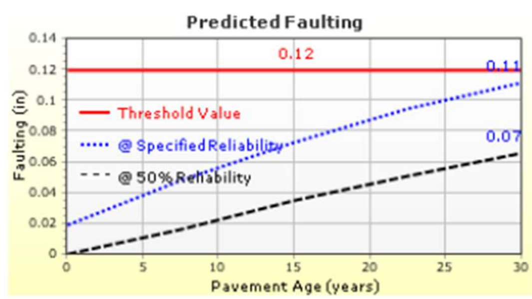
Design Outputs

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mile)	185.00	157.53	90.00	97.76	Pass
Mean joint faulting (in.)	0.12	0.11	90.00	93.35	Pass
JPCP transverse cracking (percent slabs)	10.00	4.65	90.00	99.72	Pass

Distress Charts





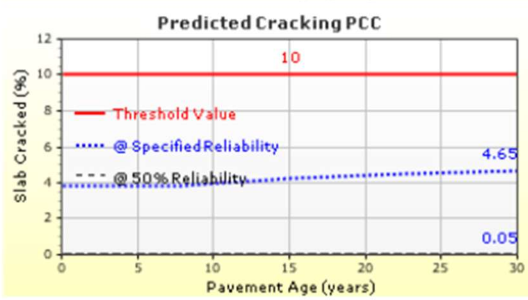


Figure D.4: Example result of unit weight of 147 pcf sensitivity analysis

Design Inputs

Design Life:	30 years	Existing construction:	-	Climate Data Sources (Lat/Lon)	36.098, -79.944
Design Type:	Jointed Plain Concrete Pavement (JPCP)	Pavement construction:	June, 2018		
		Traffic opening:	September, 2018		

Design Structure

Layer type	Material Type	Thickness (in.):
PCC	JPCP Default	10.0
Cement_Base	Lime stabilized	8.0
NonStabilized	Crushed gravel	10.0
Subgrade	A-6	Semi-infinite

Joint Design:	
Joint spacing (ft)	15.0
Dowel diameter (in.)	1.25
Slab width (ft)	12.0

Traffic

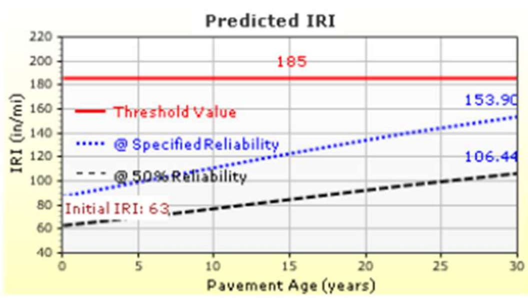
Age (year)	Heavy Trucks (cumulative)
2018 (initial)	6,000
2033 (15 years)	17,899,100
2048 (30 years)	42,454,800

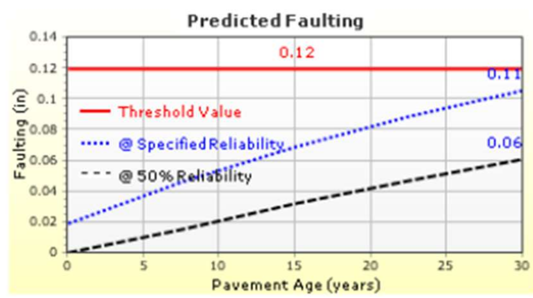
Design Outputs

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mile)	185.00	153.90	90.00	98.30	Pass
Mean joint faulting (in.)	0.12	0.11	90.00	95.34	Pass
JPCP transverse cracking (percent slabs)	10.00	4.65	90.00	99.72	Pass

Distress Charts





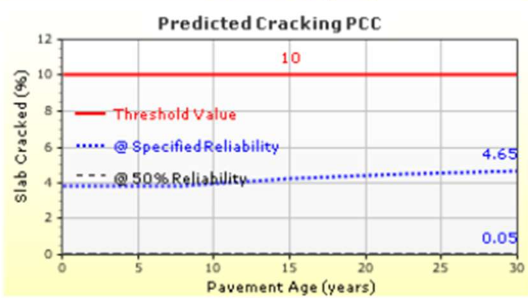


Figure D.5: Example result of unit weight of 150 pcf sensitivity analysis