RESEARCH & DEVELOPMENT

Improved Data for Mechanistic-Empirical Pavement Design for Concrete Pavements

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16. Abstract

The Mechanistic-Empirical Pavement Design Guide (M-EPDG) is a state-of-the-practice tool for pavement analysis and design. M-EPDG has been incorporated into the AASHTOware Pavement ME Design software program, and local calibration is necessary for optimal performance. To support NCDOT in use of the Pavement ME Design software for design and analysis of portland cement concrete (PCC) pavements, as well as to meet the need to support the decision to move forward with PLC concrete specifications for future NCDOT projects, a variety of PCC pavement mixtures were developed, batched, and tested. Mixtures included several coarse aggregates, Type I/II ordinary portland cement (OPC), PLC, fly ash from two sources, and fine aggregates (manufactured sand and a natural sand) used in North Carolina (NC). PLC used in these mixtures were produced by intergrinding with one of the OPCs used in the study. Tests to determine the mechanical and thermal properties of the concrete mixtures, as well as several durability performance tests were performed. A catalog of PCC characteristics for use as inputs in the Pavement ME Design software was prepared, and the impact of the new suggested inputs on NC concrete pavement design were evaluated.

The cement type (OPC or PLC) used does not highly influence the results for the suite of tests used to determine the concrete inputs for M-EPDG. Comparable performance of the PLC provides incentive to NCDOT for use of this more sustainable alternative to OPC. Although the type of coarse aggregate utilized in this study did not highly influence the laboratory test results supporting the recommended M-EPDG PCC inputs, the fine aggregate type utilized in the mixture (manufactured sand versus natural sand) did have a significant influence on two thermal PCC inputs: coefficient of thermal expansion (CTE) and thermal conductivity. A sensitivity analysis was performed to identify the changes in predicted distresses for a range of each PCC input. Several typical North Carolina concrete pavements were analyzed using previous and newly suggested PCC inputs using the original design constraints. Findings offer insight into the potentially longer service life of concrete pavements designed and constructed in the past by NCDOT. Use of the new PCC input values may 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. The benefits of deciding to reduce PCC thickness should be weighed against the risks associated with under-prediction of traffic or section loss associated with one or more diamond grinding treatments during the service life of the pavement, as well as the service life benefits that could be obtained by using a thicker PCC pavement.

As expected, results indicate use of fly ash in pavement concrete should improve durability performance. Use of PLC alone (without fly ash) did not provide distinct durability performance advantages, when compared to OPC. However, if PLC is utilized with fly ash in concrete mixtures, enhanced durability performance could be anticipated. Due to the delayed strength gain of fly ash mixtures, use of 28-day compressive strength as a PCC input in MEPDG may be unsuitable. A strong correlation was found between surface resistivity test results and rapid chloride permeability test (RCPT) results for all mixtures included this study. Findings of a limited LCA offered insight into the decrease in predicted total criteria air pollutant emissions associated with increased use of fly ash and PLC, providing confidence to NCDOT that use of PLC and fly ash in concrete infrastructure should provide environmental and sustainability benefits, as mandated by the MAP-21 legislation.

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EXECUTIVE SUMMARY

Mechanical and statistical models incorporated into the Mechanistic-Empirical Pavement Design Guide (M-EPDG) have resulted in a state-of-the-practice tool for analysis and design of pavements, and M-EPDG has been incorporated into the commercially available AASHTOware Pavement ME Design software program. Local calibration is necessary for optimal performance of AASHTOWare Pavement ME for the design and performance processes. 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 M-EPDG in pavement analysis and design. Additionally, the North Carolina Department of Transportation (NCDOT) has recently modified their standard specifications to allow portland limestone cement (PLC), but does not have performance data on concrete mixtures utilizing 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. 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.

To support NCDOT and other stakeholders using the Pavement ME Design software, as well as to support the decision to move forward with PLC concrete specifications for future NCDOT projects, a variety of concrete pavement mixtures were developed, batched, and tested in the laboratory. Mixtures included coarse aggregates obtained from the Mountain, Piedmont, and Coastal regions of North Carolina, Type I/II ordinary portland cement (OPC), PLC, supplementary cementitious materials (SCMs), and fine aggregates (manufactured sand and a natural sand) typically used in North Carolina. PLC used in these mixtures were produced by a southeastern manufacturer, produced by intergrinding with one of the OPCs used in the study. Laboratory tests to determine the mechanical properties and thermal characteristics of the concrete mixtures, as well as several durability performance tests, were performed and the results were evaluated. A catalog of concrete characteristics for use by NCDOT as inputs in the Pavement ME Design software was prepared, and the impact on North Carolina concrete pavement design were evaluated.

Laboratory testing indicated that the cement type (OPC or PLC) used does not highly influence the results for the suite of tests used to determine the concrete inputs for M-EPDG. Comparable performance of the PLC provides incentive to NCDOT for use of this sustainable alternative to OPC. Although the type of coarse aggregate utilized in this study did not highly influence the laboratory test results supporting the recommended concrete inputs for M-EPDG, the fine aggregate type utilized in the concrete mixture (manufactured sand versus natural sand) did have a significant influence on the thermal properties of two concrete inputs in M-EPDG, coefficient of thermal expansion (CTE) and thermal conductivity. Several PCC inputs recommended based on this study differ from the suggested (or default) inputs. A sensitivity analysis indicated that CTE was determined to be "Very Sensitive" for North Carolina concrete pavements for all modes of predicted distress, consistent with the findings of other researchers. 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.

Several typical North Carolina concrete pavements were analyzed using previous and newly suggested PCC inputs using the original design constraints. 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. Findings offer insight into the potentially longer service life of concrete pavements designed and constructed in the past by NCDOT. Use of the new PCC input values may 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. The benefits of deciding to reduce the concrete thickness should be weighed against the risks associated with under-prediction of traffic or section loss associated with one or more diamond grinding treatments during the service life of the pavement, as well as the service life benefits potentially linked to a thicker pavement.

As expected, results indicate use of fly ash in pavement concrete should improve durability performance. Use of PLC alone (without fly ash) did not provide distinct durability performance advantages, when compared to OPC. However, if PLC is utilized with fly ash in concrete mixtures, enhanced durability performance could be anticipated. The lower permeability exhibited by PLC concrete with fly ash is likely due to the particle packing effects in these binder systems, and is consistent with the findings of other researchers. Due to the delayed strength gain of fly ash mixtures, it may be unsuitable to utilize the 28-day compressive strength as a PCC input in MEPDG. Similar to the findings of other researchers, there is a strong correlation between surface resistivity test results and RCPT results for all mixtures included this study.

Findings of a limited LCA offered insight into the decrease in predicted total criteria air pollutant emissions associated with increased use of fly ash and PLC, providing confidence to NCDOT that use of PLC and fly ash in concrete infrastructure should provide environmental and sustainability benefits, as mandated by the MAP-21 legislation.

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LIST OF ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
ASR	alkali-silica reactivity
ASTM	American Society for Testing and Materials
cf	cubic foot
cy	cubic yard
CTE	coefficient of thermal expansion
DOT	Department of Transportation
FHWA	Federal Highway Administration
ft	foot
GWP	global warming potential
kg	kilogram
kΩ	kilo-ohm
ID	identification
LCA	lifecycle assessment
LCCA	lifecycle cost analysis
LCI	life cycle inventory
LTPP	Long Term Pavement Performance
M-EPDG	Mechanistic-Empirical Pavement Design Guide
MOE	modulus of elasticity
MOR	modulus of rupture
NBCC	National Building Code of Canada
NC	North Carolina
NCC	National Concrete Consortium
NCDOT	North Carolina Department of Transportation
NCHRP	National Cooperative Highway Research Program
Ω	ohm
OPC	ordinary portland cement
PCA	Portland Cement Association
PCC	portland cement concrete
pcf	pounds per cubic foot
pcy	pounds per cubic yard
psi	pounds per square inch
PLC	portland limestone cement
RCPT	rapid chloride permeability test
RH	relative humidity
RP	Research Project
SCMs	supplementary cementitious materials
SEE	standard error of estimate
SR	state route
TRB	Transportation Research Board
TSA	thaumasite form of sulfate attack
US	United States
VOC	volatile organic compounds
w/c	water to cement ratio
w/cm	water to cementitious materials ratio

1. INTRODUCTION AND RESEARCH OBJECTIVES

1.1 Introduction

Researched and developed over the past several decades, mechanical and statistical models incorporated into the Mechanistic-Empirical Pavement Design Guide (M-EPDG) have resulted in a state-of-the-practice tool for analysis and design of pavements (AASHTO 2008, 2010, 2015). The M-EPDG has been incorporated into the commercially available AASHTOware Pavement ME Design software program, which is currently utilized by the North Carolina Department of Transportation (NCDOT) for analysis and design of North Carolina pavements. One significant characteristic of the M-EPDG process is that users can include locally calibrated values as inputs into the Pavement ME Design software. Many local (North Carolina) inputs for flexible pavements were determined through testing supporting the Federal Highway Administration's (FHWA) "Local Calibration of the M-EPDG Using Pavement Management Systems (FHWA 2010). The purpose of this project was to develop a catalog of inputs for concrete mixtures batched with local materials to support M-EPDG calibration for rigid pavements.

The M-EPDG approach has been implemented into the Pavement ME Design software, commercially available through AASHTOWare. In Pavement ME Design, inputs for portland cement concrete (PCC) design include mixture characteristics (cement type, cementitious material content, water-to-cement (w/c) ratio, and aggregate type), mechanical and physical properties (unit weight, compressive strength, flexural strength, modulus of elasticity, Poisson's ratio thickness), and curing method. Included as user selected or computed inputs are shrinkage and strain-related values (PCC zero-stress temperature, ultimate shrinkage, reversible shrinkage, and time to develop ultimate shrinkage. Also included are thermal inputs, including coefficient of thermal expansion (CTE), heat capacity, and thermal conductivity. It has been shown that the CTE value of concrete is very influential in the performance of concrete pavements, with higher values of CTE linked to early age cracking, higher curling stresses, and premature joint deterioration. Significant variation in values of concrete CTE exists, and the variation has been highly attributed to the type, origin, and geographic location of aggregates. Coarse aggregate type has been shown to be particularly influential in the CTE of concrete. Although not as influential as CTE on pavement performance, heat capacity and thermal conductivity of concrete are also inputs to Pavement ME, and these values also vary with changes in aggregate type and mixture proportioning.

To support sustainability initiatives that promote a lower carbon footprint infrastructure, PLCs have been developed and are being increasingly utilized worldwide. PLCs are produced in a manner similar to traditional portland cements. However, a portion of the ordinary portland cement (OPC) is replaced with limestone. Environmental benefits of using PLCs include energy savings associated with reduction of required clinker and reduced CO_2 emissions due to reduced calcination of limestone in the clinker manufacture process as well as reduced fossil fuel use in manufacturing (Tennis et al. 2011). In the United States, pilot projects using PLC concrete have been implemented by state highway agencies in Utah and Colorado (Laker and Smartz 2012), as well as in a number of field installations in private construction projects (Tennis 2017). A number of other states, including Oklahoma, Utah, Iowa, Missouri, and Louisiana, currently allow the use of PLCs. Following the lead of these other state agencies that allow use of PLCs, NCDOT has recently made provisions to allow PLCs in concrete construction. As part of this project, PLC concrete made with locally available materials was batched and tested in order to support the decision to allow PLCs in North Carolina highway infrastructure.

1.2 Research Objectives

To support NCDOT and other stakeholders using the Pavement ME Design software for design and analysis of North Carolina concrete pavements, the following research objectives guided this study. These objectives also meet the need to support the decision to move forward with PLC concrete specifications for future NCDOT projects.

1. Develop and batch concrete mixtures meeting NCDOT specifications for concrete pavements that include coarse aggregates obtained from the Mountain, Piedmont, and Coastal regions of North Carolina. Utilize Type I/II cement, PLC, supplementary cementitious materials (SCMs), and fine aggregates typically used in North Carolina. Due to the increased usage of manufactured sand in North Carolina, both a manufactured sand and a natural sand were included in the study. PLC used in these mixtures were produced by a southeastern manufacturer, produced by intergrinding with one of the OPCs used in the study.

2. Perform laboratory testing to determine the mechanical properties and thermal characteristics of the concrete mixtures, as well as several durability performance tests.

3. Prepare a catalog of concrete characteristics for use by NCDOT as inputs in the Pavement ME Design software. The catalog includes inputs for both mechanical and thermal properties for both OPC and PLC concrete mixtures.

2. SUMMARY OF KEY LITERATURE FINDINGS

Literature review performed to support this work was performed in two areas: PCC inputs for M-EPDG and use of PLC in concrete mixtures. A summary of key literature findings is presented in this section. The full literature review supporting this work, along with a complete list of references, is provided in Appendix A of this report.

2.1 Summary Literature Review on PCC Inputs for Mechanistic-Empirical Pavement Design

The M-EPDG is a significant change from previously utilized pavement design methods. Over the past two decades, leading researchers have developed the mechanical and statistical models incorporated into the M-EPDG software, now available as Pavement ME Design. The mechanistic analysis of pavement responses includes traffic and climatic data that are site specific. As part of the empirical analysis, results of the mechanistic analysis are compared to data on field-observed distresses of existing pavement sections in the Long-Term Pavement Performance (LTPP) database. The M-EPDG process is an iterative approach to pavement design, with the performance of trial pavement sections compared to design performance criteria that are selected to "ensure that a pavement design will perform satisfactorily over its design life," as outlined in the American Association of State Highway and Transportation Officials (AASHTO) "Mechanistic-Empirical Pavement Design Guide: A Manual of Practice" (2008). Performance criteria for JPCP include joint faulting, transverse slab cracking, and smoothness. Threshold values for performance criteria are selected by agencies based on several considerations, including pavement characteristics that trigger major rehabilitation efforts, impact safety, and require other maintenance.

In Pavement ME Design, user inputs for trial concrete pavement sections can be site-specific or obtained by testing (Level 1), estimated through correlations to other test results for similar materials (Level 2), or default values provided in the software (Level 3). The most accurate analysis is performed through use of Level 1 inputs where possible. AASHTO recommended Level 1 (test-obtained) inputs for portland cement concrete PCC pavements and some PCC overlays include elastic modulus, Poisson's ratio, flexural strength, unit weight, CTE, thermal conductivity, and heat capacity.

Over the past several years, many state agencies (including NCDOT) have adopted M-EPDG and are currently utilizing Pavement ME Design software. The FHWA and AASHTO have been involved in assisting state agencies obtaining locally calibrated input values, and published the "Guide for the Local Calibration of the Mechanistic-Empirical Pavement Design Guide" in November 2010. As part of FHWA's local calibration effort, North Carolina was selected as the pilot state to be included in their study "Local Calibration of the MEPDG Using Pavement Management Systems." A summary of M-EPDG default values for North Carolina PCC pavement design is published in this study, FHWA Report No. HIF-11-026 (2010). Most PCC design properties, general properties, mixture properties, and strength properties are provided. However, PCC thermal properties for North Carolina concrete mixtures used in pavement applications are not available in this guide.

The CTE of concrete plays a key role in concrete pavement performance, as "the magnitudes of temperature-related deformations are directly proportional to this value during early ages (i.e., within 72 hours of paving), as well as during the pavement design life" (Mallela et al. 2005). These temperature-related deformations affect curling-induced stresses and axial stresses, which contribute to both top-down and bottom-up transverse cracking, as well as joint deterioration. As discussed by Mallela et al. (2005), higher values of concrete's CTE have been linked to:

- Early-age random cracking caused by excessive longitudinal slab movement on a highly-resistant base,
- Higher curling stresses, resulting in increased mid-slab transverse and longitudinal cracking,
- Larger amounts of slab support loss at early ages due to curling,
- Larger joint openings during cooler seasons,
- Greater magnitudes of corner deflections due to curling, and
- Excessive joint opening and closing, resulting in loss of joint sealant and subsequent faulting.

Sensitivity analyses indicate that CTE is a key input to the Pavement ME software (Mallela et al. 2005, Guclu et al. 2009). Thermal characteristics including CTE are influenced by the materials used in the concrete mixture. It has been shown that since aggregates comprise the bulk of concrete by volume, the CTE of concrete is greatly influenced by the type of aggregate used. CTE values vary widely based on aggregate type and origin. Concrete CTE values published in various literature sources range from 3 to 8×10^{-6} in/(in·°F) (5.4 to 14.4 m/(m·°C), as shown in Table 2.1. A summary of sensitivity analyses for predicted JPCP pavement performance to concrete inputs is shown in Table 2.2.

Table 2.1: Summary of values of concrete thermal property inputs for M-EPDG

Thermal Property	Suggested Range of M-EPDG Inputs (AASHTO 2015, ARA 2004)	Recommended Default Value (AASHTO 2015, ARA 2004)
CTE	4.6 x 10^{-6} to 6.6 x 10^{-6} in/(in·°F) (8.3 x 10^{-6} to 11.9 x 10^{-6} m/(m·°C))	By coarse aggregate type (see AASHTO 2015)
Thermal Conductivity	1.0 to 1.5 BTU/(ft·hr·°F) (1.73 to 2.60 W/(m·K))	1.25 BTU/(ft·hr·°F) 2.16 W/(m·K)
Heat Capacity	0.20 to 0.40 BTU/(lb·ºF) (0.837 to 1.674 kJ/(kg·K))	0.28 BTU/(lb·ºF) 1.172 kJ/(kg·K)

Table 2.2: Sensitivity of predicted JPCP pavement performance to concrete inputs

Concrete Input	Pavement Performance Parameter			Study Authors
	Terminal IRI	Mean Joint Faulting	Transverse Cracking	
Unit weight	Sensitive	Sensitive	Sensitive	Hall and Beam (2005)
	Low Sensitive to Insensitive	Low Sensitive to Insensitive	---	Guclu and Ceylan (2005)
	---	Sensitive	Sensitive	Guclu et al. (2009)
	Very Sensitive	Very Sensitive	Very Sensitive	Schwartz et al. (2011)
Modulus of Rupture	Sensitive	---	Sensitive	Hall and Beam (2005)
	Sensitive	---	Very Sensitive	Guclu et al. (2009)
	---	Sensitive	Very Sensitive	Schwartz et al. (2011)
Modulus of Elasticity	---	Sensitive	Very Sensitive	Schwartz et al. (2011)
Poisson's Ratio	---	---	Sensitive	Hall and Beam (2005)
	Sensitive to Very Sensitive	Low Sensitive to Insensitive	Sensitive to Very Sensitive	Guclu and Ceylan (2005)
	Sensitive	Sensitive	Sensitive	Guclu et al. (2009)
	---	Sensitive	---	Schwartz et al. (2011)
CTE	Sensitive	Sensitive	Sensitive	Hall and Beam (2005)
	Extreme Sensitivity	Sensitive to Very Sensitive	Extreme Sensitivity	Guclu and Ceylan (2005)
	Sensitive	Very Sensitive	Very Sensitive	Guclu et al. (2009)
	Very Sensitive	Very Sensitive	Very Sensitive	Schwartz et al. (2011)
Thermal Conductivity	---	---	Sensitive	Hall and Beam (2005)
	Extreme Sensitivity	Sensitive to Very Sensitive	Extreme Sensitivity	Guclu and Ceylan (2005), Guclu et al. (2009)
	---	Sensitive	Very Sensitive	Schwartz et al. (2011)
Heat Capacity	---	---	Low Sensitive to Insensitive	Guclu and Ceylan (2005)

Due to the significant role that CTE plays in prediction of pavement performance, as well as the diversity of aggregates available in the United States, many state agencies have performed studies to evaluate the influence of coarse aggregate on concrete CTE. State agencies performing these studies include Arkansas DOT (Tran et al. 2008), Wisconsin DOT (Naik et al. 2011), Texas DOT (Yeon et al. 2009), Alabama DOT (Sakyi-Bekoe 2009), Louisiana DOTD (Shin and Chung 2011), and Hawaii DOT (Havel et al. 2017). In a study performed for Wisconsin DOT, testing to determine the CTE and selected other mechanical properties of concrete paving mixtures produced using six different types of coarse aggregate (glacial gravel, quartzite, granite, diabase, basalt, and dolomite) was performed. As expected, the study indicated that there was a "noticeable variation in the values of the CTE of concrete with different types of aggregates."

A study on concrete CTE performed by Tran et al. (2008) for Arkansas DOT indicated that in addition to being sensitive to coarse aggregate type, CTE is can be significantly influenced by the proportion of coarse aggregates in the mixture, for some aggregate types. Research to determine the CTE for Louisiana DOTD Shin and Chung (2011) indicated that in addition to aggregate types and coarse aggregate portion, relative humidity also has a "significant influence on CTE," which is similar to the findings of a number of early studies (as presented in Yeon et al. 2009). In research performed for

Texas DOT, Yeon et al. (2009) confirmed the findings of previous studies which indicated that "the maximum CTE value (for cement paste) occurs at about 70% RH and its value is almost twice the value at 100% RH."

Although a number of studies on concrete's CTE are available in the literature, significantly less is published on the other two M-EPDG thermal inputs, heat capacity and thermal conductivity. Kim et al. (2003) found that the key factors affecting the thermal conductivity of concrete are the aggregate volume fraction and moisture condition, and that the thermal conductivity of concrete is only sensitive to age at very early ages. Shin and Kodide (2012) performed a study on the thermal conductivity of ternary mixtures for concrete pavements. Their study indicated that the thermal conductivity of mixtures included in their study was "significantly affected by the type and percentage of coarse aggregate (type) and moisture content." As part of their study, Shin and Kodide (2012) developed "a model equation that predicts the thermal conductivity of concrete as a function of moisture content and coarse aggregate percentage."

The influence of heat capacity and thermal conductivity on predicted pavement performance have been evaluated using M-EPDG software, as in a study by Guclu et al. (2009) for Iowa DOT and Johanneck et al. (2011) as part of a Pooled Fund Study. In the sensitivity analysis performed by Guclu et al. (2009), the PCC CTE and thermal conductivity had the greatest impact on distresses related to smoothness, and therefore it was recommended that Level 1 (test-obtained) values be utilized for these inputs. Using data obtained from measurements at the MnROAD facility, Johanneck et al. (2011) found that the thermal conductivity input for both the asphalt concrete (AC) and PCC layers "significantly influenced predicted pavement performance for MEPDG simulations." The sensitivity analysis performed by Guclu et al. (2009) concluded that due to the effects of material thermal properties on the M-EPDG predicted pavement performance, "evaluation of the material thermal inputs should be part of a process of local calibration and adaptation of the MEPDG."

2.2 Summary Literature Review on Portland Limestone Cements

Portland limestone cement is a blended cement which contains some proportion of ground limestone in addition to the typical clinker. Several benefits are associated with reducing the quantity of ground clinker in cement blends, such as lowered cost, reduced heat of hydration and lowered environmental impacts. Unlike pozzolonic materials, adding limestone primarily provides mechanical improvements during the clinker grinding process and during mixing and consolidation through better particle packing. Although limestone can be added separately to portland cement, PLCs are typically commercially produced by intergrinding limestone with the clinker. This process improves the grain size distribution of the blended cement.

The environmental benefits to using PLC are substantial. The production of portland cement clinker is a key contributor to global greenhouse gas emissions. The carbon footprint of concrete is strongly linked to the calcination process that is inherent to clinker production. Because the limestone is minimally processed and is not calcined, intergrinding limestone with OPC clinker results in a lower carbon footprint by way reduced fuel consumption and avoidance of calcination-linked emissions (Tennis et al. 2011). Research into the use of PLC in conjunction with other SCMs has shown even more benefits related to sustainability. In a November 2012 presentation in a Transportation Research Board (TRB) sponsored webinar on PLC concrete, "Performance of PLC Concrete: Fresh, Hardened and Durability Properties," Michael D.A. Thomas of the University of New Brunswick (a leading PLC concrete researcher in Canada) reported that using a combination of PLC or blended PLC together with SCM additions at the concrete blends can translate to CO_2 reductions of the order of 1 to 1½ tons per truckload of concrete (Tennis et al 2011).

To optimize performance of the PLCs, both the clinker and limestone are ground finer than in ordinary portland cement. This provides better particle packing (Thomas et al. 2010), and also facilitates better hydration of cement paste and ultimately increased strength and durability performance (Tennis et al. 2011). Limestone interground with OPC contributes to the hydration process through particle packing effects, nucleation effects, and chemical reactions (Tennis et al. 2011). The improved particle size distribution allows for smaller limestone particles to intersperse in the void space between larger grains of cement and limestone. The limestone particles provide an increased number of nucleation sites. These additional surfaces for precipitation of hydration products, speed up hydration reactions, and result in higher early age strengths (Thomas and Hooton 2010). The presence of very fine limestone in the void spaces also tends to decrease water demand (Hawkins et al. 2003). The chemical composition of the limestone facilitates increased reaction with calcium aluminates in the OPC or SCM, forming calcium carboaluminates (Tennis et al. 2011).

PLCs have been successfully utilized in Europe for over 25 years (Hooton et al. 2007), and are being increasingly utilized in Canada, where PLCs with up to 15% limestone are allowed in all applications with the exception of sulfate-exposed applications (Thomas et al. 2010a). International and domestic experience have resulted in several standards and specifications to govern the application of PLC. Currently, interground limestone is limited to 15% in Canada by Canadian Standard CSA A3001, although 35% interground limestone is permitted by European standard EN 197-1 (European Committee for Standardization 2000, Canadian Standards Association 2013). In the United States, OPC may contain up to

5% limestone without additional labeling requirements. In cases that exceed this threshold, US specifications for PLC are outlined in AASHTO M 240 and ASTM C595, "Standard Specification for Blended Hydraulic Cements" (AASHTO 2017, ASTM 2015). These documents outline the Type IL Portland-limestone blended cement, which includes a designation for 10% limestone replacement, (e.g. Type IL(10) is PLC with a 10% limestone replacement). Requirements for the composition of limestone in binary and ternary blends including slag, fly ash, and other SCMs, are also provided.

In the United States, pilot projects using PLC concrete have been implemented in Utah and Colorado, with over 125 miles of PLC concrete pavement in place in these two states as of Fall 2012 (Laker and Smartz 2012). Utah DOT pilot projects include highway pavements in metropolitan areas (SR 20 and 104^{th} South in Salt Lake City and UTA FrontRunner South from Salt Lake City to Provo) and rural county roads. Additionally, Utah DOT has experience in use of PLC in a segmental block retaining wall. Colorado DOT has utilized PLC concrete in a number of residential street projects and arterial roadway projects in Denver, as well as in highway pavements such as I-25 near Castle Rock and US 287 near Lamar (Laker and Smartz 2012).

Some US states have written specifications around the ASTM and AASHTO documents. Colorado DOT and Utah DOT currently allow PLC that meet ASTM C1157 performance specifications for GU (General Use), MS (Moderate Sulfate Resistance) and HS (High Sulfate Resistance) (ASTM 2011). Specifications for these two state agencies also require inclusion of SCMs in concrete mixtures used in applications that could be susceptible to alkali-silica reactivity (ASR) and/or sulfate attack (Laker and Smartz 2012). Louisiana DOTD currently allows use of PLC in concrete, and has published Special Provision HGR 05-07-13 for use on pilot concrete pavement projects to test new standard specifications. In Section 001.08.1 Cement of this Special Provision, "Type IL portland limestone cement" is listed as an allowable cement type for General Construction (Structural Class Concrete and Minor Structure Class Concrete), Concrete Pavement, and Prestressed or Precast Concrete.

Several studies indicate that PLC concrete can exhibit performance similar to concrete produced with OPC. Laker and Smartz (2012) indicated that field experience on Colorado DOT and Utah DOT projects, PLC concrete shows strength gain, set time, water demand and compatibility with fly ash and admixtures that was similar to OPC concrete. However, Hawkins et al. (2003) and Vuk et al. (2001) have also found conditions in which the Vicat set times for PLC concretes are shortened by up to an hour. The durability performance of PLC concrete has been shown to be equivalent to OPC concrete mixtures from the same clinker (Thomas and Hooton 2010). Studies performed in the early 2000s by Dhir et al. (2007) on PLC concrete produced in the United Kingdom indicated that the permeability and durability of PLC, including initial surface absorption, carbonation resistance, chloride diffusion, freeze/thaw scaling and abrasion resistance, generally followed proportional relationships with strength for most properties.

Extensive research into the durability performance of PLC Concrete has been conducted in Canada by Thomas and Hooton (2010). PLC with interground limestone replacements ranging from 3% to 19% were utilized in concrete mixtures with Canadian aggregates. Samples were prepared with varying *w/cm* ratios and a range of cement contents. Testing was performed to evaluate fresh properties, early age properties, and durability performance, along with methods to evaluate rapid chloride permeability, freeze/thaw resistance, salt scaling, shrinkage, sulfate resistance, and ASR expansion. This study concluded that for PLC with up to 15% limestone, equivalent performance as OPC from the same clinker was observed (Thomas and Hooton 2010). In tests performed for Colorado DOT and Utah DOT, Laker and Smartz (2012) indicate that in some tests for alkali-silica reactivity (ASR) (ASTM C1260 and ASTM C1567) and rapid chloride ion permeability (ASTM C1202), PLC concrete mixtures performed better (or slightly better) than OPC concrete mixtures.

There is a possibility that PLC concrete can be more susceptible to the thaumasite form of sulfate attack, or TSA, as a result of the carbonate ions that are present in the limestone. Wet, sulfate-rich conditions are required for TSA to occur and colder climates can increase the possibility of it initiating. Hooton and Thomas (2002) studied the susceptibility of Canadian PLC concrete to TSA and found that there were no cases of TSA related to use of PLC in concrete in Canada. However, research on TSA in concrete produced using PLC with limestone replacements at high levels (15% to 35%), or where carbonate fines are contributed by the aggregate, was not available for cold, wet and sulfate-rich areas (Hooton and Thomas 2012). The most commonly utilized test for TSA is CSA A3004-08 Procedure B, which is a modified version of the ASTM C1012 sulfate resistance test that is performed at low temperatures (5°C). Recently, leading researchers have criticized this test, calling it "overly severe" due to the potential for PLC and SCM combinations with a satisfactory history of field performance failing the test, conflicting results where mixtures with PLC perform better than non-PLC mixtures, and the "low maturity" of specimens when immersed in the sulfate solution at a low temperature (which stifles hydration reactions, particularly of SCMs). These researchers recommend modifying CSA A3004-08 Procedure B to "ensure sufficient hydration maturity" of the samples before subjecting them to the test (Barcelo et al. 2014)

Information on the performance of field installations of PLC concrete is becoming more readily available, as the number of pilot projects increases, and early pilot projects have been in service for several years. PLC concrete mixtures in Canada have been extensively studied, and a number of successful field trials have demonstrated suitable performance in pavements and other applications (Thomas et al. 2010a and 2010b). Data on the performance of field trials in Canada are available for privately-owned pavements at cement plants in the Canadian Provinces of Alberta, Quebec, and Nova Scotia (Thomas et al. 2010b). PLC concrete mixtures used in a trial pavement in Gatineau, Quebec exhibited similar or slightly improved resistance to chloride permeability when compared to companion OPC mixtures (Tennis et al 2011). Thomas reported that, based on the Canadian laboratory research and field studies, PLC with 12% limestone, when optimized for equal strength, can provide equivalent performance to Portland cement (Type PC).

In a 2008 paper in Cement and Concrete Research, “Bridging the gap between research and standards,” Hooton and others provided guidance in implementation of new Canadian standards based on research with Canadian materials. The 2010 National Building Code of Canada (NBCC) provides provisions for use of PLC, following the inclusion in CSA standard A3001-08, “Cementitious Materials Compendium” and CSA A23.1 “Concrete Materials and Methods of Concrete Construction” standard (Canadian Commission on Building and Fire Codes 2010). The trend towards specifications allowing PLC concrete to be utilized in transportation applications is moving rather rapidly through the United States, with many state agencies either allowing or considering allowing PLC via provisions in specifications. Oklahoma, Utah, Iowa, Missouri, and Louisiana currently allow the use of PLCs, and it appears that additional states are either considering or planning on allowing use of PLC in the near future.

3. LABORATORY TESTING PROGRAM AND RESULTS

3.1 Materials Description and Characterization

Similar to the approaches utilized by other states for determining local concrete inputs for M-EPDG, representative coarse aggregates from the Mountain, Piedmont, and Coastal regions of North Carolina were obtained and characterized. However, several market-driven and sustainability-driven factors warranted inclusion of mixtures utilizing materials not included in other M-EPDG concrete input research. To facilitate performance comparison and identification of input variation, this study compared predicted performance between mixtures containing:

- natural silica sand (meeting ASTM C33) and a manufactured sand (granitic gneiss) meeting NCDOT specification for 2MS (NCDOT 2012)
- two sources of OPC (from sources in two neighboring states) and PLC (produced using clinker from one of the PLC sources)
- two class F fly ash sources (used with both PLC and OPC mixtures)

A description of material characteristics is provided in the following sections.

3.1.1 Cementitous Materials

Two different portland cements, both Type I/II cements meeting ASTM C150, “Standard Specification of Portland Cement” were used in this research. Cement A was produced in Tennessee, and is a typically used cement for the Mountain region of North Carolina. Cement B was produced in South Carolina, and is a commonly used cement for the Coastal and Piedmont regions of North Carolina. Mill reports for both cements are provided in Appendix B as Figures B.1 and B.2. The PLC used in this research is a Type IL cement that was produced at the same South Carolina plant as cement B, and is therefore referred to as Cement BL. The PLC was produced using the same clinker as Cement B, with less than 15% limestone added per ASTM C595, “Standard Specification for Blended Hydraulic Cements.” The mill report for this PLC is also provided in Figure B.2 in Appendix B.

Several concrete mixtures in this research study utilized fly ash, obtained from two different sources. North Carolina allows for replacement of cement with fly ash in accordance with North Carolina Standard Specifications section 1024, “Materials for Portland Cement Concrete,” at a rate of 1.2 pounds of Class F fly ash per pound of cement replaced up to 20%. 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. Both fly ashes, fly ash A and fly ash B, are classified as Class F fly ash, and additional information is provided in Figures B.3 and Figure B.4 in Appendix B.

3.1.2 Coarse Aggregates

The coarse aggregate sourced from the Mountain region of North Carolina was a granitic gneiss (specific gravity of 2.62 and absorption of 1.1%). The coarse aggregate sourced from the Piedmont region was also a granitic gneiss, but from a different geologic formation (specific gravity of 2.62 and an absorption of 0.8%). The Coastal coarse aggregate was a marine limestone (specific gravity of 2.42 and absorption of 2.4%). Each coarse aggregate met the gradation requirements of No. 67 stone. Sieve analysis results for the coarse aggregates used in this study are provided in Appendix B, Tables B.1 through B.3.

3.1.3 Fine Aggregates

Both fine aggregate samples were selected from the Piedmont region due to their central location in the state, as well as the prevalence of much of North Carolina’s concrete pavement construction in this region. The primary fine aggregate chosen for this research, is a manufactured sand (specific gravity of 2.65, absorption of 0.3%, fineness modulus 2.54) meeting NCDOT’s 2MS specification. A natural sand (specific gravity 2.64, absorption 0.74%, and a fineness modulus of 2.54) meeting ASTM C33 was used in three of the eighteen mixtures. Sieve analysis results for the fine aggregates used in this study are shown in Appendix B, Tables B.4 and B.5.

3.1.4 Admixtures

A commercially available air entraining admixture (MasterAir AE 200 manufactured by BASF) and a mid-range water-reducing admixture (MasterPolyheed 997 manufactured by BASF) were utilized in all mixtures. The target slump was 1.5 inches without water reducing admixture, with an allowable increase in slump of 1 to 3 inches after use of water reducing admixture to facilitate consolidation of concrete in test specimen molds. 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.

3.2 Concrete Mixtures

Based on discussions with NCDOT personnel on the project steering committee, the concrete mixture matrix shown in Figure 3.1 was developed. As shown in Figure 3.1, the mixture matrix consisted of eighteen total concrete mixtures, each a variation in materials usage from a typical concrete paving mixture used in North Carolina. In summary, the eighteen concrete mixtures can be grouped as follows:

- Base 9 mixtures (shown in orange) - coarse aggregate source was varied between Coastal, Piedmont, and Mountain
- Mixtures with sand type variation (shown in blue) - natural sand used instead of manufactured sand
- Mixtures with fly ash (shown in yellow and green) - 20% fly ash replacement from two fly ash sources

Parameters held constant for all eighteen mixtures included a w/cm of 0.48, a coarse aggregate content of 11 cf/cy, a target air content of 5.5% ± 0.5%, a maximum slump without water reducing admixture of 1.5 inches, and a target slump with water reducing admixture of 1 to 3 inches. All straight cement mixtures (shown in blue and orange) had a cement content of 573 pcy, and all fly ash mixtures (shown in yellow and green) had 20% fly ash replacement rates, resulting in 480 pcy of cement and 144 pcy of fly ash. The Piedmont coarse aggregate was utilized as the “control” coarse aggregate for the mixtures where sand type was varied (shown in blue) and for the mixtures where fly ash was utilized (shown in yellow and green). A mid-range water reducer was used in all mixtures to achieve the target slumps.

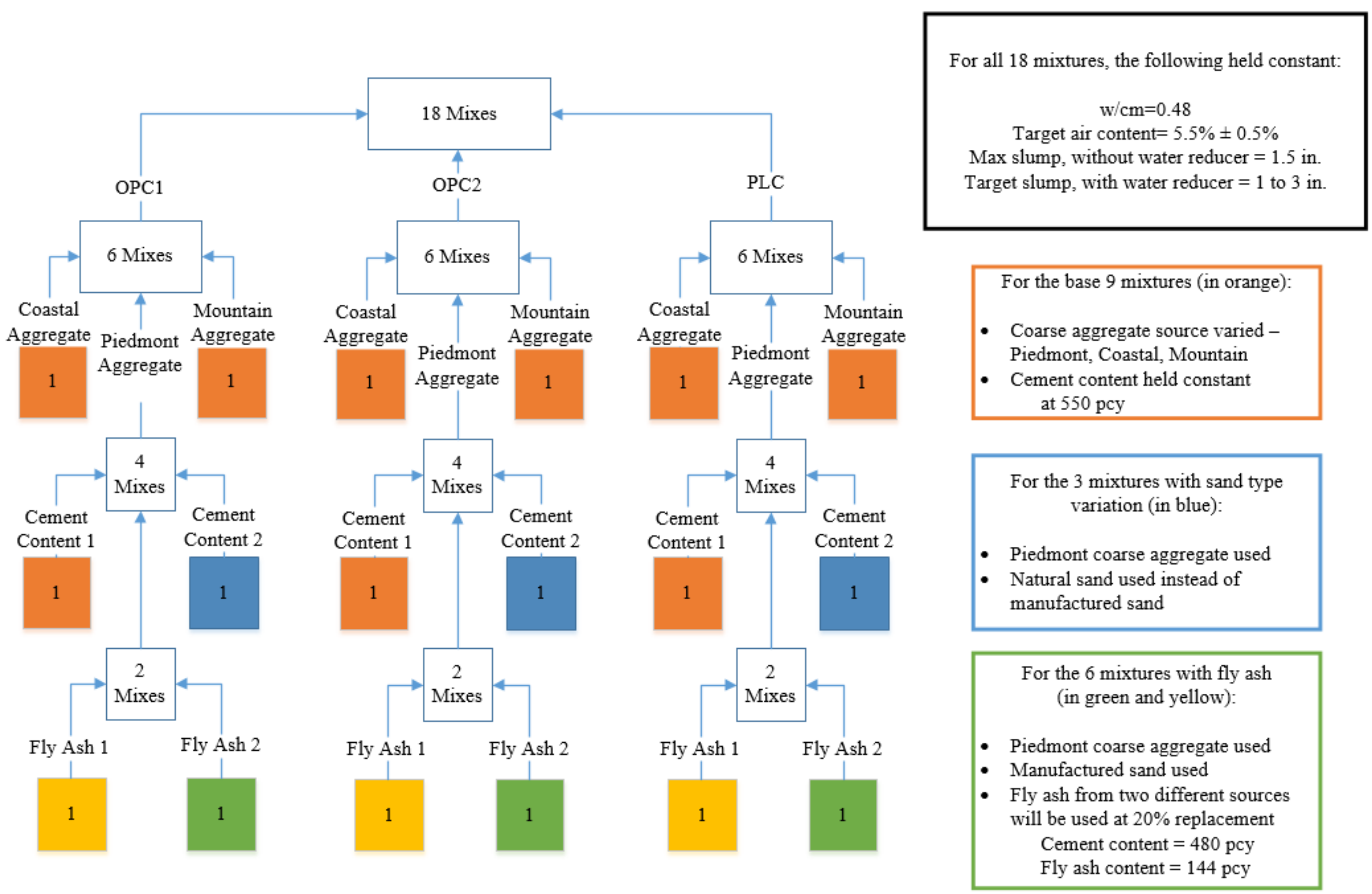

Figure 3.1: Concrete mixture matrix with supporting details

The concrete mixtures developed for this project were each given a mixture ID, which is summarized as follows:

- First letter, coarse aggregate type: C = Coastal, P = Piedmont, M = Mountain
- Second letter, cement type: A = OPC source A, B = OPC source B, C = PLC
- Third letter, fly ash type: N = None, A = fly ash source A, B = fly ash source B
- Fourth letter, fine aggregate type: M = manufactured sand, N = natural sand

A summary of the mixtures, along with the proportions of materials used in each mixture, is presented in Table 3.1.

Table 3.1: Concrete mixtures with material proportions

Mixture ID*	Material Types				Mixture Proportions, pcy				
	Cement Type and Source	Coarse Aggregate	Fine Aggregate	Fly Ash	Cement	Fly Ash	Coarse Aggregate	Fine Aggregate	Water
C.A.N.M	OPC Source A	Coastal	Manufactured Sand	None	573	0	1661	1260	275
M.A.N.M		Mountain	Manufactured Sand	None	573	0	1798	1260	275
P.A.N.M		Piedmont	Manufactured Sand	None	573	0	1798	1260	275
P.A.N.N			Natural Sand	None	573	0	1798	1184	275
P.A.A.M			Manufactured Sand	Source A	460	137	1798	1260	304
P.A.B.M			Manufactured Sand	Source B	460	137	1798	1260	304
C.B.N.M	OPC Source B	Coastal	Manufactured Sand	None	573	0	1661	1260	275
M.B.N.M		Mountain	Manufactured Sand	None	573	0	1798	1260	275
P.B.N.M		Piedmont	Manufactured Sand	None	573	0	1798	1260	304
P.B.N.N			Natural Sand	None	573	0	1798	1184	304
P.B.A.M			Manufactured Sand	Source A	460	137	1798	1260	275
P.B.B.M			Manufactured Sand	Source B	460	137	1798	1260	275
C.BL.N.M	PLC (produced using OPC from Source B)	Coastal	Manufactured Sand	None	573	0	1661	1260	275
M.BL.N.M		Mountain	Manufactured Sand	None	573	0	1798	1260	275
P.BL.N.M		Piedmont	Manufactured Sand	None	573	0	1798	1260	275
P.BL.N.N			Natural Sand	None	573	0	1798	1184	275
P.BL.A.M			Manufactured Sand	Source A	460	137	1798	1260	304
P.BL.B.M			Manufactured Sand	Source B	460	137	1798	1260	304

*Note: Explanation of Mixture ID coding:

First letter, coarse aggregate type: C = Coastal, P = Piedmont, M = Mountain

Second letter, cement type: A = OPC source A, B = OPC source B, BL = PLC

Third letter, fly ash type: N = None, A = fly ash source A, B = fly ash source B

Fourth letter, fine aggregate type: M = manufactured sand, N = natural sand

3.3 Testing Program and Results

The testing program for fresh and hardened concrete properties performed as part of this research study is summarized in Table 3.2. In addition to tests to confirm the fresh properties met the targets described in Section 3.2, tests were performed to evaluate the fresh mechanical properties, thermal properties, and durability performance of each of the eighteen mixtures.

Table 3.2: Testing program

		Test	Protocol	Age(s) in days	Replicates
Fresh Concrete Tests		Air content	ASTM C231	Fresh	1
		Slump	ASTM C143	Fresh	1
		Fresh density (unit weight)	ASTM C138	Fresh	1
		Temperature	AASHTO T 309	Fresh	1
Hardened Concrete Tests	Mechanical Properties	Compressive strength	ASTM C39	3, 7, 28, 90	3 each age
		Modulus of elasticity and Poisson's ratio	ASTM C469	28	2
		Modulus of rupture	ASTM C78	28	2
	Thermal Properties	Coefficient of thermal expansion	AASHTO T 336	28	3
		Thermal conductivity	ASTM C518	56, after 49 day wet cure and 7-day conditioning at 50% RH	3
		Heat capacity			3
	Durability Performance	Resistivity	AASHTO TP 95-11	3, 7, 28, 90	3 each age
		Rapid chloride permeability	ASTM C1202	28	2
		Freezing and thawing resistance	ASTM C666, procedure A	per standard	3
		Shrinkage	ASTM C157	per standard	3
		Cracking potential	ASTM C1581	per standard	3
		Thaumasite attack *	CSA A3004-C5	per standard	3 at each temperature

* Mortar was batched separately from the concrete used for the rest of the test program.

Each mixture was prepared in four batches, allowing the research team to mix adequate quantities of concrete for groups of tests and offering scheduling flexibility for more efficient testing. The fresh concrete tests listed in Table 1 were measured for each batch. For each batch, a set of compressive strength cylinders was also cast in order to verify consistency between batches. A breakdown of the batches for each of the eighteen mixtures is outlined below:

- Batch 1: cracking potential rings and rapid chloride penetration test (RCPT)
- Batch 2: freeze thaw test and super air meter
- Batch 3: modulus of elasticity, heat capacity, and thermal conductivity
- Batch 4: compressive strength, modulus of rupture, and shrinkage

3.3.1 Fresh Concrete Properties

In order to mitigate the influence of a wide range of air contents on the test results, a decision was made to restrict air content to 5.0% to 6.0%. Batches not meeting an air content between 5.0% to 6.0% (measured using the Type B pressure meter) were discarded and the batch was remixed. This relatively tight acceptable air content resulted in the wasting of a number of batches of concrete for air contents outside of this narrow range. However, review of the test results indicates that this was a sound decision, as general trends likely attributable to materials (and not air content differences) are evident in hardened concrete test results. A summary of test results for each fresh concrete property test for each batch is presented in Appendix B. Slump test results are provided in Table B.6, ASTM C231 air content test results are provided in Table B.7, unit weight test results are provided in Table B.8. SAM test results for select mixtures (Batch 2, from which freeze-thaw specimens were produced), along with corresponding C231 air content measurements, are presented in Table B.9.

3.3.2 Mechanical and Thermal Properties

Mechanical and thermal property tests were performed using the test methods outlined in Table 3.2. A summary of these results (typically the average of two or three specimens) is provided in Table 3.3. Supporting data, providing the result of each test and averages/standard deviations is provided in Appendix B, including 28-day compressive strength for each batch (batches 1-3, Table B.10), 28-day modulus rupture (Table B.11), 28-day modulus of elasticity (Table B.12), Poisson's ratio (Table B.13), and CTE (Table B.14).

Most of these test methods are fairly conventional. Tests for heat capacity and thermal conductivity, however, are rarely included in studies to determine concrete inputs for M-EPDG due to issues with the recommended test methods (Shin and Kodide 2012). For this study, a method of testing the bulk thermal conductivity of intact concrete specimens was used. Tests were performed using the Fox50 Heat Flow Meter Instrument by TA Instruments in accordance to ASTM C518, "Standard Test Method for Steady-State Thermal Transmission Properties by Means of the Heat Flow Meter Apparatus." Three representative rectangular prisms approximately 1.5 inch by 1.5 inch by 1 inch thick were sawcut from a 4 in diameter by 8 inch concrete cylinder which had been wet cured for 49 days. Specimens were then placed in an environmental chamber at 72°F and 50% relative humidity for seven days prior to testing. Three specimens were tested for each mixture at age 56 days.

For thermal conductivity testing, the temperature at one end of the test chamber was set to 20°C (68°F), while the temperature at the other end of the chamber was set to 30°C (86°F), with the value for thermal conductivity obtained at 25°C (77°F). Average results are presented in Table 3.3, with supporting data for each test specimen presented in Appendix B, Table B.15. For volumetric heat capacity testing, results were obtained at 25°C (77°F), using two temperature steps, 20°C and 30°C (68°F and 86°F). Average results are presented in Table 3.3, with supporting data for each test specimen presented in Appendix B, Table B.16. Test results were adjusted to account for the thermal characteristics of the cushioning pads and parchment paper that were placed in series with the specimens to protect the sensor coatings per the equipment manufacturer's instructions. The capability of the equipment to accommodate large samples offered several benefits over the more typical use of small, pulverized concrete samples that are required for differential scanning calorimetry testing due to the equipment's very small crucible volume. The larger, bulk concrete specimens maintain distinct aggregate and paste phases and can be accurately conditioned to a particular moisture state. This allowed the direct comparison of results for the impact of changes in materials on these thermal inputs.

For each of the eighteen mixtures, mechanical and thermal property test results are summarized in Table 3.3. Averages were computed for groups of mixtures based on aggregate type, cement type, and presence/absence of fly ash for further evaluation, as well as identification of a recommended catalog of M-EPDG inputs. The recommended catalog of inputs for use by NCDOT in M-EPDG is presented in Table 3.4. Following Table 3.4 is a discussion of these results and a description of how the proposed catalog values were identified.

Table 3.3. Results of laboratory testing for mechanical and thermal properties

Mixture ID*	Unit Weight, pcf	Compressive Strength, psi	Modulus of Elasticity, psi	Poisson's Ratio	MOR, psi	CTE, in/in/°F	Thermal Conductivity, BTU/(ft·hr·°F)	Heat Capacity, BTU/(lb·°F)
C.A.N.M	138	5,360	3,730,000	0.22	730	4.23×10^{-6}	0.810	0.219
M.A.N.M	146	5,030	2,540,000	0.18	570	4.46×10^{-6}	0.870	0.199
P.A.N.M	145	5,020	2,920,000	0.20	680	4.57×10^{-6}	0.920	0.203
P.A.N.N	142	5,400	3,400,000	0.15	740	5.40×10^{-6}	1.24	0.202
P.A.A.M	141	4,270	3,220,000	0.23	650	4.42×10^{-6}	0.890	0.201
P.A.B.M	141	3,780	2,840,000	0.22	570	4.43×10^{-6}	0.890	0.201
C.B.N.M	139	5,960	3,490,000	0.21	750	4.28×10^{-6}	0.890	0.216
M.B.N.M	145	5,100	2,760,000	0.20	640	4.46×10^{-6}	0.950	0.206
P.B.N.M	143	4,850	3,340,000	0.20	670	4.63×10^{-6}	0.950	0.203
P.B.N.N	142	4,390	3,510,000	0.19	720	5.31×10^{-6}	1.12	0.201
P.B.A.M	143	4,050	2,700,000	0.21	540	4.46×10^{-6}	0.900	0.202
P.B.B.M	141	3,140	2,510,000	0.18	620	4.52×10^{-6}	0.900	0.203
C.BL.N.M	139	5,560	3,690,000	0.22	680	4.30×10^{-6}	0.870	0.202
M.BL.N.M	145	4,790	3,020,000	0.20	610	4.56×10^{-6}	0.910	0.203
P.BL.N.M	144	5,020	2,430,000	0.18	660	4.54×10^{-6}	0.800	0.202
P.BL.N.N	141	5,190	3,040,000	0.15	750	5.32×10^{-6}	1.18	0.196
P.BL.A.M	142	3,750	2,690,000	0.16	650	4.57×10^{-6}	0.900	0.196
P.BL.B.M	141	3,780	2,720,000	0.19	560	4.56×10^{-6}	0.880	0.203

*Note: Explanation of Mixture ID coding:
First letter, coarse aggregate type: C = Coastal, P = Piedmont, M = Mountain
Second letter, cement type: A = OPC source A, B = OPC source B, BL = PLC
Third letter, fly ash type: N = None, A = fly ash source A, B = fly ash source B
Fourth letter, fine aggregate type: M = manufactured sand, N = natural sand

Table 3.4: Proposed catalog of M-EPDG inputs for concrete

Materials			M-EPDG Input						
Coarse Aggregate	Fine Aggregate	Fly Ash	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)
Piedmont	Manufactured Sand	No	145	3,000,000	0.19	660	4.63×10^{-6}	0.22	0.95
Piedmont	Manufactured Sand	Yes	142	2,500,000			4.57×10^{-6}		0.90
Piedmont	Natural Sand	No	142	3,400,000	0.16	740	5.40×10^{-6}		1.20
Mountain	Manufactured Sand	No	146	2,700,000	0.19	660	4.56×10^{-6}		0.95
Coastal	Manufactured Sand	No	139	3,500,000			4.30×10^{-6}		0.90

The lower specific gravity of the Coastal coarse aggregate (a marine limestone) caused a lower unit weight input for these concrete mixtures. Poisson's ratios for mixtures containing manufactured sand were consistent for mixtures with all types of coarse aggregates (0.19). However, for mixtures containing natural sand, a lower Poisson's ratio was measured, and 0.16 is suggested for use based on an average of the test results. The MOR values for mixtures with the natural sand were significantly higher than the test results for mixtures with manufactured sand and average values of 660 psi and 740 psi are recommended for manufactured and natural sand mixtures, respectively.

The use of PLC did not appear to adversely affect any of the mechanical or thermal properties of the concrete mixtures. Mixtures utilizing the PLC (denoted in Tables 3.1 and 3.3 with BL as the second letter in the Mixture ID) exhibited similar performance to mixtures utilizing cements A and B. This finding is similar to those of many studies, and supports the decision of NCDOT to allow PLC to improve the sustainability of NC highway infrastructure. Inputs for MEPDG for PLC concrete should be the same utilized as those for concrete utilizing conventional portland cement.

The coarse aggregates utilized in concrete mixtures have historically been targeted as most influential in the CTE of concrete, and it is acknowledged that the concrete CTE measurements obtained in this study were largely influenced by the CTE of the coarse aggregates utilized. However, in this study which used only North Carolina aggregate sources, the material with the most surprising effect on CTE appears to be fine aggregate, with a large difference evident between mixtures containing manufactured sand, and mixtures containing natural sand. Mixtures that included the manufactured sand had CTE values between 4.23 x 10^{-6} and 4.57 x 10^{-6} in/in/°F. However, the CTE of the mixtures using the natural sand averaged a much higher 5.40 x 10^{-6} in/in/°F, ranging from 5.31 x 10^{-6} to 5.40 x 10^{-6} in/in/°F. These are lower than the AASHTO recommended input for granite of 5.8 x 10^{-6} in/in/°F (AASHTO 2015). The natural silica sand likely had a higher CTE than the manufactured sand, which was produced from a metamorphosed granitic gneiss material. Quartzite 6.1 x 10^{-6} to 7.2 x 10^{-6} in/in/°F) and sandstone (6.1 x 10^{-6} to 6.7 x 10^{-6} in/in/°F) have been shown to have higher CTEs, than granitic aggregates 4.0 x 10^{-6} to 5.0 x 10^{-6} in/in/°F) (Wang et al. 2008), providing a supporting cause for these CTE test results.

The effect of coarse aggregate type on CTE was also observed, but to a lesser extent. The coastal aggregate mixtures had lower CTE values with a range of 4.23 x 10^{-6} to 4.30 x 10^{-6} in/in/°F, with an average of 4.30 x 10^{-6} in/in/°F suggested for use. The Piedmont coarse aggregate mixtures that incorporated manufactured sand, including the Piedmont mixtures with fly ash, all had similar CTE values ranging from 4.42 x 10^{-6} to 4.63 x 10^{-6} in/in/°F. This is notably lower than the AASHTO recommended input for granite of 5.8 x 10^{-6} in/in/°F (AASHTO 2015). The Mountain aggregate mixtures had similar CTE values to that of the Piedmont coarse aggregate / 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} to 4.57 x 10^{-6} in/in/°F, with 4.56 x 10^{-6} in/in/°F suggested for use.

Similar to the 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 ranged from 0.80 to 0.95 BTU/(ft·hr·°F), with an average of 0.90 or 0.95 BTU/(ft·hr·°F), based on averaging of results for mixtures with similar coarse aggregate, fine aggregate, and fly ash content. This is much lower than the recommended input value of 1.25 BTU/(ft·hr·°F) (AASHTO 2015). Thermal conductivity test results for the natural sand mixtures were far closer to the MEPDG default value, with the average of 1.20 BTU/(ft·hr·°F) suggested for use.

The type of coarse aggregate utilized in a mixture appeared to have the greatest influence over the heat capacity, although minimal variation in heat capacity occurred regardless of mixture components, with of test results for all eighteen mixtures ranging from 0.20 to 0.22 BTU/(lb·°F). Heat capacity test results for all of the Piedmont and Mountain coarse aggregate mixtures were typically 0.20 BTU/(lb·°F), whereas typically the heat capacity test results for the Coastal coarse aggregate mixtures were typically 0.22 BTU/(lb·°F). These results are again significantly lower than the recommended input value of 0.28 BTU/(lb·°F) (AASHTO 2015).

Additional tests were performed to broaden the dataset obtained for thermal conductivity and heat capacity. Although several published sources provide generally accepted values for concrete thermal conductivity and heat capacity, most do not provide information on the test method utilized, specimen composition, or sample conditioning. Very few recent studies have been identified that consider the influence of moisture state on these thermal properties, which are highly sensitive to the material's moisture content. For the Catalog of Inputs presented in Table 3.4, thermal conductivity and heat capacity values are those obtained for specimens tested at 50% relative humidity (RH) conditioning (three specimens per mixture). The research team performed testing on the same specimens conditioned to two different moisture states, saturated surface dry (SSD) and oven dried, to evaluate the role of moisture content on these thermal inputs.

Average thermal conductivity test results for the oven dried, 50% RH and SSD specimens are shown in Figure 3.2, with supplemental test results shown in Table B.17 and B18 in Appendix B. As can be seen in Figure 3.2, thermal conductivity increased with moisture content. This is consistent with other published literature. The natural sand mixtures (the purple, orange and blue lines above the cluster of other lines below) exhibited higher thermal conductivities in both the oven dried and SSD states, as well as the results of tests at the 50% RH state presented in Table 3.3.

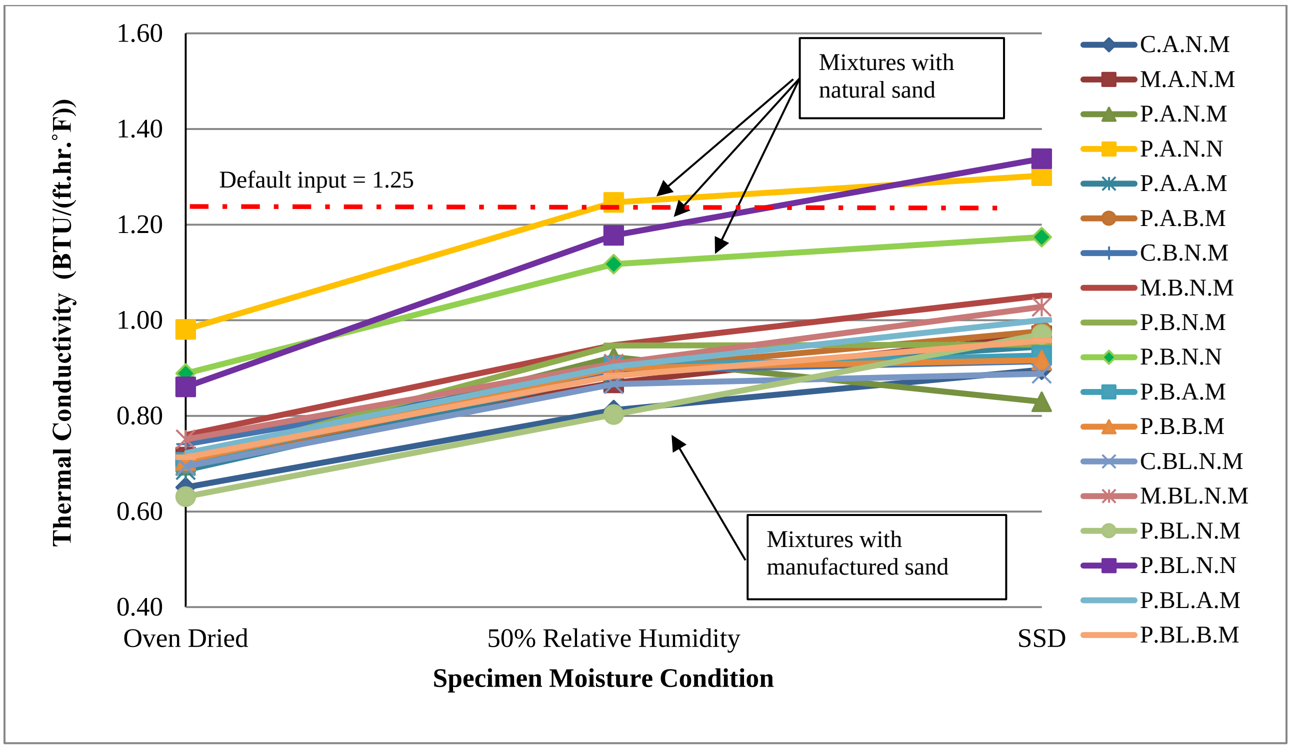

Figure 3.2: Thermal conductivity results at different moisture states

In Figure 3.3, test results for heat capacity for the three moisture states are shown. Overall, these results show smaller variation based on moisture state. Of interest, the heat capacity of the SSD specimens was found to be slightly lower than the heat capacity of specimens measured at 50% relative humidity. This is contrary to often cited work published by Valore (1980) at PCA (only SSD values measured, others computed based upon a derived equation), but is consistent with more recent work published by Kodide (2010) at Louisiana State University.

It is generally accepted that concrete pavements in service have moisture gradients that are typically between 50% and 100% (Jansen et al. 1998), and all measured values were substantially lower than the global default value suggested for use in M-EPDG of 0.28 BTU/(lb·°F) (ARA 2004). Figures 3.2 and 3.3 indicates that although some change exists between concrete in these two states, a single value between these points could likely be representative. A summary of test results presented in Table 3.5, provides grouped averages for recommended inputs based on moisture state.

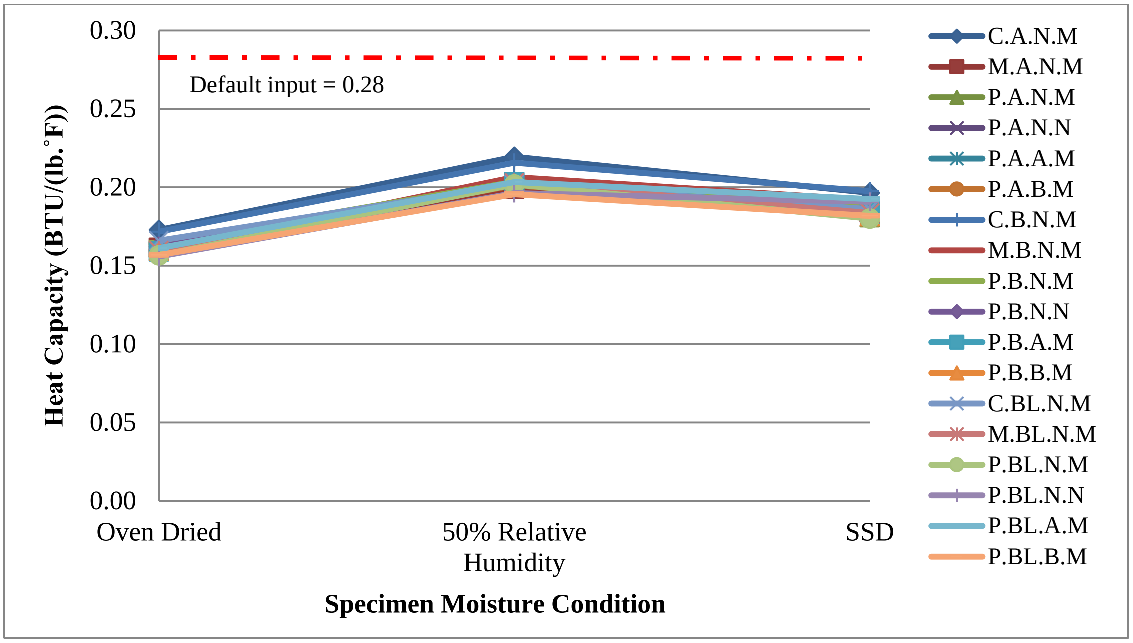

Figure 3.3: Heat capacity at different moisture states

Table 3.5: Summary of test results for thermal conductivity and heat capacity for concrete in different moisture states

	Thermal Conductivity (BTU/(ft·hr·°F))			Heat Capacity (BTU/lb·°F)		
Moisture State	OD	50% RH	SSD	OD	50% RH	SSD
Mixtures containing manufactured sand	0.705	0.890	0.948	0.162	0.203	0.186
Mixtures containing natural sand	0.910	1.181	1.271			

3.3.3 Durability Performance

Although concrete is an inherently durable material, its long term performance is strongly linked to permeability, since the transport of aggressive and deleterious agents is typically facilitated by water (Mindess et al., 2003). The most substantial permeability-related durability issue for concrete infrastructure is reinforcing steel corrosion. However, even for the plain pavement concretes that are the focus of this study, an important durability consideration is the permeability of material in proximity to the pavement joints. Salt solutions placed on roadways can enter the concrete through the joints and form calcium oxychloride, which is expansive and leads to joint spalling (Prannoy et al. 2016).

Supplementary cementitious materials (SCMs) such as fly ash decrease permeability through pozzolanic activity, which fills some internal void spaces with hydration products. Use of PLC reduces permeability by lowering the water demand necessary to achieve workability, and thereby lowers the *w/cm* of the mixture (Tennis et al., 2011). PLC and fly ash may also have positive durability impacts by consuming excess calcium hydroxide, reducing the total content of portland cement and affecting the volumetric stability of concrete. In order to evaluate the potential durability of the mixtures considered in this study, permeability characteristics were used as proxy indicators.

RCPT

Results of RCPT tests conducted at 28-days and 90-days of age are presented in Table 3.6. These results may be interpreted with the qualitative descriptors given in Table 3.7, which is provided in ASTM C1202. Higher amounts of charge passed are indicative of greater permeability to the chloride ion. In general, the change passed decreased as the concrete aged from 28 days to 90 days after casting. Mixtures that contained fly ash featured substantially lower charge passed after 90 days. The lowest permeability was discovered in specimens that contained both fly ash and PLC.

Table 3.6: Rapid chloride ion permeability test results

Mixture ID	Charge Passed (Coulombs)	
	28 Days	90 days
C.A.N.M	6,720	4,782
M.A.N.M	6,828	5,240
P.A.N.M	7,170	5,300
P.A.N.N	4,881	3,471
P.A.A.M	6,401	1,773
P.A.B.M	6,134	1,562
C.B.N.M	6,021	4,629
M.B.N.M	6,056	5,286
P.B.N.M	6,860	5,120
P.B.N.N	4,394	3,227
P.B.A.M	4,591	1,980
P.B.B.M	5,225	1,651
C.BL.N.M	6,769	5,433
M.BL.N.M	6,504	4,985
P.BL.N.M	6,550	4,540
P.BL.N.N	4,330	3,449
P.BL.A.M	3,682	1,331
P.BL.B.M	4,337	1,323

Table 3.7: ASTM C1202 RCPT index

Charge Passed (Coulombs)	Chloride Ion Permeability
>4,000	High
2,000-4,000	Moderate
1,000-2,000	Low
100-1,000	Very Low
<100	Negligible

Surface Resistivity

Results of surface resistivity tests are presented in Table 3.8. These results can be qualitatively described with the permeability rating given in Table 3.9, which reflects guidance included AASHTO TP 95-11 (AASHTO 2011). In general, the surface resistivity increased from early ages to later ages, which indicates that additional hydration time reduces permeability. Samples that included fly ash were found to have substantially higher resistivity (or lower permeability to chlorides) than samples that did not contain fly ash. The highest resistivity was measured in specimens that contained both fly ash and PLC. Only concrete with a combination of cement, interground limestone (PLC) and SCMs would be considered to have “very low” chloride ion permeability in accordance with Table 3.9. All of the concrete mixtures that did not contain fly ash would be considered to have “high” permeability to chloride ion. PLC without fly ash was not sufficient to significantly reduce the permeability of the concrete mixtures that were studied. This trend was also found from RCPT results, which were described in the previous section.

Table 3.8: Surface resistivity test results

Mixture ID	Surface Resistivity (KiloOhm-cm)			
	3 Days	7 Days	28 Days	90 Days
C.A.N.M	4.1	4.9	6.7	9.8
M.A.N.M	3.1	3.7	6.0	7.8
P.A.N.M	3.6	4.3	6.9	8.9
P.A.N.N	4.6	5.4	7.5	9.6
P.A.A.M	3.1	3.6	7.8	26.6
P.A.B.M	3.5	3.6	7.5	24.3
C.B.N.M	4.5	5.5	7.0	8.7
M.B.N.M	4.5	4.7	6.7	7.8
P.B.N.M	4.8	5.2	7.3	9.3
P.B.N.N	8.0	8.7	10.7	10.8
P.B.A.M	5.0	5.4	10.5	32.9
P.B.B.M	5.0	5.4	9.8	26.6
C.BL.N.M	4.8	5.5	6.6	8.1
M.BL.N.M	5.9	6.2	7.6	8.5
P.BL.N.M	5.0	5.4	7.6	9.1
P.BL.N.N	7.1	9.0	9.5	10.3
P.BL.A.M	4.6	5.6	12.6	37.4
P.BL.B.M	4.8	5.6	12.5	35.5

Table 3.9: AASHTO TP95-11 surface resistivity index

Resistivity measured with 4”x8” Cylinder (kΩ·cm)	Chloride Ion Permeability
<12	High
12-21	Moderate
21-37	Low
37-254	Very Low
>254	Negligible

Freeze-thaw durability

Testing to evaluate the freeze-thaw durability was performed in accordance with ASTM C666, Procedure A. A summary of test results is shown in Table 3.10 and Figure 3.4. Generally, most concrete mixtures performed well in freeze thaw testing. Many state agencies specify that to be freeze-thaw durable, concrete mixtures should have a durability factor no less than 80 at 300 cycles (shown with dotted line in Figure 3.4), although failure criteria ranging between 60 and 95 (shown with the bracket in Figure 3.4) are found in specifications. All mixtures included in this study had durability factors greater than 70. Mixtures containing manufactured sand with Piedmont or Coastal coarse aggregates had durability factors greater than 90, indicating that good freeze-thaw durability performance could be expected. Mixtures using natural sand (each of which also utilized Piedmont coarse aggregate) had durability factors ranging from 74 to 82, significantly lower than the Piedmont coarse aggregate mixtures containing manufactured sand. Mixtures that included the Mountain coarse aggregate tended to have the lowest durability factors, between 77 and 80.

Table 3.10: Freeze-thaw durability test results

Specimen ID	Cycles Completed	Mass Change (%)	Average Post Test DF
C.A.N.M	300	0.52	96.63
M.A.N.M	300	-1.41	77.92
P.A.N.M	300	0.08	95.73
P.A.N.N	300	-0.94	81.70
P.A.A.M	300	-0.79	95.59
P.A.B.M	300	-1.51	94.65
C.B.N.M	300	-0.09	99.00
M.B.N.M	300	-2.22	78.69
P.B.N.M	300	0.29	98.02
P.B.N.N	300	-1.09	81.03
P.B.A.M	300	-1.25	95.90
P.B.B.M	300	-0.72	94.65
C.BL.N.M	300	0.00	98.99
M.BL.N.M	300	-2.48	79.52
P.BL.N.M	300	0.27	100.06
P.BL.N.N	300	-1.60	74.20
P.BL.A.M	300	-2.43	94.65
P.BL.B.M	300	-1.11	94.34

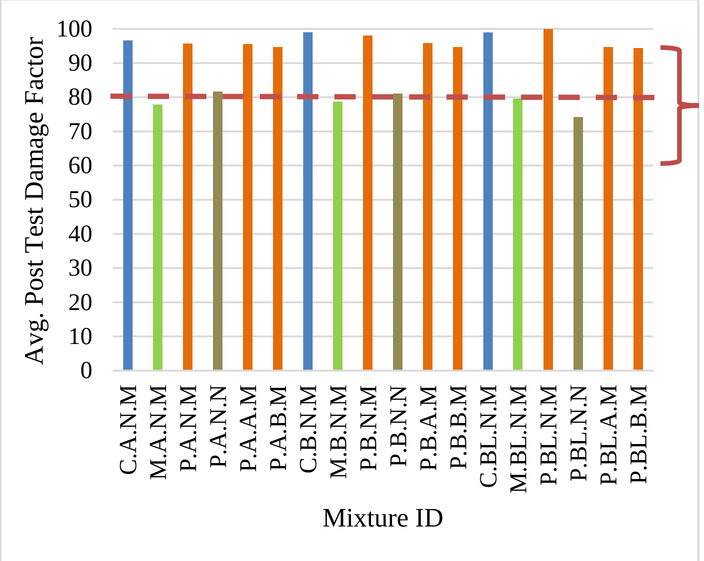

Figure 3.4: Freeze-thaw durability test results

Drying shrinkage

Shrinkage tests were performed per ASTM C157, using concrete beam specimens consisting of 4 inch by 4 inch by 11¼ inch prisms. After demolding, specimens were cured in lime water to an age of 28 days, after which they were stored in in air (drying room) at a temperature of 73 ± 3 °F and 50 ± 4% relative humidity. A length comparator apparatus was utilized to take measurements at 4, 7, 14, and 28 days, and 8, 16, 32, and 64 weeks. A summary of results of later-age shrinkage test results are shown in Figure 3.5, with supplemental data provided in Appendix B in Table B.19.

Overall, the PLC mixtures tended to exhibit slightly greater shrinkage than cement B in a number of mixtures. However, the differences in the amount of drying shrinkage observed between the PLC and OPC mixtures is judged to be very minimal. In almost all cases, at 32 weeks, the difference in drying shrinkage between the PLC mixtures and the OPC 2 (cement B) mixtures is 0.01% or less. Mokarem et al. (2003) reported that the change in length due to drying shrinkage should be less than 0.04% at 28 days and 0.05% at 90 days to reduce the probability of cracking. At 28 days (4 weeks), all mixtures exhibited length changes due to shrinkage less than 0.04%, meeting the threshold suggested by Mokarem et al. (2003). Since measurements were taken at test durations suggested in the test protocol, 90 day measurements were not made as part of this study. However, measurements taken at 8 weeks (56 days) and 16 weeks (112 days) suggest that many, but not all mixtures would also meet the suggested performance threshold of 90-day length change of 0.05% (or less) suggested by Mokarem et al. (2003). AASHTO PP 84-17, "Standard Specification with Commentary for Performance Engineered Concrete Pavement Mixtures" states a prescriptive limit of 420 microstrain at 28 days of drying for pavement applications. The 28-day microstrain for the 18 pavement mixtures tested range from 200 to 423 microstrain, with all but one mixture passing the 28-day recommended prescriptive limit in AASHTO PP 84-17. The one mixture that did not meet this recommended limit, P.BL.N.M, was only slightly over the recommended target at 423 microstrain.

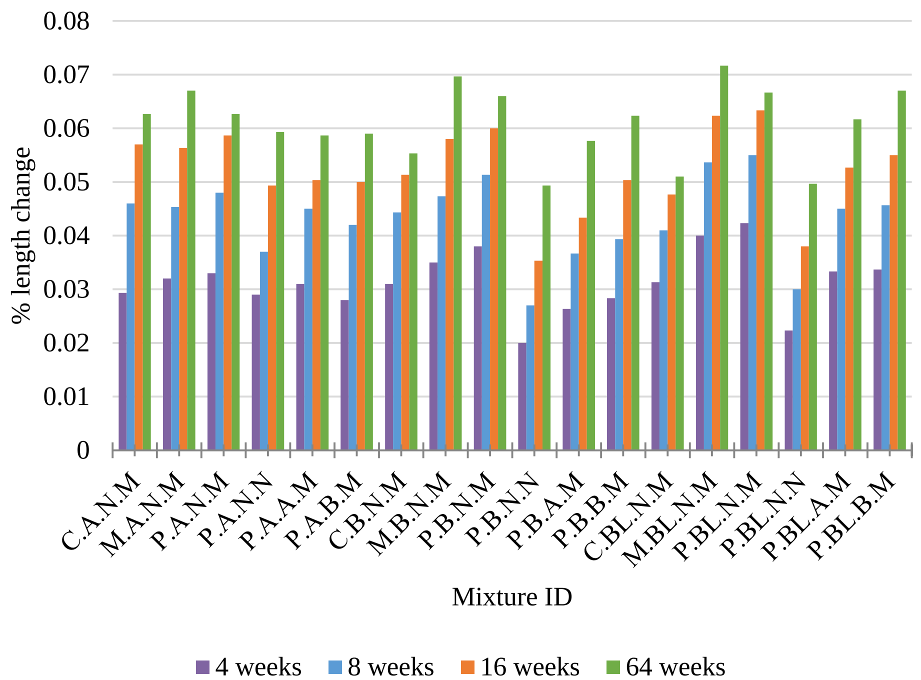

Figure 3.5 Summary of ASTM C157 shrinkage tests

Table 3.11: Strain at 28-days of drying, ASTM C157

Mixture ID	Average 28-day microstrain
C.A.N.M	293
M.A.N.M	320
P.A.N.M	330
P.A.N.N	290
P.A.A.M	310
P.A.B.M	280
C.B.N.M	310
M.B.N.M	350
P.B.N.M	380
P.B.N.N	200
P.B.A.M	263
P.B.B.M	283
C.BL.N.M	313
M.BL.N.M	400
P.BL.N.M	423
P.BL.N.N	223
P.BL.A.M	333
P.BL.B.M	337

Restrained Shrinkage Rings

Results from ASTM C1581 cracking potential tests are shown in Table 3.12. Three specimen were tested for each mixture, with four strain gages mounted on each testing apparatus. Mixtures included in this study generally showed a high resistance to cracking. A number of the mixtures tested early in the project were allowed to remain on the testing apparatus up to 56 days, several specimens not exhibiting a crack during this time period (denoted by N.C. in Table 3.12). After observing this through two sets of rings, the team became concerned about completing all tests during the project timeframe, and shortened the duration of tests to approximately 1 month (recommended by the ASTM C1581 standard. Although many rings did not crack during the test, several rings showed relatively early cracking (less than 14 days). The low-slump paving mixtures proved challenging to consolidate into the narrow form/inner ring used for the ASTM C1581 test apparatus. It is suspected that specimens with very early age cracking may have experienced cracking in isolated areas of lesser consolidated concrete. One set of data was deemed unreliable due to an error in the data acquisition system. These mixtures (P.A.B.M, P.B.B.M, and P.BL.B.M, shown in gray in Table 3.12) are mixtures where fly ash B was used with the Piedmont coarse aggregate. Since the "sister" mixtures for this group – fly ash A with the Piedmont coarse aggregate (P.A.A.M, P.B.A.M., P.BL.A.M) showed reliable results, loss of this data was somewhat mitigated. Supporting data, including the rate of strain per day and stress rate, is provided in Appendix B in Table B.20. Although this test did not reveal significant trends, it could be useful as NCDOT considers provisions for performance engineered mixtures in the future.

Table 3.12: ASTM C1581 cracking potential test results

Mixture ID	Time to Cracking (days)				Duration of Test (days)
	Specimen			Average	
	1	2	3		
C.A.N.M	11.25	N.C.	N.C.	N/A	34
M.A.N.M	13.75	13.75	N.C.	N/A	56
P.A.N.M	9.5	N.C.	N.C.	N/A	32
P.A.N.N	24.25	22.25	N.C.	N/A	31
P.A.A.M	18.75	18.75	N.C.	N/A	56
P.A.B.M	**	**	**	-	-
C.B.N.M	7.25	14.75	N.C.	N/A	34
M.B.N.M	N.C.	N.C.	6	N/A	56
P.B.N.M	N.C.	N.C.	N.C.	N.C.	32
P.B.N.N	20.25	22.5	16	19.5	31
P.B.A.M	N.C.	N.C.	20	N/A	56
P.B.B.M	**	**	**	-	-
C.BL.N.M	14.75	7.5	N.C.	N/A	34
M.BL.N.M	N.C.	N.C.	N.C.	N.C.	56
P.BL.N.M	N.C.	N.C.	N.C.	N.C.	32
P.BL.N.N	22.25	29.25	19.5	23.75	31
P.BL.A.M	18.25	28.25	25	23.75	56
P.BL.B.M	**	**	**	-	-

Key
Early crack (<=14 days)
Late crack (14-28 days)
N.C. = no crack observed at 28 days
** = Data acquisition error

Thaumasite Attack

The susceptibility of PLC concrete to damage due to formation of thaumasite (similar to sulfate attack) has been documented in the literature, with evidence indicating an enhanced susceptibility to this attack in cold weather. Tests to determine the susceptibility of the PLC concrete mixtures to the conventional form of sulfate attack and to thaumasite attack were performed on mortar bars of size 1" by 1" by 11", exposed to a sulfate solution and stored at two different temperatures (5°C and 23°C). Per the CSA 3004-C8 standard, six specimens for each mixture were prepared, of which three were stored at 23°C (Procedure A, assessing resistance to conventional sulfate attack in a manner similar to ASTM C1012), and three were stored at 5°C (Procedure B, assessing the resistance to the potential for the thaumasite form of sulfate attack). Length measurements of the bars were taken after specified durations of soaking in the sulfate solution. The percent change of length was computed for each bar, and the average percent length change of three bars was determined. The average percent change in length for bars stored at 23°C is shown in Table 3.13, and the average percent change in length for bars stored at 5°C is shown in Table 3.6. For this test, mixture IDs are slightly different than those used in other tests, since this test is performed on mortar specimens (no coarse aggregate). In both tables, the first column provides the Mixture ID. The first letter represents type of cement (A-OPC 1, B-OPC 2, and BL-PLC), the second letter represents the type of fly ash (N-no fly ash, A- fly ash A, and B-fly ash B), and in case of natural sand mixtures, the third letter N denotes natural sand. Supporting test data is provided in Appendix B, Tables B.21 and B.22.

Overall, minimal differences in performance were observed between the PLC and OPC concretes and mortars were observed. From Figure 3.13, at 26 weeks all mixtures passed the CSA A3004-08 criteria to be classified as moderate sulfate resisting (less than <0.10% expansion at 6 months when stored at 23 °C). Most of the mixtures passed the CSA A3004-08 requirement for high sulfate resistance (<0.05% expansion at 6 months when stored at 23 °C).

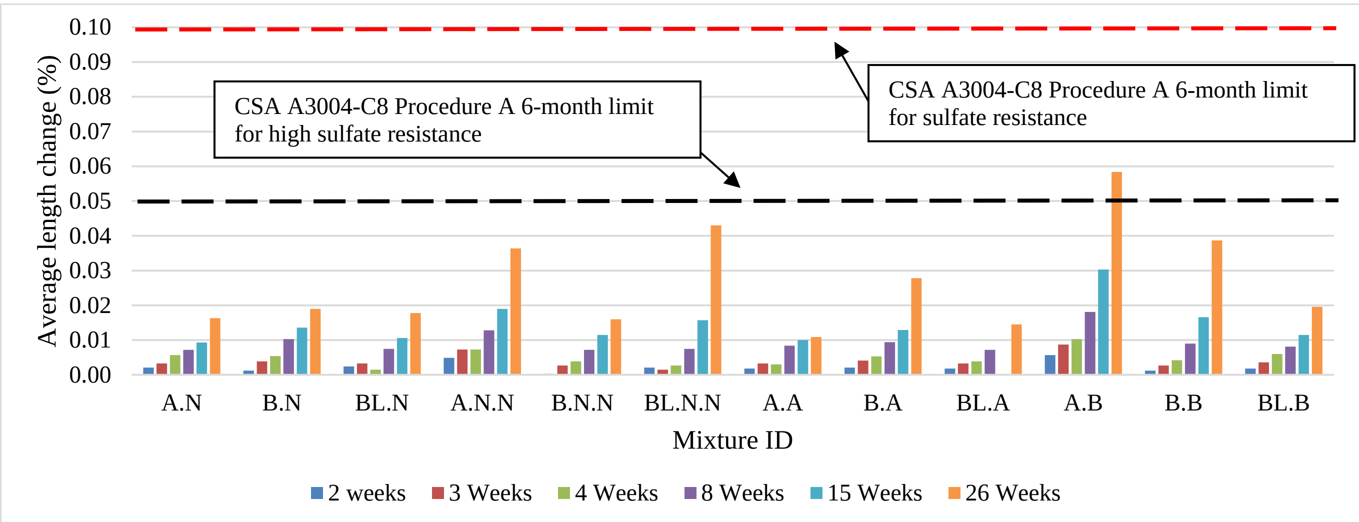

Figure 3.6: Conventional sulfate attack test results (CSA A3004-C8 Procedure A, specimens stored at 23°C)

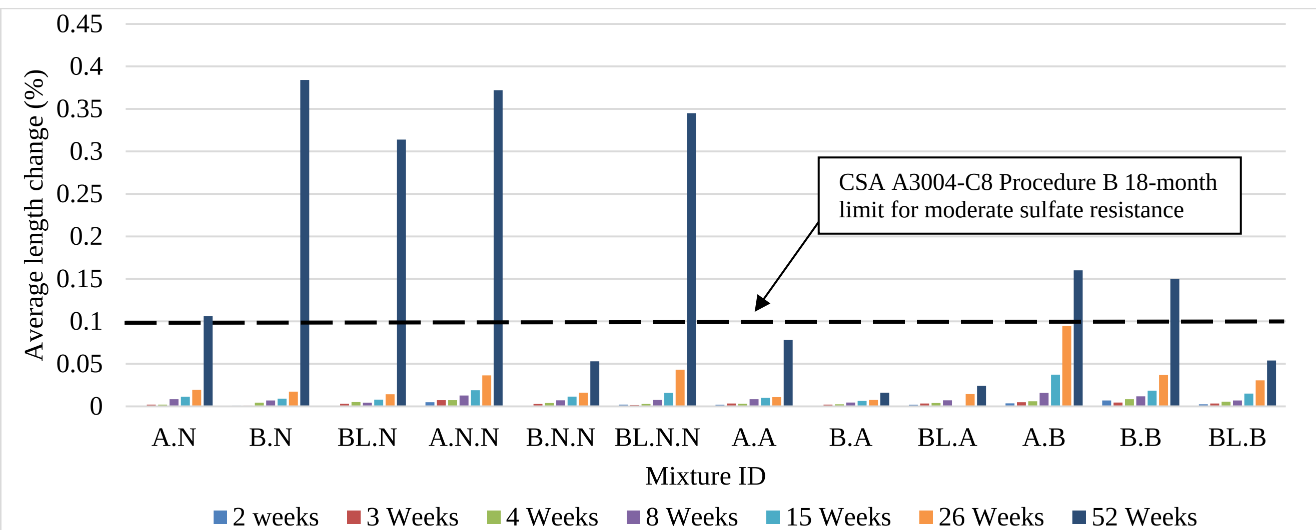

Figure 3.7: Thaumasite form of sulfate attack (CSA A3004-C8 Procedure B, specimens stored at 5°C)

As described previously, both OPC mixtures are Type I/II cements, and the PLC (Cement BL) used for this study was produced by intergrinding limestone with one of the OPC Type I/II cements (Cement B). Test results up to 12 months of age are presented in Figure 3.7. From Figure 3.7, it can be observed that although a number of mixtures exceeded the

CSA A3004-C8, Procedure A requirements for moderate sulfate resistance (<0.10% at 18 months at when stored at 5^{o}C) at 12 months of age, high average length changes do not appear to be specifically associated with the PLC mixtures. Mixtures utilizing fly ash generally performed better in the Procedure B test, regardless of cement type. This is consistent with findings of other researchers who recommend use of SCMs with PLC in sulfate environments to increase resistance to thaumasite attack (Ramezanianpour and Hooton 2013), and more recent findings indicating that the CSA A3004-08 Procedure B test is too severe due to the potential for some PLC and SCM combinations to fail the test despite having a history of adequate field performance (Barcelo et al. 2014). It is noted that Hooton and Thomas (2002) studied the susceptibility of Canadian PLC concrete (much of which is in service in colder climates) to thaumasite and found that there were no cases of thaumasite attack related to use of PLC in concrete in Canada. Other criticisms of the CSA 3004-08 procedure B test include the "low maturity of specimens when they are put in the solutions at 5°C," which slows hydration rates, particularly for SCMs (Barcello et al. 2014). Based on these critiques, these leading researchers in this area, "recommend modifying CSA A3004-08 procedure B to ensure sufficient hydration maturity before the samples are subjected to sulfate attack at low temperatures (Barcello et al. 2014)." At this time, leading researchers in this area continue to investigate modifications to the CSA 3004-08 Procedure B test.

Specifications for two state (Colorado and Utah) agencies allowing PLC also require inclusion of SCMs in concrete mixtures used in applications that could be susceptible to alkali-silica reactivity (ASR) and/or sulfate attack (Laker and Smartz 2012). This approach, requiring use of SCMs in all concrete mixtures (both PLC and OPC) used in applications that could be susceptible to ASR and/or sulfate attack, is recommended for use by NCDOT.

3.4 Summary of Laboratory Findings

Based upon the results presented in previous sections of this report, laboratory findings of this project are:

Mechanical properties

- The rate of strength gain for mixtures containing PLC appears to be reasonably similar to the corresponding Type I/II cement.
- Mixtures containing fly ash typically had compressive strengths that were notably lower than the corresponding non-fly ash mixture. However, 28-day modulus of rupture test results for the fly ash mixtures were relatively similar to those of the non-fly ash mixtures.
- Modulus of elasticity values (at 28-days) for all eighteen mixtures were within the range of 2,400,000 psi to 3,700,000 psi. This is slightly lower than the suggested range of 3,000,000 psi to 4,000,000 psi suggested in the MEPDG literature. The mixtures containing the coastal coarse aggregate tended to exhibit higher moduli of elasticity than mixtures with coarse aggregates from the other two regions.
- Many of the mixtures exhibited Poisson's ratio test results that were higher than the suggested range provided in the MEPDG literature (0.15 to 0.18).

Thermal properties

- Use of the manufactured sand appears to have a significant effect on the coefficient of thermal expansion and thermal conductivity of the concrete mixtures. The effect of sand type on heat capacity is not readily evident.
- Mixtures using the Piedmont and Mountain coarse aggregates had coefficient of thermal expansion values significantly lower than the recommended value for granite aggregates suggested in the MEPDG literature.
- Mixtures using the Coastal coarse aggregate had coefficient of thermal expansion values lower than the recommended value for limestone aggregates suggested in the MEPDG literature.
- Mixtures containing the natural sand had a notably higher coefficient of thermal expansion than those containing the manufactured sand.
- For all mixtures, measured values for heat capacity were notably lower than the default values suggested in the MEPDG literature.
- Regardless of sand type, coarse aggregate type, or cementitious materials utilized, an MEPDG input for heat capacity of 0.20 BTU/lb·ft appears to be reasonable. The default value is 0.28 BTU/lb·ft.

- For mixtures with the manufactured sand, regardless of coarse aggregate source or cementitious materials utilized, an MEPDG input for thermal conductivity of 0.80 to 0.90 BTU/(ft·hr·°F) appears to be reasonable. The default input value is 1.25 BTU/(ft·hr·°F).
- Mixtures with the natural sand had a higher thermal conductivity, closer to the default value of 1.25 BTU/(ft·hr·°F).

Durability performance

- Mixtures containing fly ash exhibited a significant reduction in later-age (90-day) chloride permeability.
- Mixtures containing both fly ash and PLC showed a noticeable decrease in permeability from the fly ash only (no PLC) mixtures.
- Because of the range of permeability characteristics observed across the tested mixtures, project-based permeability targets should be used to design and verify the performance of mixtures.
- Some PLC mixtures exhibited slightly greater shrinkage than cement B mixtures, however shrinkage performance of mixtures with all cements was almost always within acceptable performance limits identified in the literature.
- For the ASTM C157 shrinkage test, the 28-day microstrain for the 18 pavement mixtures tested range from 200 to 423 microstrain. All but one mixture passed the 28-day recommended prescriptive limit in AASHTO PP 84-17.
- In freeze-thaw testing, all mixtures included in this study showed acceptable performance, with durability factors at 300 cycles greater than 70.
- After freeze-thaw testing for 300 cycles, mixtures utilizing natural sand had durability factors significantly lower than mixtures containing manufactured sand.
- Mixtures that included the Mountain coarse aggregate tended to have the lowest durability factors, between 77 and 80.
- Overall, minimal differences in performance were observed between the PLC and OPC concretes and mortars were observed in CSA 3004-C8 sulfate attack testing (both Procedure A and Procedure B). For the Procedure A test, all mixtures passed the CSA A3004-08 6-month criteria to be classified as moderate sulfate resisting.
- Although at 12 months of age a number of mixtures exceeded the CSA A3004-C8 Procedure B requirements for moderate sulfate resistance (prescribed at 18 months), high average length changes do not appear to be specifically associated with the PLC mixtures. Mixtures containing fly ash generally performed better in the Procedure B test, regardless of cement type, consistent with findings of other researchers who recommend use of SCMs with PLC in sulfate environments to increase resistance to thaumasite attack (Ramezanianpour and Hooton 2013). Many leading researchers find this CSA A3004-C8 Procedure B test to be “overly severe” and recommend modifying the test (Barcelo et al. 2014).

4. EVALUATION OF RESULTS

4.1 Evaluation of New Concrete Inputs for MEPDG Design

The catalog of recommended inputs for PCC in the AASHTOWare Pavement ME software differ from some inputs currently utilized by NCDOT. 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 AASHTOWare Pavement ME Design software. Version 2.1 of the software, current at the time of this study, was utilized. Results of this effort are detailed in Section 4.1.1. Additionally, new PCC inputs were utilized to re-analyze the potential performance of four rigid pavements recently designed by NCDOT using AASHTOWare. The impact of the new inputs on predicted performance, along with potential changes to PCC thickness, were assessed. The results of this effort are detailed in Section 4.1.2.

4.1.1 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 central North Carolina (Greensboro, NC) was utilized for all analysis. A one-at-a-time (OAT) analysis, for each of the concrete input values was performed. As the inputs were varied across the desired range, the predicted distresses for each pavement section were compared. This approach facilitated evaluation of the impact that each of the different concrete mixture inputs has on the predicted concrete pavement performance. This approach also allowed identification of the concrete materials inputs that are most sensitive for North Carolina MEPDG pavement design. 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 4.1 and Table 4.2.

Table 4.1: 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 4.2: 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

The specific methods and full range of input values are available in a MS thesis prepared as part of this project (Blanchard 2016). OAT analysis for each of the concrete input values was performed. 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 (Blanchard 2016). Based on laboratory testing for the seven concrete inputs, values range as follows:

- unit weight ranges from 138 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)

The OAT sensitivity analysis performed using the batch processing capabilities of the AASHTOWare Pavement ME software, with a total of 35 pavement performance simulations performed for this work – seven variables of interest, each tested at five levels within the range (lowest range value, 25th percentile of the range, 50th percentile of range, 75th percentile of the range, highest range value), as shown in Table 4.3.

Table 4.3: Inputs 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

The impact of adjusting each input parameter value was used to rate the sensitivity as "Very Sensitive," "Sensitive," or "Neutral" as shown in Table 4.4 along with corresponding ranges. 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 4.4 provides the coding for relative degree of sensitivity as "Very Sensitive" (VS), "Sensitive" (S), and "Neutral" (N), used in the overview of sensitivity analysis results provided in Table 4.5.

Table 4.4. Sensitivity analysis descriptions and corresponding ranges

	Average Change in Distress		
	Terminal IRI, in/mile (mm/km)	Mean Joint Faulting, in (mm)	Transverse Cracking (% slabs cracked)
Very Sensitive (VS)	3.0 and greater (47.3 and greater)	0.01 and greater (0.25 and greater)	1.0 and greater
Sensitive (S)	2.99 to 1.00 (47.2 to 15.8)	0.009 to 0.001 (0.24 to 0.025)	0.99 to 0.10
Neutral (N)	0.99 to 0.00 (15.7 to 0)	0.00 (0.00)	0.09 to 0.00

Table 4.5. Summary of sensitivity analysis results, providing the effect on predicted distress by increasing each input value

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

Note: VS = Very Sensitive, S = Sensitive, N = Neutral

Results presented in Table 4.5 provide an indication of the effects of predicted distress of increasing each input value. 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. Detailed analysis, including plots of the results for the sensitivity of Terminal IRI, mean joint faulting, and transverse cracking to each input is presented in Blanchard (2016) and Appendix C of this report. Key findings, particularly with regards to concrete thermal inputs, are summarized herein.

Consistent with the findings of other researchers, CTE was determined to be a "Very Sensitive" input for North Carolina concrete pavements for all modes of predicted distress. The plotted result for sensitivity of predicted joint faulting, slab cracking, and IRI to CTE are shown in Figures 4.1 through 4.3.

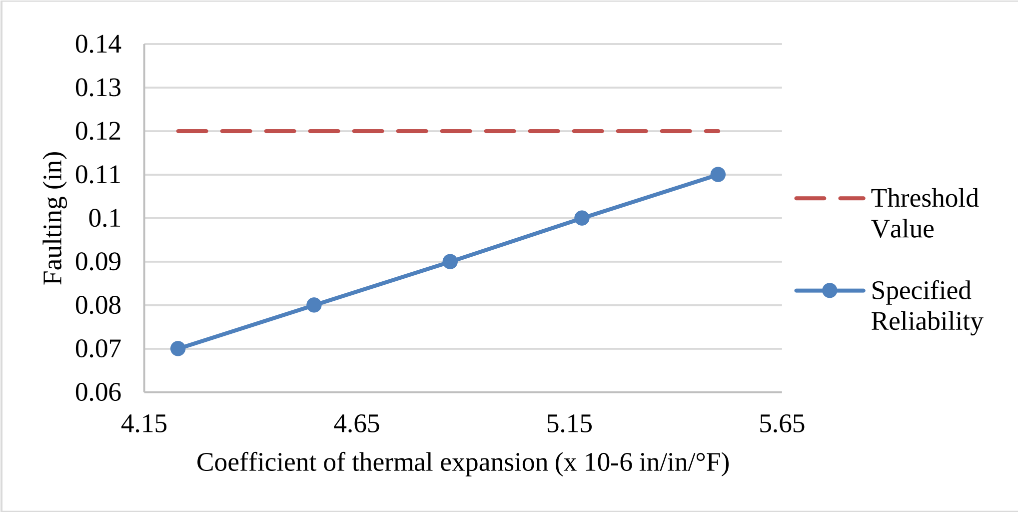

Figure 4.1: Sensitivity of predicted joint faulting to changes in concrete CTE

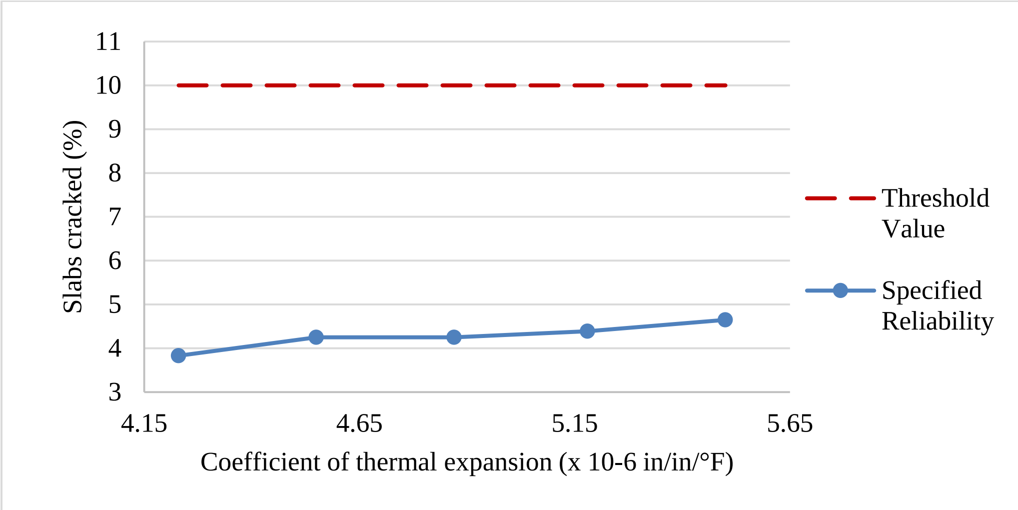

Figure 4.2: Sensitivity of predicted transverse cracking to changes in concrete CTE

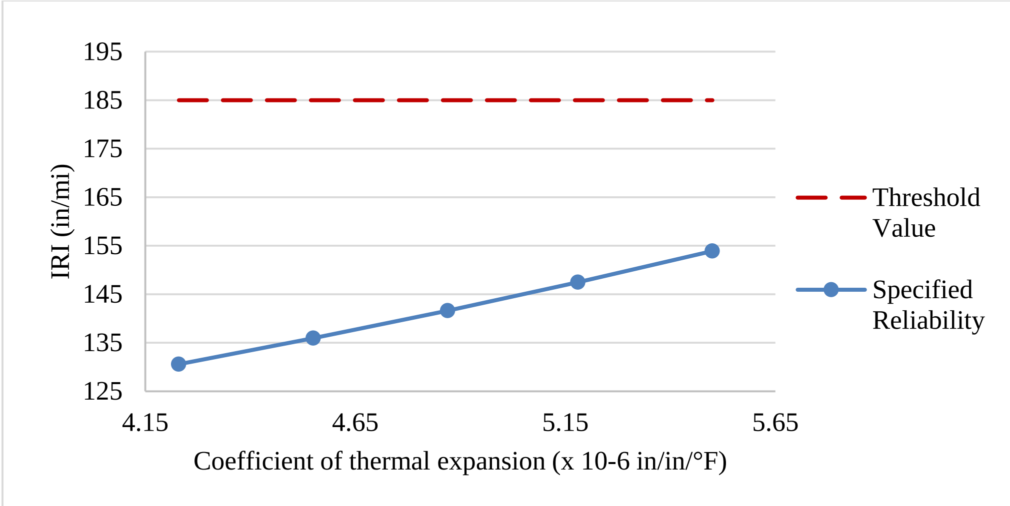

Figure 4.3: Sensitivity of predicted IRI to changes in concrete CTE

Transverse cracking was found to be "Very Sensitive" and mean joint faulting was found to be "Sensitive" to thermal conductivity (shown in Figures 4.4 and 4.5, respectively). However, terminal IRI does not appear to have much sensitivity to thermal conductivity ("Neutral"), as shown in Figure 4.6. It was 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 slightly decreases, indicating an optimum range of inputs for optimum predicted performance exists for this input.

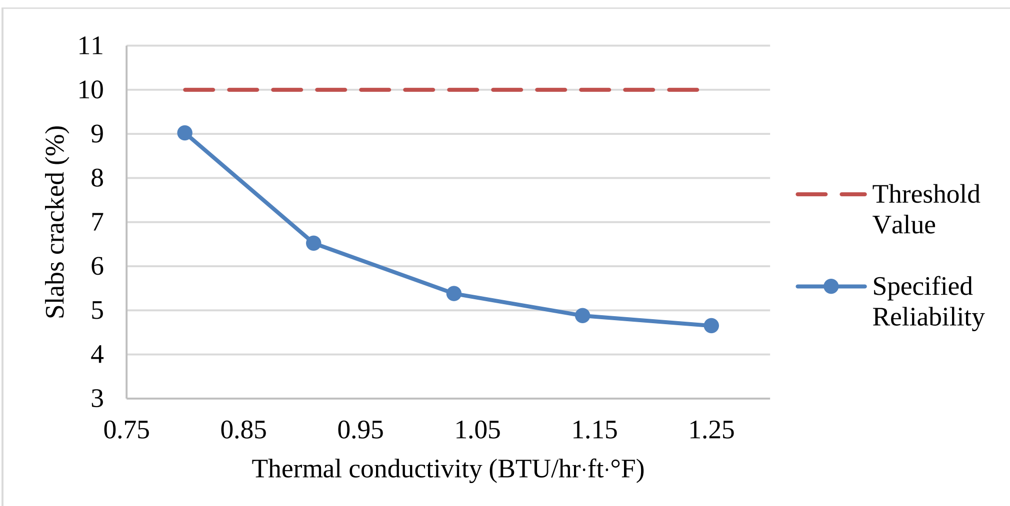

Figure 4.4: Sensitivity of predicted slab cracking to changes in concrete thermal conductivity

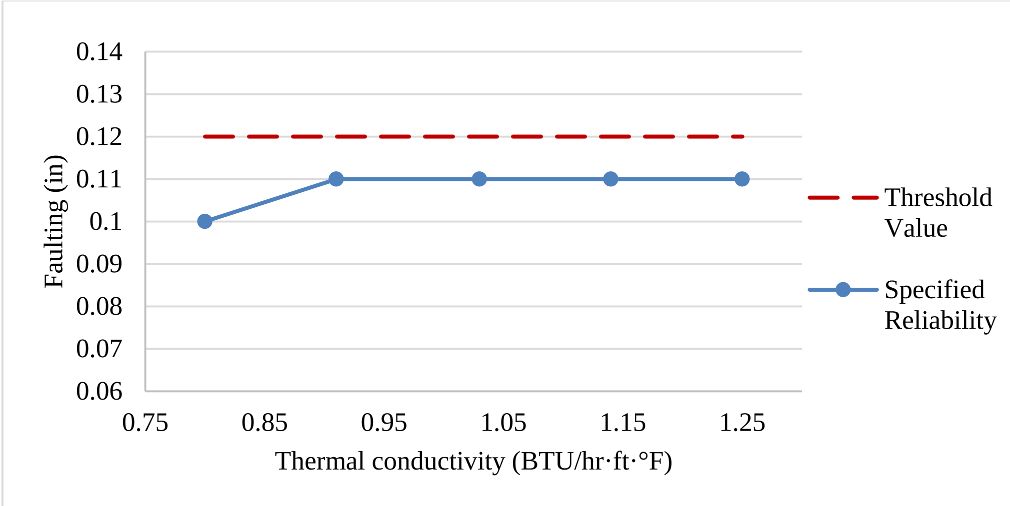

Figure 4.5: Sensitivity of predicted joint faulting to changes in concrete thermal conductivity

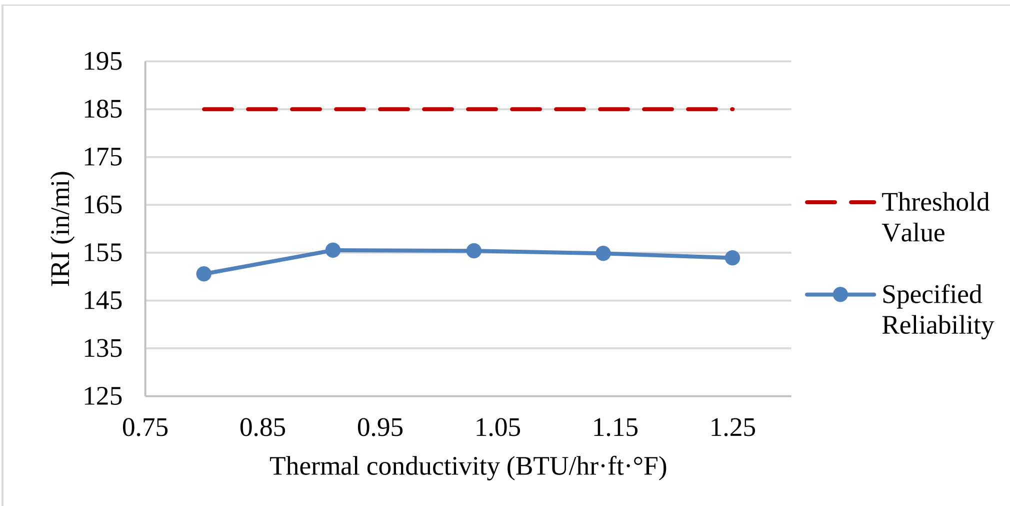

Figure 4.6: Sensitivity of predicted IRI to changes in concrete thermal conductivity

Similar to the findings of other researchers, terminal IRI and mean joint faulting do not appear to have much sensitivity to heat capacity ("Neutral"), shown in Figure 4.7 and Figure 4.8, respectively. However, transverse cracking was found to be "Sensitive" to heat capacity, shown in Figure 4.9. 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. Also of interest, Terminal IRI and mean joint faulting were found to be "Very Sensitive" and "Sensitive" to unit weight, respectively.

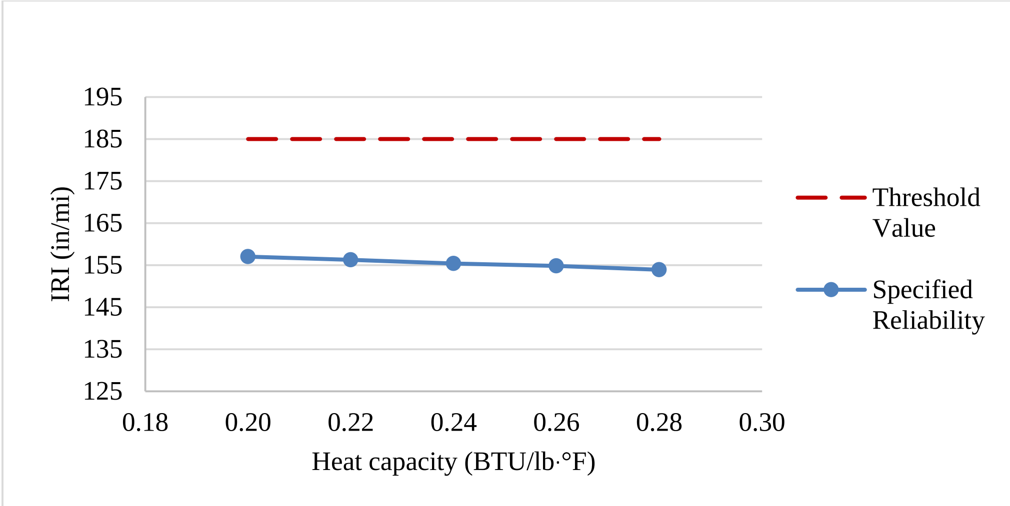

Figure 4.7: Sensitivity of predicted IRI to changes in concrete heat capacity

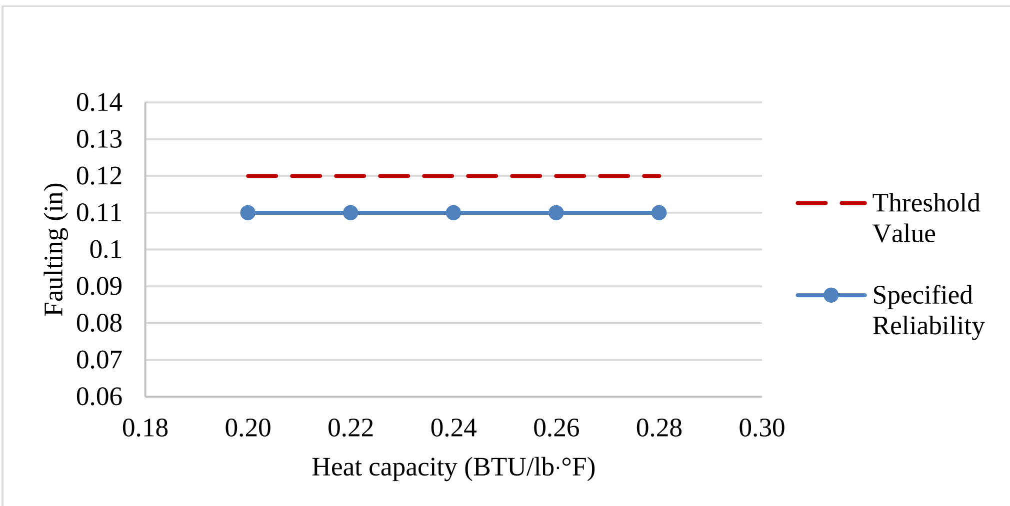

Figure 4.8: Sensitivity of predicted joint faulting to changes in concrete heat capacity

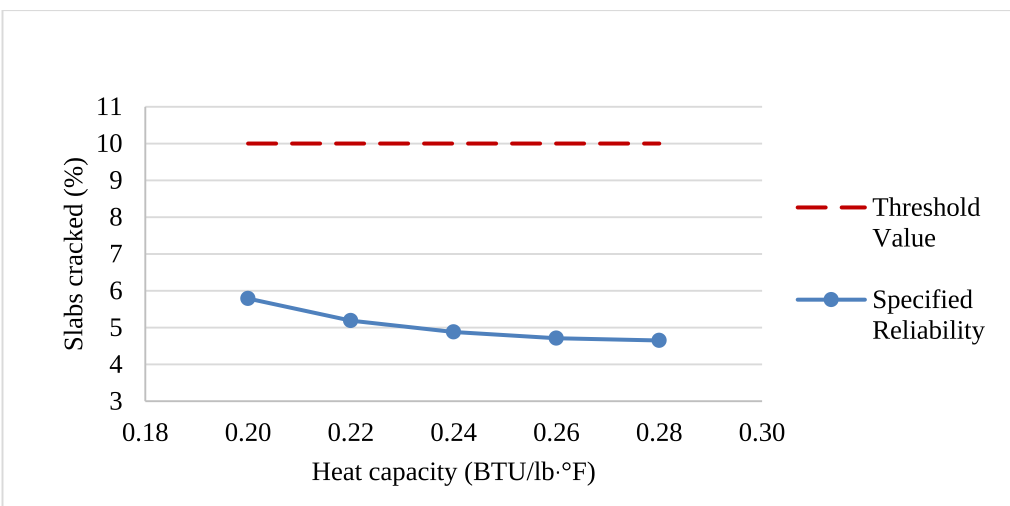

Figure 4.9: Sensitivity of predicted transverse cracking to changes in concrete heat capacity

4.1.2 Evaluation of Potential Impact on Pavement Design Thickness

Four recent rigid pavement projects selected as representative examples of roadways recently designed for construction in several regions of North Carolina (summarized in Table 4.6), were used for analysis to determine the impact of the new inputs on typical rigid pavement design and analysis. For most of these designs, NCDOT used target values of 185.00 in/mile for terminal IRI, 0.12 in for mean joint faulting, and 10.00% for JPCP transverse cracking. It is noted that for the rural freeway project, a target value of 172.00 in/mile for terminal IRI, 0.12 in for mean joint faulting, and 15.00% for JPCP transverse cracking was utilized. Reliability for each of these criteria was set at 90%. A map of showing the location of these projects is provided in Appendix C, Figure C.22.

Table 4.6: Selected typical projects analyzed for impact of new M-EPDG concrete inputs

Project ID	Project Type	Region	NC County	Initial Two-Way AADTT	Number of lanes in each direction	PCC thickness, in
I-4400	Interstate	Mountain	Buncomb	13,400	4	10.5
U-2579	Urban freeway	Piedmont	Forsythe	11,064	3	11
R-2536	Rural freeway	Piedmont	Randolph	1,573	2	9.5
U-2519	Urban freeway	Coastal & Piedmont	Cumberland	4,550	2	10

4.1.2.1 Predicted Performance Using New Inputs

For each of the four selected pavement projects, the analyses were re-run using the new laboratory-obtained input values associated with the coarse aggregates local to that region, along with the appropriate laboratory-obtained input values obtained for both OPCs and the PLC. These results were compared to the performance predictions of the original design, which did not utilize locally-calibrated inputs. Inputs and analysis results for the Interstate project in the Mountain region (Project I-4400) are presented in detail in Table 4.7, along with a description of results. A similar approach was utilized for the three other selected projects, and for brevity, a summary of results of the analysis for three additional sections presented in Table 4.8. The results of the analysis for Project U-2579 (urban freeway in the Piedmont region), Project R-2536 (rural freeway in the Piedmont region), and U-2519 (urban freeway in the area that could be considered either Coastal or Piedmont region) are presented in Appendix C, Tables C.1 through C.5.

For the Interstate project in the Mountain region, the as-designed inputs and performance predictions are summarized in the left-most column of data in Table 4.7. The analysis was re-run using concrete inputs for mixtures with locally available coarse aggregates, along with OPC and PLC from both sources (available in this region), as shown in the three right-most columns of Table 4.7. This pavement was 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 (Cement BL) were similar to those of the OPC (Cements A and B). As can be seen in Table 4.7, the predicted performance of the pavement designed with both sets of inputs (Cements 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.

Table 4.7: Analysis of interstate project using previous inputs and new inputs for concrete in the Mountain region

			Interstate Project Mountain Region As-Designed	Manufactured Sand with Cement A	Manufactured Sand with Cement B	Manufactured Sand with Cement BL
Layer 1: PCC	Pavement thickness, in		10.5	10.5	10.5	10.5
	Cementitious material content, pcy		600	550	550	550
	Water to cementitious material ratio		0.42	0.48	0.48	0.48
	Unit weight, pcf		150	145	145	145
	28-day compressive strength, psi		Not used	5030	5100	4790
	28-day modulus of rupture, psi		650	570	641	606
	28-day modulus of elasticity, psi		4,200,000	2,540,000	2,760,000	3,020,000
	Poisson's ratio		0.17	0.18	0.20	0.20
	Coefficient of thermal expansion, x 10^{-6} in/(in°F)		6.0	4.46	4.46	4.56
	Heat capacity, BTU/(lb·°F)		0.28	0.20	0.21	0.20
	Thermal conductivity, BTU/(ft·hr·°F)		1.25	0.87	0.95	0.91
Layer 2:			4 in of flexible pavement			
Layer 3:			8 in of lime stabilized base			
Layer 4:			12 in of A-5 subgrade			
Layer 5:			Semi-infinite layer of A-5 subgrade			
Climate data			Asheville, NC			
Distress	Terminal IRI, in/mile	Target: 185	162.5	142.8	141.2	146.0
	Mean joint faulting, in	Target: 0.12	0.11	0.07	0.08	0.08
	JPCP transverse cracking (% slabs)	Target: 10.00	8.59	4.25	3.83	4.25
Reliability	Terminal IRI, %		96.88	99.42	99.51	99.18
	Mean joint faulting, %		94.30	99.78	99.69	99.40
	JPCP transverse cracking, %		93.88	99.87	99.96	99.87

Similar findings were made using the new inputs for the three Piedmont and Coastal region projects, as shown by the analysis results (predicted distresses and reliabilities) summarized in Table 4.8. For each of the three projects, use of new, locally calibrated input values resulted in prediction of reduced roughness, reduced joint faulting, and a decrease in slab cracking. Use of concrete manufactured sand is predicted to provide improved performance over sections constructed of concrete using natural sand. Predicted performance of the pavements using PLC is equivalent (or potentially improved) over that of the sections using OPC.

Table 4.8: Summary of analysis of projects using previous inputs and new concrete inputs

				Project As-Designed	Manufactured Sand with Cement B	Manufactured Sand with Cement BL	Natural Sand with Cement B	Natural Sand with Cement BL
Urban Freeway - Piedmont Region	PCC Thickness, in			11	11	11	11	11
	Distress	Terminal IRI, in/mile	Target: 185	131.9	117.8	112.1	126.7	121.8
		Mean joint faulting, in	Target: 0.12	0.08	0.06	0.05	0.07	0.07
		JPCP transverse cracking (% slabs)	Target: 10.00	4.39	3.83	3.83	3.83	3.83
	Reliability	Terminal IRI, %		99.83	99.99	100.00	99.93	99.97
		Mean joint faulting, %		99.34	99.98	100.00	99.76	99.92
		JPCP transverse cracking, %		99.83	99.96	99.96	99.96	99.96
Rural Freeway - Piedmont Region	PCC Thickness, in			9.5	9.5	9.5	9.5	9.5
	Distress	Terminal IRI, in/mile	Target: 172	136.1	125.9	120.0	133.5	129.0
		Mean joint faulting, in)	Target: 0.12	0.07	0.06	0.05	0.07	0.07
		JPCP transverse cracking (% slabs)	Target: 15.00	7.98	4.65	3.83	4.49	3.83
	Reliability	Terminal IRI, %		99.13	99.76	99.91	99.35	99.62
		Mean joint faulting, %		99.81	99.98	100.00	99.80	99.92
		JPCP transverse cracking, %		99.59	100.00	100.00	100.00	100.00
Urban Freeway – Coastal / Piedmont Region	PCC Thickness, in			10	10	10	10	10
	Distress	Terminal IRI, in/mile	Target: 185	144.7	129.4	123.2	139.8	134.1
		Mean joint faulting, in	Target: 0.12	0.11	0.07	0.06	0.09	0.08
		JPCP transverse cracking (% slabs)	Target: 10.00	4.25	3.83	3.83	3.83	3.83
	Reliability	Terminal IRI, %		99.24	99.89	99.96	99.56	99.78
		Mean joint faulting, %		97.31	99.82	99.97	98.60	99.40
		JPCP transverse cracking, %		99.87	99.96	99.96	99.96	99.96

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

Using the four NCDOT projects previously described and shown in Table 4.6, along with the catalog of suggested concrete inputs presented in Table 3.4, analyses with AASHTOWare Pavement ME software were performed to identify the reduction in thickness of PCC that could be obtained with the new recommended inputs. The original design was re-run using the new concrete inputs, then run at successively reduced PCC thickness in ½ inch increments until target distress criteria were no longer met. For each of the following analyses, the size of the dowel bars was modified per NCDOT specifications (NCDOT 2012b), which specifies an increase in dowel bar diameter with increased slab thickness (summarized in Table 4.9). Therefore, as the slab section was reduced to evaluate performance, the appropriate dowel bar diameter was included in the analysis. 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 4.9: Table I – Dowel bars, as found in NCDOT standard drawing 700.01 (NCDOT 2012b)

Slab thickness (inches)	Dowel bar diameter, D (in)	Dowel bar length, L (in)
8 or less	1	14
8½ to 9½	1-1/8	16
10 to 10½	1¼	18
11 or greater	1½	18

Inputs and analysis results for the urban freeway project in the Piedmont region are presented in detail in Table 4.10, along with a description of results. A similar approach was utilized for the three other selected projects, and for brevity, a summary of results of the analysis for three additional sections presented in Table 4.11. For the urban freeway in the Piedmont region, inputs and predicted performance of the original project as-designed are shown in the left-most column of data are shown in Table 4.10. The next column to the right shows the inputs and analysis results using the same design parameters as the original project, with the new recommended Piedmont coarse aggregate concrete inputs with manufactured sand. The two rightmost columns show the results of the analysis using the new concrete inputs with modified concrete pavement thickness in half inch increments (with dowel bar diameters also reduced to match NCDOT specifications) until target distress criteria were no longer met. As can be seen in Table 4.10, the new concrete inputs for Piedmont concrete facilitate sufficient performance of a thinner PCC section (up to 1 in thinner). However, a significant decrease in predicted performance occurs between section thicknesses of 11 in and 10.5 in due to the change in the dowel bar size of 1.5 in to 1.25 in.

Table 4.10: Evaluation of potential PCC thickness reduction of urban freeway using new recommended inputs for Piedmont region concrete

			Urban Freeway Piedmont Region As-Designed	Manufactured Sand 11 in	Manufactured Sand 10.5 in	Manufactured Sand 10 in
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, pcy		600	550	550	550
	Water to cementitious material 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)		6.0	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 in of flexible pavement			
Layer 3:			8 in of lime stabilized base			
Layer 4:			12 in 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	Target: 185	131.9	115.2	143.8	143.0
	Mean joint faulting, in	Target: 0.12	0.08	0.05	0.10	0.10
	JPCP transverse cracking (% slabs)	Target: 10.00	4.39	3.83	3.83	3.83
Reliability	Terminal IRI, %		99.83	99.99	99.31	99.36
	Mean joint faulting, %		99.34	99.99	96.89	97.26
	JPCP transverse cracking, %		99.83	99.96	99.96	99.96

A summary of analysis results for the other three projects is presented in Table 4.11, with the full set of supporting data presented in Appendix B, Tables C.6 through C.11. Analysis of two of the other three projects indicated that the new concrete inputs indicate that these sections could also potentially be reduced up to 1 inch prior to distress criteria targets not being met. However, a significant decrease in predicted performance occurs between section thicknesses corresponding to a change in the dowel bar diameter per NCDOT specifications (2012). This analysis approach on the Interstate - Mountain Region project indicated that despite using the new PCC input values, this project could not be reduced in thickness (due to failing at mean joint faulting predicted performance) while remaining within the target ranges specified in AASHTOWare Pavement ME by NCDOT. This finding, however, is linked to the design practice of reducing dowel diameter at concrete thicknesses less than 11 in, which was verified to be the key driver of reduction in mean joint faulting below the target values.

Table 4.11: Summary of potential thickness reductions of pavements using new concrete inputs

				Project As-Designed	Reduced Thickness Obtained with New PCC Inputs
Interstate - Mountain Region	PCC Thickness, in			10.5	10.5*
	Distress	Terminal IRI, in/mile	Target: 185	162.5	141.6
		Mean joint faulting, in	Target: 0.12	0.11	0.08
		JPCP transverse cracking (percent slabs)	Target: 10.00	8.59	3.83
	Reliability	Terminal IRI, %		99.83	99.49
		Mean joint faulting, %		99.34	99.62
		JPCP transverse cracking, %		99.83	99.96
Rural Freeway - Piedmont Region	PCC Thickness, in			9.5	8.5
	Distress	Terminal IRI, in/mile	Target: 172	136.1	142.8
		Mean joint faulting, in	Target: 0.12	0.07	0.08
		JPCP transverse cracking (percent slabs)	Target: 15.00	7.98	7.35
	Reliability	Terminal IRI, %		99.13	98.35
		Mean joint faulting, %		99.81	99.37
		JPCP transverse cracking, %		99.59	99.77
Urban Freeway - Coastal/Piedmont Region	PCC Thickness, in			10	9
	Distress	Terminal IRI, in/mile	Target: 185	144.7	148.3
		Mean joint faulting, in	Target: 0.12	0.11	0.10
		JPCP transverse cracking (percent slabs)	Target: 10.00	4.25	4.39
	Reliability	Terminal IRI, %		99.24	98.94
		Mean joint faulting, %		97.31	96.58
		JPCP transverse cracking, %		99.87	99.83
* Note: At 10 inch PCC thickness, predicted mean joint faulting exceeds target values due to reduction in dowel diameter from 1.5 inches to 1.125 inches					

4.2 Durability Performance of Mixtures Using Fly Ash and Portland Limestone Cement

Several experiments were undertaken to investigate the potential durability benefits offered by pairing PLC with fly ash in these concrete mixtures. Test results are presented in Section 3 and Appendix C. Additional analysis of these test results was performed, particularly to observe trends in performance associated with use of PLC and fly ash in the mixtures and is presented in this section.

4.2.1 Surface Resistivity Test Results

The surface resistivity results indicate some of the more distinctive differences between the mixtures. General trends show that surface resistivity values are uniformly and significantly higher for the fly ash mixtures and even higher for the fly ash-Portland limestone cement mixtures after 90 days of curing time. Higher values surface resistivity are associated with lower permeability and greater expected durability. These trends are also supported by the literature (Polder 2001).

Figure 4.10 depicts the increase in surface resistivity between the 28th and 90th day of curing. After 28 days of curing time, little difference in surface resistivity is discernable between any of the mixtures, and all mixtures would be classified as "high" permeability concrete. As is apparent in Figure 4.10 the surface resistivity increased slightly between 28 and 90 days of curing time for mixtures that contained portland cement only, or portland cement with interground limestone. However, the mixtures that contained fly ash developed very high resistivity by the 90th day of curing time. The combined impact of including fly ash and interground limestone provided the greatest increase in surface resistivity. Figure 4.11 presents similar data, but the samples included in the chart are limited to those with similar aggregates- piedmont aggregates and manufactured sand and only fly ash from source "A." This subset of samples with reduced variables between mixtures makes it clear that the benefit of including both fly ash and interground limestone is greater than the sum of their individual contributions.

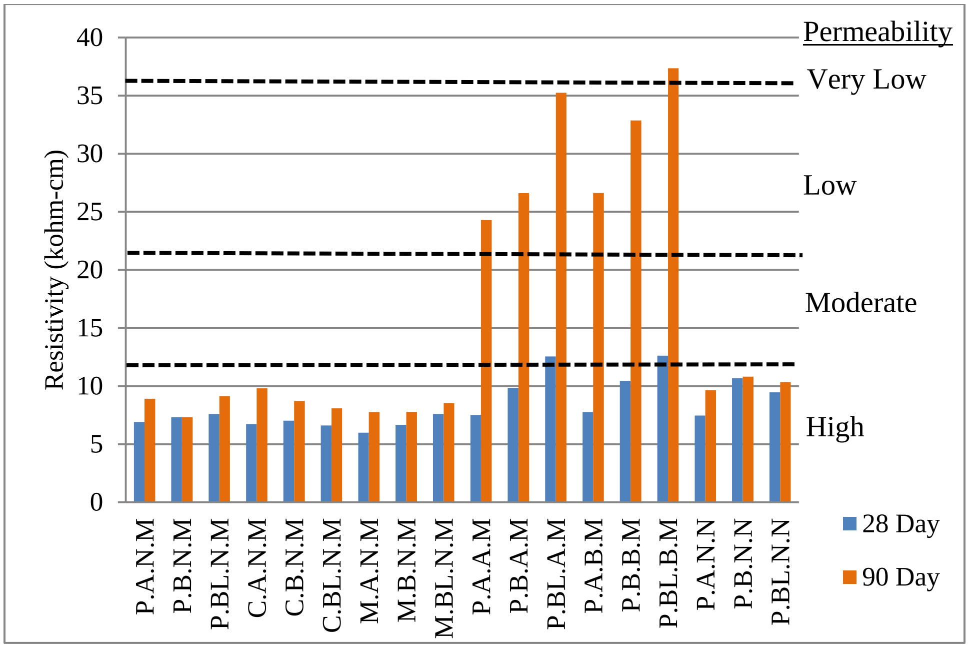

Figure 4.10: 28-day & 90-day surface resistivity results

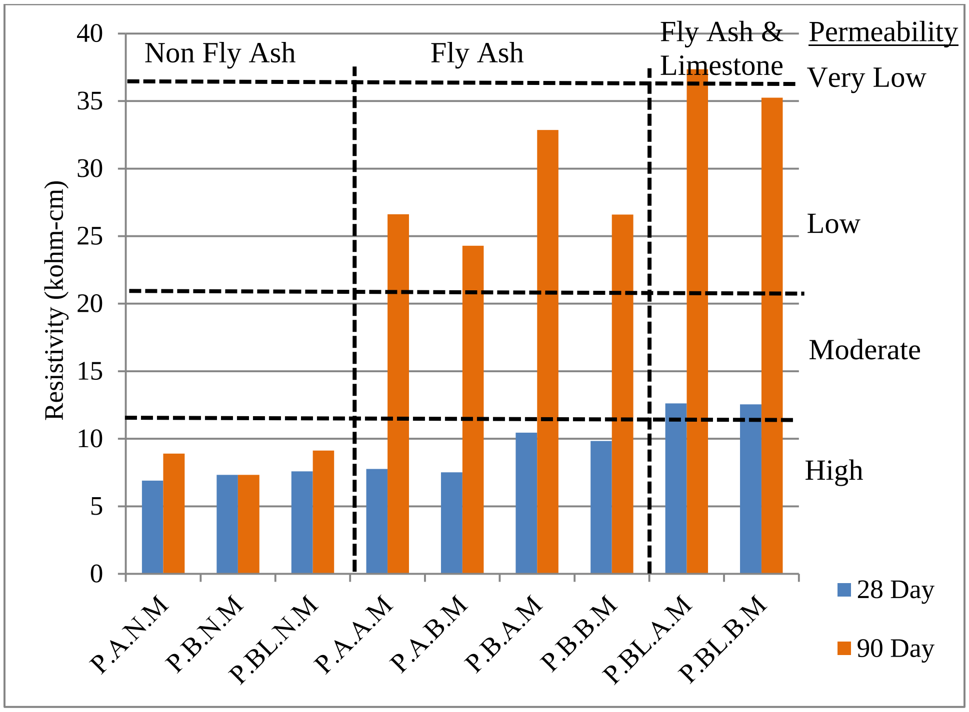

Figure 4.11: Select 28-day & 90-day surface resistivity results

4.2.2 Rapid Chloride Permeability Test Results

The RCPT provides another electrically determined estimate of permeability. In this test, the quantity of charge passed (measured in Coulombs) is correlated with concrete permeability. Lower results are more favorable. In this research two samples for each mixture were tested on the 28th and 90th day of curing. The results were averaged. The results from all mixtures tested are shown in Figure 4.12. In general, (as expected) the charge passed was reduced as the concrete was allowed more curing time. The mixtures containing fly ash passed less charge as compared to the mixtures without fly ash, and the mixtures containing limestone have an even more noticeable reduction in permeability when compared to the mixtures that only contain fly ash. These results were consistent with findings that have been reported in literature (Mohr et al. 2000, Langan and Ward 1990), and are also strongly correlated with the results found by way of the surface resistivity test. Data presented in Figure 4.13 confirms other trends that are similar between the two tests for permeability. The permeability of mixtures containing fly ash decreased substantially between the 28th and 90th day of curing. Mixtures what did not contain fly ash, did not substantially reduce in their permeability as the concrete aged. Finally, as was also seen in the surface resistivity results, the combination of both fly ash and interground limestone (PLC) produced the greatest benefit.

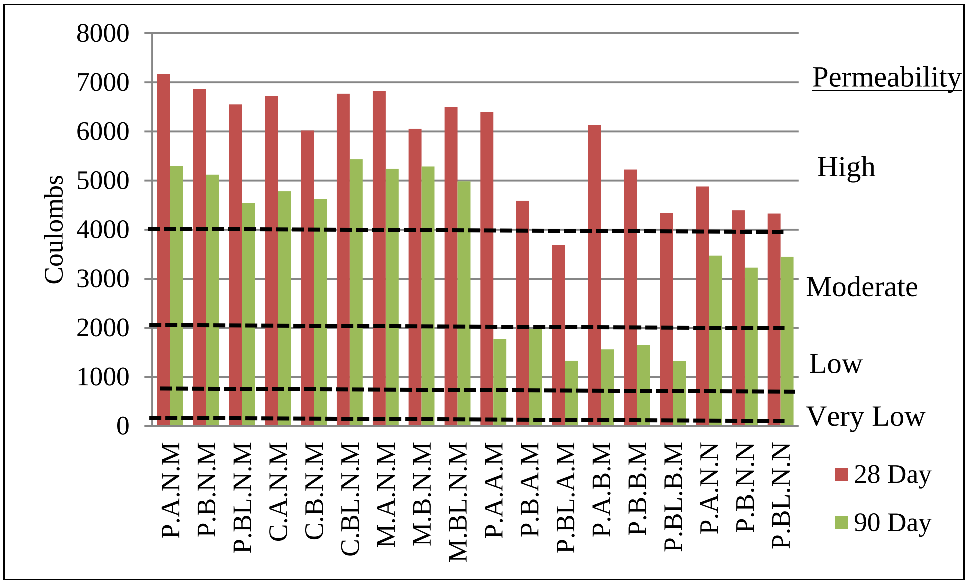

Figure 4.12: RCPT results

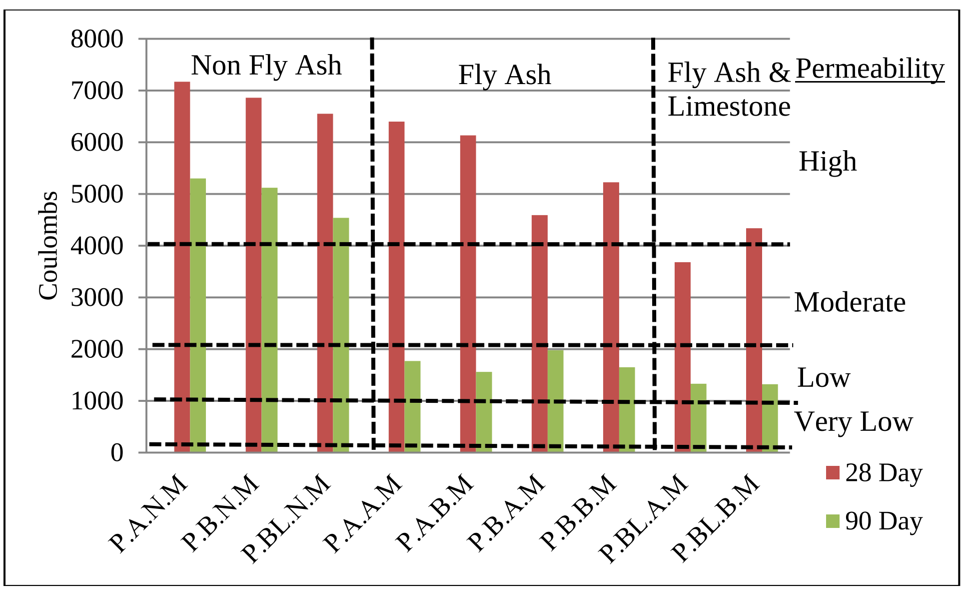

Figure 4.13: Select RCPT results

4.2.3 Correlation Between Surface Resistivity and Rapid Chloride Permeability Test

To assess the potential durability performance of concrete mixtures, a number of industry stakeholders are interested in use of surface resistivity test measurements in lieu of the more costly and time-consuming RCPT. Morriset al. (1996) and Rupnow and Icenogle (2011) have previously shown the tests to produce strongly correlatable results. Results presented in this report confirm a strong correlation. Figure 4.14 presents the relationship between results of the two tests after 28 days of curing. Because of the scaling of the units used for each result, the relationship between the two is exponential, represented by the formula of the best fit line displayed on the plot. The relationship between the data and the best fit line formula has R^2=.79, which indicates very good correlation, especially given the large number of variables in the mix design. The correlation became stronger at 90 days of curing (Figure 4.15) with R^2=0.94, which indicates excellent correlation.

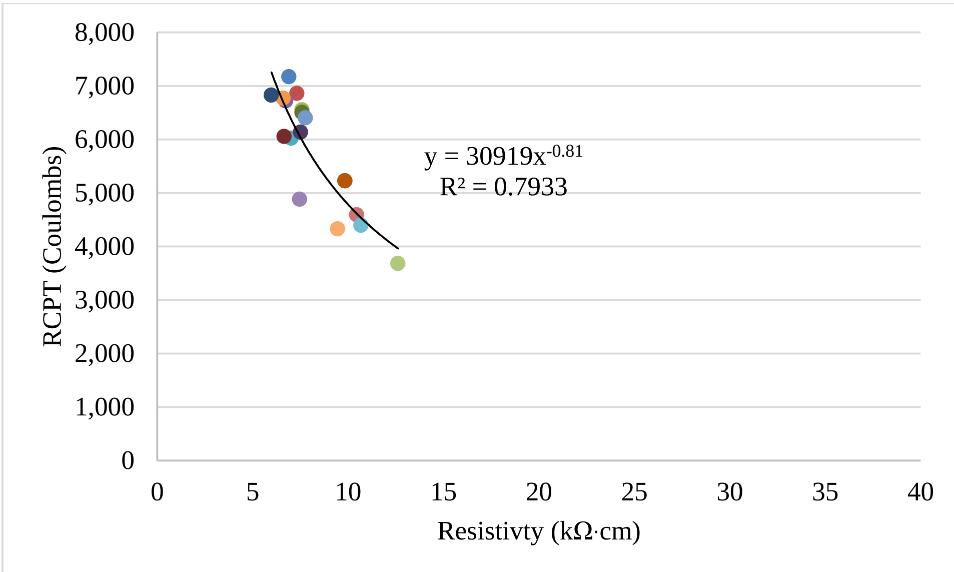

Figure 4.14: 28-day test results - RCPT vs. surface resistivity

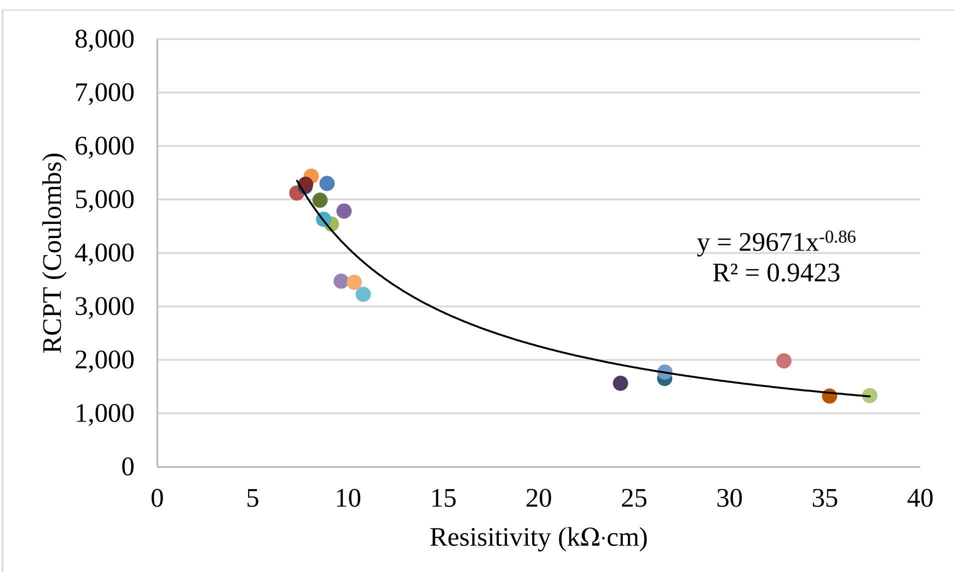

Figure 4.15: 90-day test results - RCPT vs. surface resistivity

The results found in the comparison of the RCPT and the surface resistivity test showed a very tight correlation of the two results, similar to the correlation between these two test results found by other researchers (Rupnow and Icenogle 2011). In Figure 4.16, where data from both 28-day and 90-day tests is plotted together, it is clear that the results of the RCPT test at both 28-days and 90-days can be reasonably predicted using the surface resistivity meter test results. Use of the surface resistivity meter on compressive strength test cylinder specimens should provide a rapid durability assessment of concrete for NCDOT.

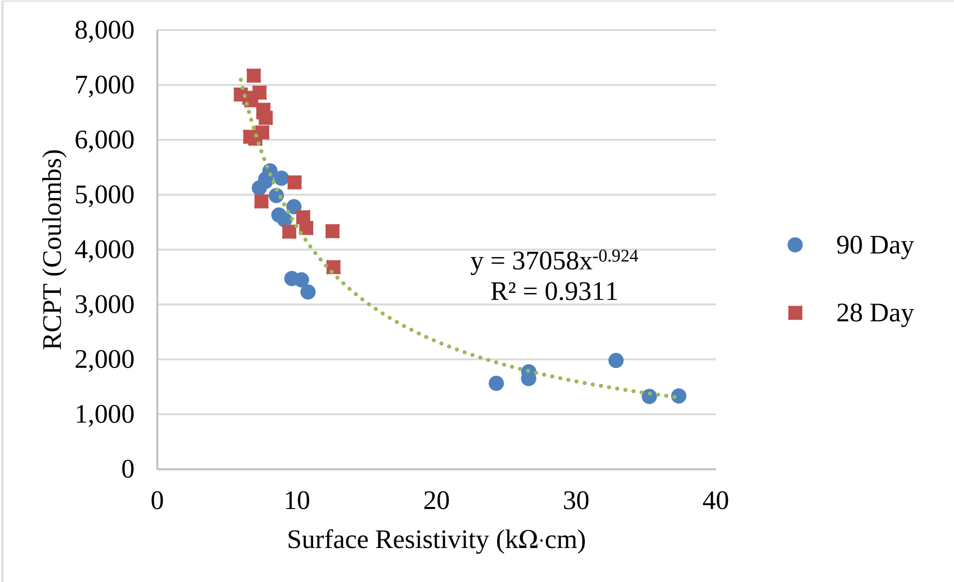

Figure 4.16: Correlation between RCPT results and surface resistivity test results

4.2.4 Relationship of Compressive Strength to Permeability

Although durability characteristics can sometimes be correlated with compressive strength of concrete, results from these did not show a strong association. In fact, in most cases, the concrete that was associated with the higher surface resistivity and lowest RCPT values (both indicators of good expected durability) were also the specimens with the lowest compressive strength. Figures 4.17 and 4.18 illustrate this trend. Results indicated that permeability and expected durability are more strongly associated with the presence of fly ash and interground limestone than they are with high levels of compressive strength. The mixtures with SCMs are circled to highlight their position on the low permeability region of the scale.

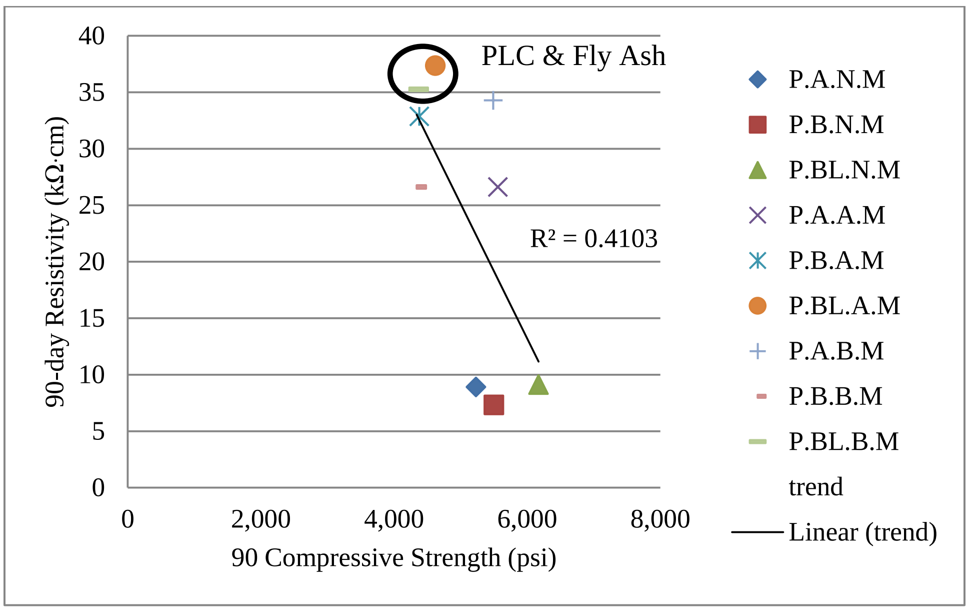

Figure 4.17: 90-day resistivity vs. 90-day compressive strength

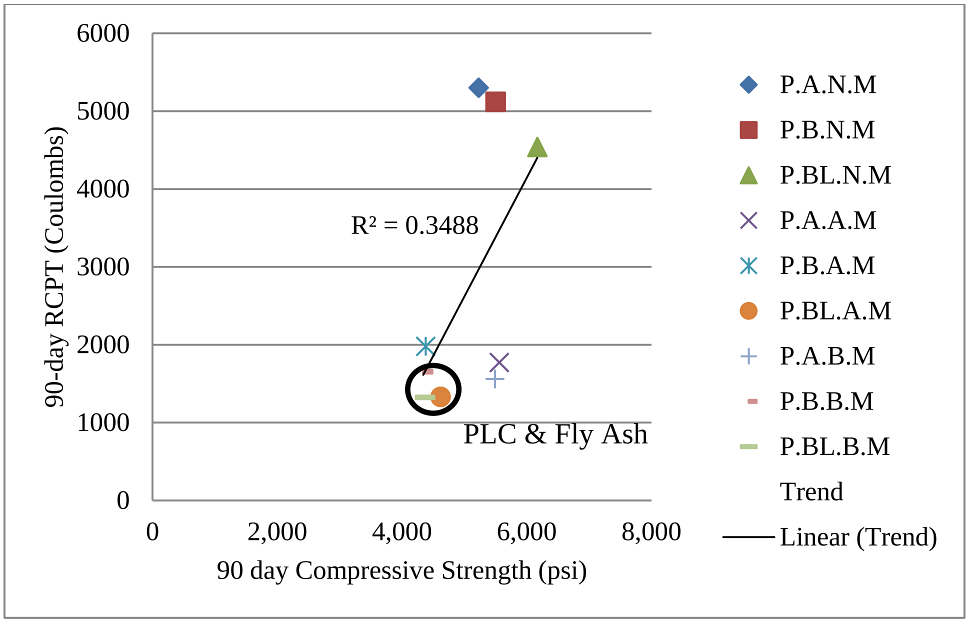

Figure 4.18: 90-day RCPT vs. compressive strength

4.3 Limited Lifecycle Assessment

4.3.1 Introduction and Goals

To quantify the sustainability benefits that may be associated with use of PLC, a *limited* life cycle assessment (LCA) analysis was performed using a web-based LCA tool, Green Concrete. Developed by researchers at the University of California at Berkeley, the Green Concrete LCA tool was specifically designed to aid in quantification and comparison of the potential environmental impacts of different concrete mixtures produced. This tool can also be used to help the industry stakeholders evaluate the environmental impacts of different materials and technologies utilized in concrete construction, and to make choices based on the potential environmental impacts of the considered alternatives (Gursel and Custodio 2017).

The Green Concrete web tool is based on MS-Excel operations. The web tool consists of user inputs and results sections, connected with a reference data pool and a "processes and calculations" computational engine. To support the LCA, many input values can either be user-identified, or default values can be selected. The reference data pool consists of four life cycle inventory (LCI) data sets. LCI datasets supporting the analysis include electricity grid mix data, transportation data, facilities operation data, and fuel (pre-combustion and combustion) data. This LCI of materials, fuels, and electricity are organized to support analysis of each material production phase in the process and calculation section. The functional unit in the Green Concrete tool can be defined as the unit volume of ready-mix concrete exiting the concrete plant. Results from the LCA include resources use, primary energy use, water consumption, and air emissions. Air emissions include global warming potential (GWP) in CO_2 equivalents (CO_2-eq) for production of concrete, cement, and admixtures. Air emissions also include air pollutants released during production process such as CO, NO_X, lead, PM_{10}, SO_2, and volatile organic compounds (VOC).

To aid in justifying the use of PLC (in lieu of OPC) with respect to sustainability in future pavement projects in North Carolina, the goal of this *limited* LCA analysis was to quantify the environmental impact, as measured by total criteria air pollutant emissions, associated with production of concrete produced using cement of different limestone contents (0%, 5%, 10%, 12%, 15%, and 20%). Analyses were also performed on concrete produced using fly ash (and companion mixtures without fly ash) in order to evaluate the reduction in environmental impacts associated with addition of fly ash in the concrete mixtures. Additionally, the impact of changes in technology for finish milling and change energy source in electricity grid were also analyzed. The functional unit in the Green Concrete tool is defined as the unit volume (m^3) of ready-mix concrete exiting the concrete plant. Therefore, for this analysis, the amount of concrete considered for comparison between mixture and/or processing alternatives is 1 m^3 of concrete produced.

4.3.2 System Boundaries

The system boundaries utilized by the Green Concrete LCA tool is shown in Figures 4.19 and 4.20. Included within the system boundary is the production of cement, SCMs, and aggregates along with energy sources including fuels for energy and transportation. The system boundary excludes burdens from the work force such as accidents, infrastructure, and human resources. The analysis also excludes the energy required to produce the fuels that are needed to produce cement, admixtures, aggregates, SCMs, and concrete.

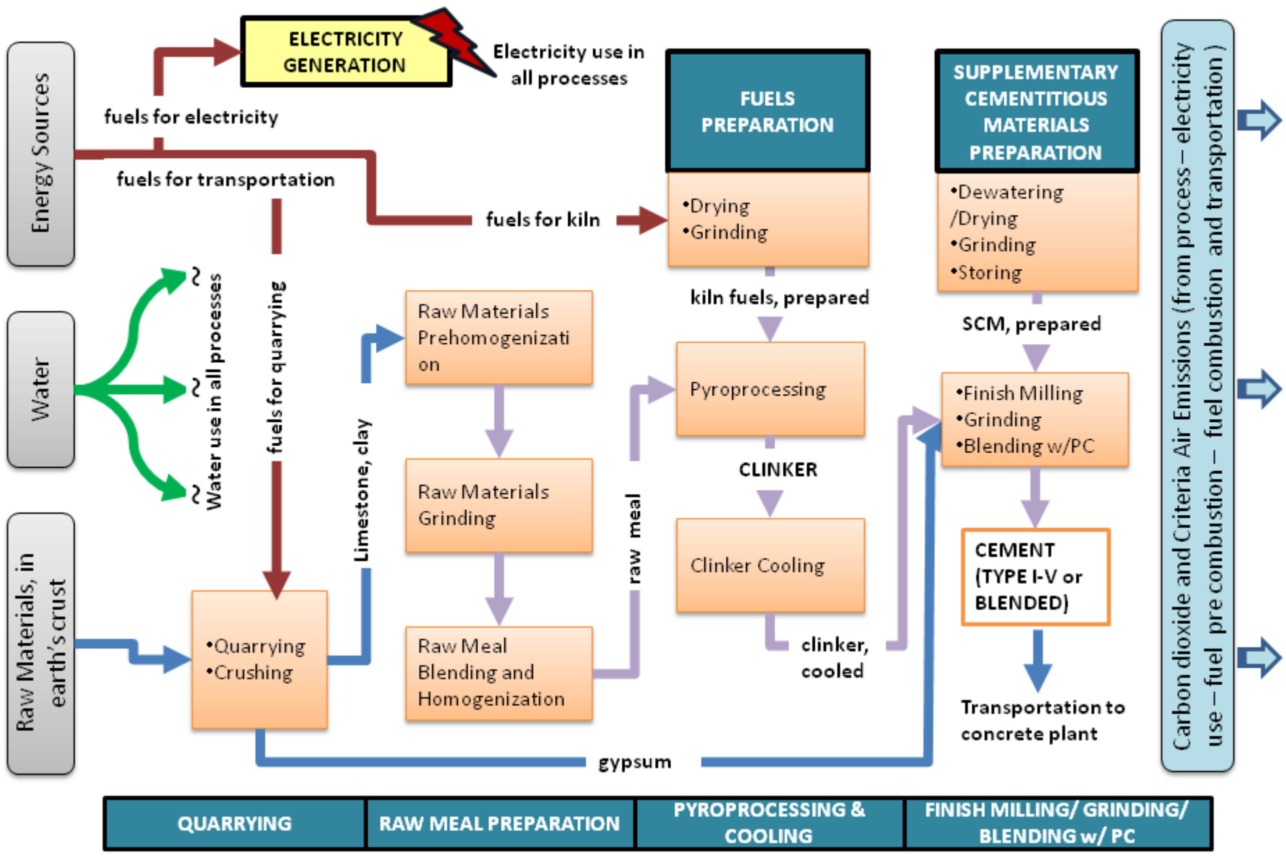

Figure 4.19: Cement production processes (from the Green Concrete Web Tool)

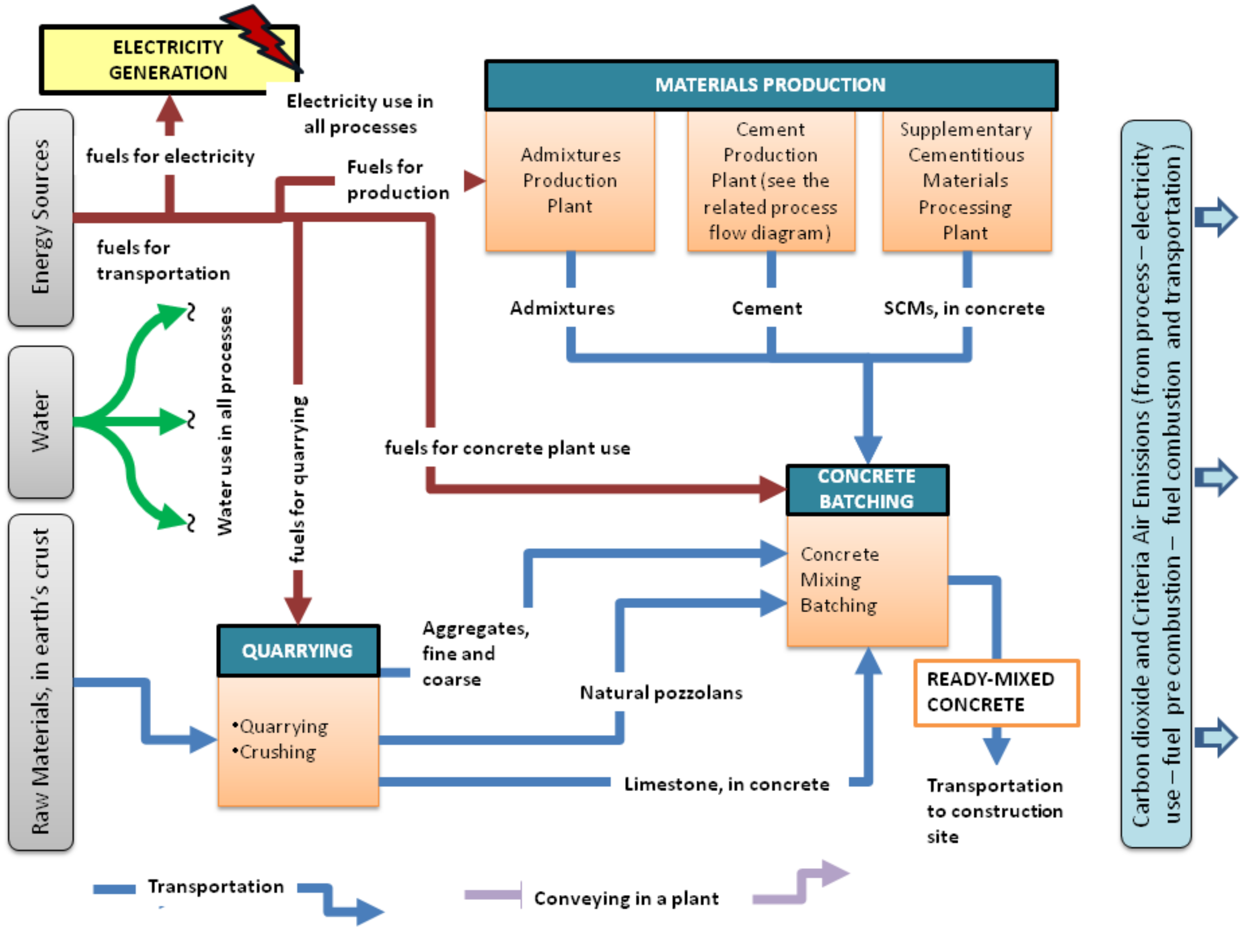

Figure 4.20: Concrete production processes (from the Green Concrete Web Tool)

4.3.3 Inputs

To define the cement plant configuration, fuel sources, distance commuted for materials delivery and in-plant hauling, and technology used in plant operations (with the exception of finish milling) were held constant, with values utilized in this analysis obtained from a southeast region cement manufacturer through a confidential survey. Data on raw materials in production of cement were also collected from that manufacturer using a confidential survey. This data included information on the amount of cement clinker, gypsum, and mode of transportation to the plant.

Data on raw materials for concrete production were collected from the laboratory testing performed as part of this work and from available data from the Green Concrete web tool. Other data used to support the Green Concrete LCA analysis includes fuel and electricity, transportation, processing technology, emissions, and cement production technologies and plant operation assumptions. In the interest of brevity, a description of the available data and choices for each of these inputs is presented in Appendix D, and is published in Chimmula (2016).

For each concrete mixture included in this limited LCA, concrete mixture proportions for 1 m^3 were input, with quantities for each mixture component (including cement, SCMs, aggregates, water, and admixtures) input in units of kg/m^3. The quantities of cement and limestone used for analysis are provided in Table 4.12. Based on the mixture design used for this study, the amount of coarse aggregate, fine aggregate, water, water reducing admixture, and air entraining admixture used for analysis are 1067.69 kg, 746.61 kg, 172.61 kg, 3.077 kg, and 0.77 kg respectively. The type of OPC selected for analysis was portland cement moderate sulfate resistance, type II. The cement type and amount of raw materials utilized in the production of the cement was calculated by web tool based on quantity of inputs given in material quantities section.

Table 4.12: Mixture proportions for limited LCA

Mixture type (cement type)	Cement content (kg/m^3)	Limestone content (kg/m^3)	Fly ash content (kg/m^3)	Coarse aggregate content (kg/m^3)	Fine aggregate content (kg/m^3)	Water (kg/m^3)	Water reducing admixture (kg/m^3)	Air entraining admixture (kg/m^3)
0% limestone without fly ash	326.30	0	0	1067.69	746.61	172.61	3.08	0.77
0% limestone with fly ash	261.04	0	65.26	1067.69	746.61	172.61	3.08	0.77
5% limestone without fly ash	309.98	16.01	0	1067.69	746.61	172.61	3.08	0.77
5% limestone with fly ash	247.98	16.01	62.31	1067.69	746.61	172.61	3.08	0.77
10% limestone without fly ash	293.67	32.63	0	1067.69	746.61	172.61	3.08	0.77
10% limestone with fly ash	234.93	32.63	58.74	1067.69	746.61	172.61	3.08	0.77
12% limestone without fly ash	287.14	39.15	0	1067.69	746.61	172.61	3.08	0.77
12% limestone with fly ash	229.71	39.15	57.44	1067.69	746.61	172.61	3.08	0.77
15% limestone without fly ash	277.35	48.94	0	1067.69	746.61	172.61	3.08	0.77
15% limestone with fly ash	221.88	48.94	55.48	1067.69	746.61	172.61	3.08	0.77
20% limestone without fly ash	261.04	65.26	0	1067.69	746.61	172.61	3.08	0.77
20% limestone with fly ash	208.83	65.26	52.21	1067.69	746.61	172.61	3.08	0.77

4.3.4 Calculations and Methodology

The framework supporting the Green Concrete LCA tool are shown in Figure 4.21 The process and calculation sections are additional details regarding the computational approach of the Green Concrete webtool are provided at the Green Concrete website and within Chimmula (2016).

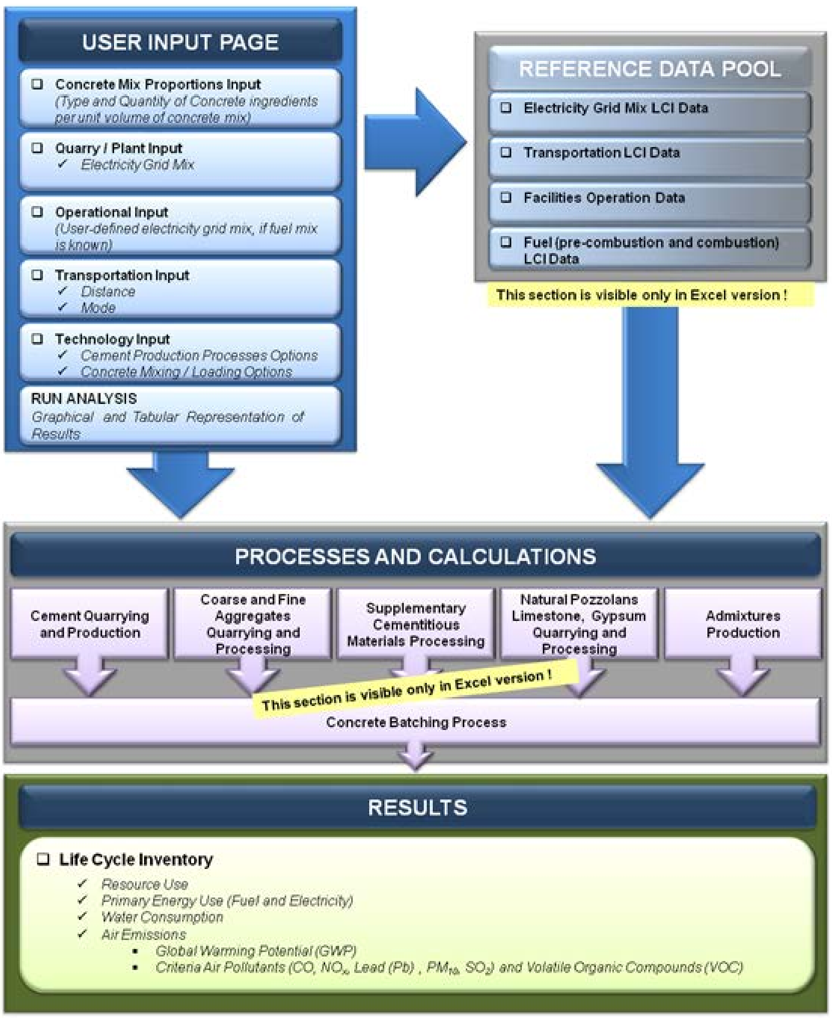

Figure 4.21: LCA structure of Green Concrete tool (from Green Concrete)

4.3.5 Output (Results)

For this study, LCA analysis was performed by running sequential analyses with the Green Concrete LCA web tool using the different percentages of limestone in the cement, as shown in Table 4.12. Output of the Green Concrete LCA includes graphs showing energy consumed and global warming potential (GWP) at each phase (shown in Figure 4.22), along with a table showing criteria air pollutants released at each phase (shown in Table 4.23). Units displayed in Figure 4.23 and Table 4.12 are in kilograms, although it is important to note that these results are based on one m^3 concrete and could therefore be considered as units of "kg/m^3 of concrete." It is also noted that due to the complexities associated with the LCA analysis, particularly those associated with GWP, this *limited* LCA analysis only focused on quantifying the contribution of criteria air emissions for the mixtures of interest using the configurations and inputs described above. At the time of this study, the publication LCA Framework for Pavements (Harvey et al. 2016) was not available, and is recommended for future LCA analyses.

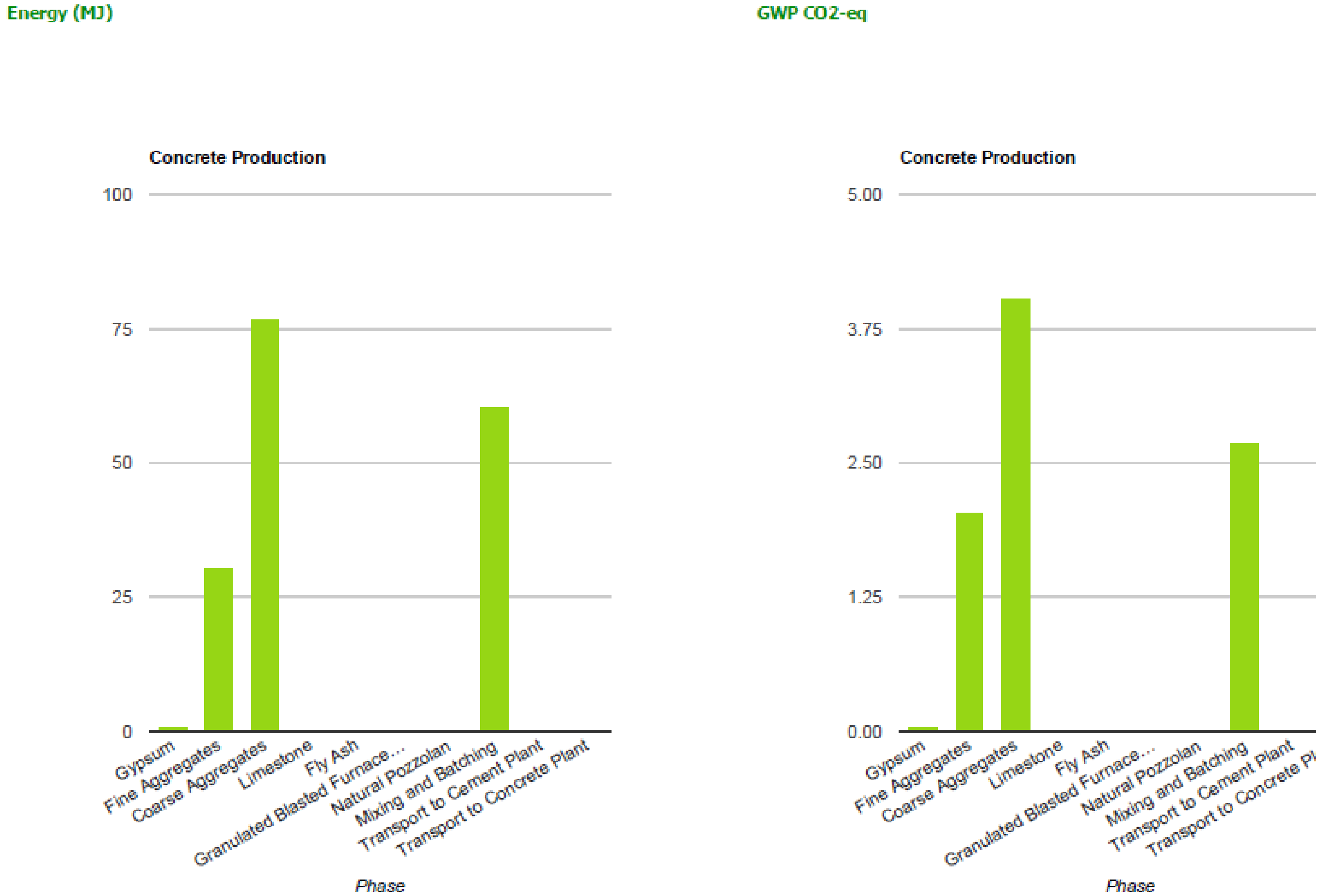

Figure 4.22 Sample graphs of energy and global warming potential (GWP) in CO_2-eq from Green Concrete Analysis (from Green Concrete web tool)

Table 4.12 Sample table of criteria air emissions from Green Concrete LCA analysis (from Green Concrete web tool)

Phase	CO (kg)	Lead (kg)	NOx (kg)	PM10 (kg)	SO2 (kg)	VOC (kg)
Gypsum	0.000	0.000	0.000	0.002	0.000	0.000
Fine Aggregates	0.008	0.000	0.003	0.019	0.005	0.000
Coarse Aggregates	0.018	0.000	0.005	0.028	0.009	0.000
Cement: Quarrying	0.011	0.000	0.004	0.000	0.006	0.000
Cement: Raw Materials Prehomogenization	0.000	0.000	0.000	0.000	0.000	0.000
Cement: Raw Materials Grinding	0.010	0.000	0.004	0.000	0.009	0.000
Cement: Raw Meal Blending/Homogenization	0.000	0.000	0.000	0.000	0.000	0.000
Cement: Pyroprocessing	99.090	0.015	0.949	0.026	0.722	0.012
Cement: Clinker Cooling	0.003	0.000	0.001	0.000	0.003	0.000
Cement: Finish Milling and Grinding and Blending with PC	0.019	0.000	0.007	0.000	0.016	0.000
Cement: In-Cement Plant Convey	0.001	0.000	0.000	0.000	0.001	0.000
Fly Ash in Cement	0.000	0.000	0.000	0.000	0.000	0.000
Granulated Blast Furnace Slag in Cement	0.000	0.000	0.000	0.000	0.000	0.000
Plasticiser	0.000	0.000	0.000	0.000	0.000	0.000
Superplasticiser	0.002	0.000	0.006	0.000	0.011	0.001
Retarder	0.000	0.000	0.000	0.000	0.000	0.000
Accelerating Admixture	0.000	0.000	0.000	0.000	0.000	0.000
Air Entraining Admixture	0.000	0.000	0.000	0.000	0.000	0.000
Waterproofing	0.000	0.000	0.000	0.000	0.000	0.000
Limestone	0.000	0.000	0.000	0.000	0.000	0.000
Fly Ash	0.000	0.000	0.000	0.000	0.000	0.000
Granulated Blasted Furnace Slag	0.000	0.000	0.000	0.000	0.000	0.000
Natural Pozzolan	0.000	0.000	0.000	0.000	0.000	0.000
Mixing and Batching	0.006	0.000	0.003	0.000	0.005	0.000
Transport to Cement Plant	0.000	0.000	0.000	0.000	0.000	0.000
Transport to Concrete Plant	0.000	0.000	0.000	0.000	0.000	0.000

For this study, the Green Concrete LCA analysis was performed using cements with 0%, 5%, 10%, 12%, 15%, and 20% limestone content. For this base analysis, the technology used for finish milling was ball mill for all cements. The impact of addition of fly ash to the same base concrete mixture was explored in the LCA analysis, along with the potential changes in criteria air pollutant emissions associated with changes in finish milling technology and selected changes in energy grid source mix. It is noted that the PLC used in this study contained approximately 12% interground limestone. The following was determined via the LCA analysis:

1. Increasing limestone content in PLC results in a reduction in total criteria air emissions roughly equivalent to the percentage of limestone content. For the materials used in this simulation, by increasing the limestone content in cement from 0% to 20%, total criteria air pollutant emissions may be reduced up to 20%. (shown in Figure 4.23).
2. By replacing fly ash up to 20% in cement quantity, the predicted total criteria air pollutant emissions for concrete were reduced up to 20% (shown in Figure 4.23).
3. The type of finishing mill utilized for intergrinding the limestone into cement and selected changes to the source of electricity for the South Carolina electricity grid used in the analysis did not significantly influence the air pollutants predicted for a cubic meter of concrete (shown in Appendix D, Figures D.1 and D.2).

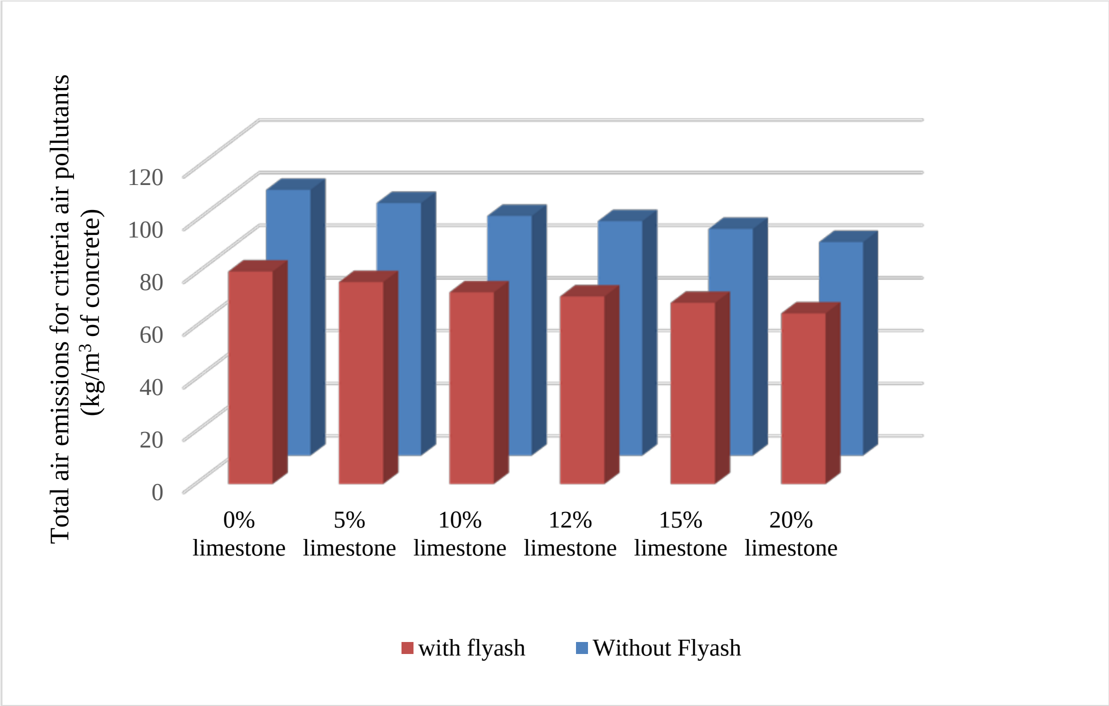

Figure 4.23: Criteria air pollutant emissions predicted by Green Concrete LCA for mixtures with/without interground limestone and with/without fly ash

4.4 Industry Forecast for PLC

The Portland Cement Association recently surveyed US cement plants to determine the extent of production of PLC during the period of 2012 to 2016. With a response from approximately 89% of domestic cement plants (87 of 98 plants), PCA determined that approximately 2.74 million metric tons of PLC was produced by 30 plants nationwide. About 890,000 metric tons of PLC was produced in 2016 (an almost 70% increase from 2015 production), and around 24% of the plants surveyed were producing PLC in 2016 (Tennis 2017). However, PCA also estimates that overall consumption of portland cements in the US was 91.9 million metric tons, and therefore domestic PLC production amounted to less than 1% of cement consumption (PCA 2017). Seven cement plants located in the southeast region (AL, GA, KY, MS, NC, TN, SC, VA) reported producing PLC at least one time between 2012 and 2016 (Tennis 2017). Plants responding to the survey had an option to self-identify, and plants located in the southeastern region indicating that they had produced PLC (Type IL) cement in 2017 included one plant in Atlanta, GA, one plant in Harleyville, SC, one plant in Clinchfield, GA, and two plants in Alabama.

Discussions with PCA and industry representatives indicates that PLC is not readily available to the North Carolina market at the time of the writing of this report (Tennis 2017 and others). The decision to produce PLC is driven by economic considerations, and production logistics (such as kiln production rates, grinding production rates, or storage logistics). However, the PCA concludes in their study that “as more specifying agencies, like state DOTs begin accepting and using PLCs, the volume of PLC produced may increase in those regions (Tennis 2017).”

Figure E.1 (provided in Appendix E) shows a map of the United States indicating acceptance of PLC by state DOTs, prepared by PCA as part of their PLC production survey. As of July 2017, two southeastern state DOTs (Georgia and Alabama) had not yet accepted PLC use by including it in their specifications. Given the demand for cement within these markets, particularly Georgia, it could be anticipated that PLC will be available for use by NCDOT if these states chose to allow PLC. It is our understanding that Georgia DOT is sponsoring ongoing research in this area to support their decision regarding PLC.

At the National Concrete Consortium (NCC) cement industry representatives at the local and regional level were approached by the project PI regarding the potential costs of PLC. Based on these informal discussions, it is anticipated that the cost of PLC may not differ substantially from OPC. Based on the similar performance of these cements in laboratory testing, a cost-benefit analysis to evaluate the long-term economic benefits that could be realized by use of PLC concrete in lieu of OPC concrete does not appear to be appropriate at this time.

5. SUMMARY AND CONCLUSIONS

Laboratory test results for the matrix of mixtures were analyzed, and a catalog of inputs was proposed for consideration for use by NCDOT. This proposed catalog, along with the suite of data from supporting laboratory test results, should provide confidence to North Carolina Pavement designers about inputs for use in design of PCC pavements. In addition to providing M-EPDG inputs, the performance of PLC concrete was evaluated and compared to conventional OPC and OPC flyash mixtures with North Carolina materials. Key findings from the laboratory testing included:

- 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 concrete inputs for M-EPDG. Comparable performance of the PLC provides incentive to NCDOT for use of this sustainable alternative to OPC. Modifications to specifications to accommodate PLC are not recommended at this time, although, following the lead of some state agencies, NCDOT could consider specifying use of SCMs with PLCs when sulfate resistance is required.

- The fine aggregate utilized in the concrete mixture (manufactured sand versus natural sand) had significant influence on the thermal properties of two concrete inputs in M-EPDG, CTE and thermal conductivity. The reason for this influence is likely due to the mineral composition of the manufactured sand (granitic gneiss), which likely has a lower CTE than the natural silica sand.

- 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 concrete inputs for M-EPDG.

- Use of fly ash in concrete pavement mixtures may make it unsuitable to utilize the 28-day compressive strength as a PCC input in MEPDG due to the delayed strength gain. This finding could lead to a specification change in the future.

- Use of fly ash in pavement concrete should improve durability performance. Use of fly ash in concrete pavement mixtures provided significant reductions in chloride ion permeability (between 28-days and 90-days, an average reduction in charge passed of approximately 218% vs. 32% for fly ash vs. non-fly ash mixtures, respectively).

- Use of PLC alone (without fly ash) did not provide distinct durability performance advantages, when compared to OPC. However, if PLC is utilized with fly ash in concrete mixtures, enhanced durability performance could be anticipated. The lower permeability exhibited by PLC concrete with fly ash is likely due to the particle packing effects in these binder systems, and is consistent with the findings of other researchers. When PLC was paired with fly ash, chloride permeability (as measured by the RCPT) was approximately 30% lower than that of OPC/fly ash mixtures, at both 28-days and 90-days of age.

- Similar to the findings of other researchers, there is a strong correlation between surface resistivity test results and RCPT results for all mixtures included this study. Due to the ease of performing the surface resistivity test, as well as the difficulty and limitations of the RCPT, it is recommended that NCDOT investigate the potential for increased use of surface resistivity in concrete specifications.

- Although durability characteristics can sometimes be correlated with compressive strength of concrete, results from these did not show a strong association. In fact, in most cases, the concrete that was associated with the higher surface resistivity and lowest RCPT values (both indicators of good expected durability) were also the specimens with the lowest compressive strength.

A sensitivity analysis was performed using AASHTOWare Pavement ME to compare the relative sensitivity of each input on predicted pavement distresses. Key findings included:

- 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.

- 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.

Several typical North Carolina concrete pavements were analyzed using previous and newly suggested PCC inputs using the original design constraints. Key findings included:

- Optimal use of M-EPDG requires a local calibration that involves comparison of distresses measured on a number of local projects with the distresses predicted by the MEPDG software. Calibration coefficients in the distress prediction models are adjusted to minimize the difference between the measured and calculated distresses. Use of the measured properties in local calibration should lead to better correlation between measured and calculated distresses. This will result in a smaller standard estimates of error (SEE) and thinner designs at higher reliability levels.

- 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.

- Use of the new PCC input values may result in the design of slightly thinner concrete pavements in the future. In one case, a 1” thinner concrete pavement section analyzed utilizing the new input values provided satisfactory performance. Thinner pavements will reduce the amount of materials used in pavement construction, resulting in lower costs and environmental impact of concrete pavement. The benefits of deciding to reduce the concrete thickness should be weighed against the risks associated with under-prediction of traffic or section loss associated with one or more diamond grinding treatments during the service life of the pavement.

Based on information solicited from PCA representatives (Tennis 2017) and from local industry representatives, PLC is not readily available for use in the North Carolina market. At this time, it is anticipated that the price of PLC will be similar to the price of OPC. Based on these findings, a limited LCA using the online concrete industry-focused LCA webtool Green Concrete was performed to quantify the potential sustainability benefits of using PLC. Findings of the limited LCA indicated that use of PLC could potentially reduce the total criteria air pollutant emissions associated with concrete production to an extent roughly proportional to the percentage of interground limestone. Additionally, replacing OPC with fly ash resulted in a similar decrease in predicted total criteria air pollutant emissions, roughly proportional to the percentage of cement replaced by fly ash. This analysis provides some confidence to NCDOT that use of PLC and fly ash in concrete infrastructure should provide environmental and sustainability benefits, as mandated by the MAP-21 legislation.

6. VALUE OF RESEARCH FINDINGS and RECOMMENDATIONS

6.1 Value of Research Findings

- PCC inputs for North Carolina concrete pavements identified in this study indicated that some default input values (or input values used by NCDOT) were conservative. Based on analysis performed as part of this work, these new locally-representative input values can result in design pavement thicknesses up to 1 inch thinner than currently being utilized. To quantify the value of this finding for a previously constructed PCC roadway, a computation of the potential initial cost savings of reducing 1 inch (or even 1/2 inch) of concrete thickness could be performed by deducting from the contract price an appropriate cost per cubic yard of (as-constructed) concrete. This savings could also be quantified on an annual basis or on a percent cost savings per lane-mile basis.

- Conversely, not reducing the thicknesses of concrete pavements may offer longer service lives with reduced maintenance. The value of this extended service life and potentially reduced maintenance cannot be quantified at this time. However, if quantified through an LCCA or other analysis, benefits over the lifecycle associated with this additional thickness could very likely be significantly greater than the initial cost savings associated with a slightly thinner pavement.

- To help quantify the potential sustainability benefits of use of PLC instead of OPC, and to support the increased use of fly ash, the potential reduction in criteria air pollutants (in kg or lb of emissions) could be computed for a roadway or roadways. Using the findings of the limited LCA analysis, the reduction in total pollutant emissions per cubic meter of concrete is roughly proportional to the replacement rate of PLC and/or fly ash. Calculations using this approximate reduction in total pollutant emissions, along with a given volume of concrete produced for a PCC roadway (or roadways), will provide an estimate of the total reduction in estimated criteria air pollutant emissions savings.

6.2 Recommendations

Following are the recommendations pertaining to the findings of this study:

- The catalog of recommended PCC inputs for M-EPDG was provided to the Steering and Implementation Committee in digital format prior to publication of this report. It is our understanding that these inputs were provided to an engineering consultant retained to assist NCDOT in local calibration efforts. If issues arise regarding correlation between measured and calculated distresses for certain types of rigid concrete pavements (or rigid pavements in specific areas), additional laboratory testing could be performed to refine the catalog of inputs.

- It was evident from this study that the potential shift from use of natural sand to manufactured sand in some area will affect not only fresh and early age properties of concrete during the construction phase, but will also affect the long-term performance of concrete pavements. Generally, the measured properties of concrete utilizing manufactured sand provide evidence that these mixtures could provide enhanced durability performance. Additional efforts to better understand the potential performance of these pavements could aid in determining durations for maintenance actions, expected service life for life cycle cost purposes.

- Use of PLC should provide equivalent performance to OPC in North Carolina concrete mixtures, while providing sustainability benefits associated with reduced emissions. NCDOT should encourage use of PLC in North Carolina infrastructure. NCDOT should specify inclusion of SCMs if PLCs are utilized in high sulfate environments.

- Incorporating fly ash into OPC and PLC concrete provided test results linked to significant durability performance improvements over non-fly ash mixtures. Use of fly ash in North Carolina pavements should provide enhanced durability, extended service lives and, potentially, savings over the life cycle.

- When PLC was utilized with fly ash, durability performance test results improved over those where OPC was utilized with fly ash. The potential performance benefits of mixtures including both PLC and fly ash could be explored in a laboratory study aimed at optimization of these mixtures.

- Surface resistivity testing shows great promise for rapid evaluation of the durability performance of North Carolina concrete mixtures. Additional research to support implementation of surface resistivity is suggested (this has been supported via funding of NCDOT RP 2018-14).

- The Super Air Meter (SAM) test shows promise to provide information on the potential freeze-thaw durability of North Carolina concrete mixtures. Additional research to support implementation of the SAM for appropriate projects is suggested (this has been supported via funding of NCDOT RP 2018-14).

- A more robust LCA could potentially be performed using plant-specific data and a more robust LCA tool (such as the Athena LCA tool, or other), to provide confidence in the quantification of the sustainability benefits of use of PLC in North Carolina concrete pavements. The LCA framework for pavements (Harvey et al. 2016) is the most appropriate means of performing this assessment.

7. IMPLEMENTATION AND TECHNOLOGY TRANSFER PLAN

Findings of this study confirmed that allowing PLC in NCDOT specifications is appropriate, and may offer advantages associated with enhanced durability if PLC are utilized with fly ash in concrete infrastructure. Since PLC are not currently available on the market, stakeholder confidence in use of these cements could not readily be judged. As PLC become available for use in North Carolina, the research team can provide feedback to stakeholders interested in using these cements.

Research Product 1	Catalog of recommended PCC inputs for M-EPDG
Suggested User	Pavement Design & Collection Unit, Materials & Tests Unit
Recommended Use	The catalog of recommended PCC inputs for M-EPDG was provided to the Steering and Implementation Committee in digital format prior to publication of this report. It is our understanding that these inputs were provided to an engineering consultant retained to assist NCDOT in local calibration efforts. The research team is available to answer questions or provide feedback on this effort as requested.
Recommended Training	None recommended at this time.

Research Product 2	Digital database of test results from laboratory testing
Suggested User	Pavement Design & Collection Unit, Materials & Tests Unit
Recommended Use	Information contained in this database could serve as reference data for evaluation of concrete mixtures and/or test methods in future work. Data could also be used to supplement additional databases on maintained by the Materials and Tests Unit.
Recommended Training	None recommended at this time.

Research Product 3	Laboratory test data indicating that PLC should perform similarly to OPC in North Carolina pavement concrete mixtures
Suggested User	Pavement Design & Collection Unit and Materials & Tests Unit
Recommended Use	This information supports NCDOT's decision to allow PLCs, and could also aid in industry acceptance of PLCs once available for use in the North Carolina market.
Recommended Training	None recommended at this time.

Research Product 4	Laboratory test data confirming the durability benefits of use of fly ash in pavement mixtures.
Suggested User	Pavement Design & Collection Unit and Materials & Tests Unit
Recommended Use	This information could be utilized to support decisions to specify that fly ash be incorporated in concrete for certain projects.
Recommended Training	None recommended at this time.

Research Product 5	Limited LCA analysis results quantifying the potential total criteria air pollutant emissions savings that could be realized with use of PLC and fly ash.
Suggested User	Pavement Design & Collection Unit and Materials & Tests Unit
Recommended Use	This information could be utilized to provide evidence that NCDOT is moving towards a more sustainable infrastructure, consistent with MAP-21 initiatives.
Recommended Training	None recommended at this time.

Research Product 6	**S**urface resistivity measurements of a variety of North Carolina concrete pavement mixtures.
Suggested User	Materials & Tests Unit
Recommended Use	Surface resistivity tests results have been shown to strongly correlate with results of ASTM C1202 RCPT results by this study and others. Surface resistivity measurements could be utilized to specify more durable concrete and to evaluate the durability of existing concrete.
Recommended Training	If surface resistivity is integrated into procedures utilized by the Materials & Tests Unit, minimal training on the device would be required. AASHTO standard T 358-17 can be used as guidance for use of the surface resistivity meter in the laboratory setting. UNC Charlotte personnel could meet with Materials & Tests Unit personnel to assist in training, if requested.

Research Product 7	Super Air Meter (SAM) tests of a variety of North Carolina concrete pavement mixtures.
Suggested User	Materials & Tests Unit
Recommended Use	The SAM device has been shown to be useful in evaluating the potential freeze-thaw durability of a concrete mixture prior to being placed. This data can be used in the ongoing evaluation of this test device for use in North Carolina.
Recommended Training	This test is outlined in AASHTO TP 118, and online training videos are available. UNC Charlotte personnel could meet with Materials & Tests Unit personnel to assist in training, if requested.

8. REFERENCES

Note: References listed below are cited in the body of the report. A full list of references utilized to support this work is provided at the end of Appendix A, which contains the complete Literature Review for this project.

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APPENDICES

FOR
FINAL REPORT

North Carolina Department of Transportation
Research Project No. 2015-03

Improved Data for Mechanistic-Empirical Pavement Design for Concrete Pavements

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APPENDIX A – LITERATURE REVIEW AND REFERENCES

A.1 Motivation for Study and Research Needs

Local calibration is necessary for optimal performance of AASHTOWare Pavement ME for the design and performance processes (AASHTO 2010). Although locally calibrated inputs for flexible (asphalt) pavements were determined as part of a national study (FHWA 2010), local calibration of concrete pavements has not been performed 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). 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 M-EPDG 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. The following literature review provides background information in support of these motivations and needs.

A.2 Mechanistic-Empirical Pavement Design for Rigid Pavements

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

A.2.1 Mechanistic-Empirical Pavement Design Guide (M-EPDG)

M-EPDG is a state-of-the-practice tool for design of pavements for the transportation industry. The M-EPDG process uses mechanistic models to compute pavement responses to traffic and climate loads to forecast damage over time, predicting the performance of a user-specified pavement section through its design life. The cumulative damage is empirically related to observed pavement distresses through the local calibration process. If a user changes input values such as climate, traffic, and pavement information, the M-EPDG 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 several sources (AASHTO 2015). However, the local calibration of M-EPDG, 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 M-EPDG 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 predicts the performance associated with three main distress failure modes for jointed plain concrete pavements (JPCP): transverse cracking, mean joint faulting, and terminal international roughness index (IRI) (AASHTO 2015). Threshold performance values for each distress are identified by the local agency depending on a variety of project conditions and agency preferences. A brief description of each type of distress along with the generally agreed upon M-EPDG input parameters found to influence each distress, is discussed subsequently. Additional details regarding the sensitivity of distress to each M-EPDG input are presented in Sections A.1.4.

Transverse cracking is measured in "percent slabs" (cracked) for use in M-EPDG and includes both predicted bottom-up and top-down cracking. Bottom-up cracking typically results 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 (AASHTO 2010). Top-down transverse cracking is primarily associated with fatigue loading from truck traffic loads with certain axle spacings, 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). In sensitivity analyses previously performed for local calibrations of M-EPDG, an increase in concrete 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 (Gulcu et al. 2009).

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. Sensitivity analyses have shown that an increase in concrete Poisson's ratio and CTE leads to an increase in faulting, while an increase in concrete unit weight and thermal conductivity leads to a decrease in faulting (Gulcu et al. 2009).

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 quantifies the smoothness or roughness of the roadway, and is measured in "inches per mile." Sensitivity analyses have shown that several concrete inputs affect smoothness: an increase in Poisson's ratio and an increase in CTE each lead to an increase in smoothness, while an increase in thermal conductivity, MOR, and compressive strength leads to a decrease in smoothness (Guclu et al. 2009). Distresses are interrelated, in that joint faulting and (to some extent) transverse cracking will affect smoothness (IRI), and therefore relationships between inputs and predicted performance are not always straightforward. For example, a larger input value for concrete unit weight will result in an increase in predicted cracking, but a decrease in predicted faulting. Therefore, some researchers have identified unit weight to be "insensitive" for smoothness (Guclu et al. 2009).

A.2.2 Local Calibration of M-EPDG

Due to the variety of locally available materials utilized for pavement construction across the United States, as well as different subgrade conditions, climatic conditions, construction preferences, and other influencing factors, local calibration of M-EPDG is highly recommended by AASHTO for both rigid and flexible pavements. The local calibration and validation process reduces bias and error in observed results, facilitates the best working predictions and models, and provides confidence in the design process (AASHTO 2008). The calibration-validation process 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 includes confirming the predicted distress are similar to the actual observed distresses in the field (AASHTO 2010).

The suggested input values for materials in the global calibration of M-EPDG were determined using a representative sample of test sites around the country, primarily those included in the long-term pavement performance program (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 M-EPDG software unreliable (AASHTO 2010). The M-EPDG 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 incorporate the lowest level of local 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 M-EPDG, 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.

A.2.3 PCC Inputs in M-EPDG for Concrete Pavement Design

Rigid pavement design using M-EPDG requires several 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 M-EPDG are thermal conductivity, heat capacity, and coefficient of thermal expansion. The following sections contain information on the concrete materials-related inputs in M-EPDG. 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 M-EPDG.

A.2.3.1 Mechanical Properties

M-EPDG 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 M-EPDG 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 M-EPDG, 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 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 M-EPDG, 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 106 psi (for inadequate pavement condition) to 4 x 106 psi (for adequate pavement condition), with 1 x 106 psi to 3 x 106 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 M-EPDG, 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).

A.2.3.2 Thermal Properties

Thermal properties of PCC that are utilized as inputs in M-EPDG are CTE, heat capacity, and thermal conductivity. Thermal properties have been shown by several research studies to be significant in influencing the performance of pavements in M-EPDG (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 (Neville 2011, Mehta and Montiero 2014).

Cracks in a concrete structure can be caused by thermal effects, including the heat of hydration or occurrence of temperature gradient (Kook-Han et al. 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 M-EPDG 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 M-EPDG. 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, tested using AASHTO T 336 (AASHTO 2009). 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. 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 M-EPDG, the AASHTO 1993 Pavement Design Guide. The CTE was used for transverse joint sealant design and longitudinal reinforcement design (Tran et al. 2008). A focus of a number of recent studies supporting local calibration of M-EPDG for highway agencies, the CTE of concrete has been shown to be influenced by multiple components of a concrete mixture (including the cement paste, aggregates, and moisture), as well as characteristics such as age, and environmental factors (such as temperature fluctuations and relative humidity) (Naik et al. 2011). The greatest variation in the CTE value in concrete comes from the aggregate that is used (McCarthy et al. 2014, Naik et al. 2011, Sakyi-Bekoe 2008, Tanesi et al. 2007). 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).

A.2.4 Sensitivity Analysis

An analysis commonly utilized after local calibration of M-EPDG is a sensitivity analysis, which allows users to understand which variables will have the greatest influence on the predicted pavement performance. The Sensitivity Evaluation of M-EPDG 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 to develop 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 relatable to this research and will be useful for comparison of a similar sensitivity analysis with North Carolina inputs (and are therefore detailed herein).

In Table A.1, the findings of the sensitive inputs for M-EPDG as determined by Schwartz et al. (2011) are shown, sorted by degree of 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 observed in Table A.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.

Table A.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.

Research results in continued updates to the M-EPDG process, and subsequently to the AASHTOWare Pavement ME software. Therefore, the sensitivity of some parameters has changed over time as research supporting the initiative advances. For example, Schwartz et al. (2011) found that in versions of the M-EPDG 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. At the time of this study, the version of AASHTOWare Pavement ME software currently available and used in this study is version 2.1.

A.2.4.1 Sensitivity of M-EPDG to Input Values for Concrete Material Properties

Input values for PCC in M-EPDG fall under two 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, many states (including Alabama, Florida, Iowa, Louisiana, Oklahoma, and Hawaii) 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, Havel et al. 2015).

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 their 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.

A.2.4.1.1 Mechanical Properties

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).

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.

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.

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.

A.2.4.1.2 Thermal Properties

Three thermal properties of concrete are utilized as M-EPDG inputs. The first is CTE, which as discussed earlier, has been shown to be a very sensitive input value for rigid pavements, and much research exists in this area (Mallela et al. 2005, Tran et al. 2008, Sakyi-Bekoe 2008, Tanesi et al. 2010, Naik et al. 2011, Shin and Chung 2011, Gudimettla et al. 2012, McCarthy et al. 2014). 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.

Coefficient of Thermal Expansion

Aggregates typically account for 70% to 80% of the volume of concrete, 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 2011). 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 M-EPDG 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 single concrete 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 M-EPDG 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 M-EPDG.

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.

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, and also found that as the input value for thermal conductivity increases the predicted damage of each distress decreases.

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.

A.2.5 Materials Used in North Carolina Pavements

This study was funded by NCDOT to support development 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. The state is typically divided into three regions (Coastal, Mountain, and Piedmont), and variation exists in characteristics and engineering properties of materials sourced from (and by) each region. In Figure A.1, a map of North Carolina shows the generally accepted boundaries to the Mountain, Piedmont, and Coastal regions. The following sections provide background information on materials commonly utilized in North Carolina pavements along with information relevant to identification and use of materials-specific inputs in M-EPDG.

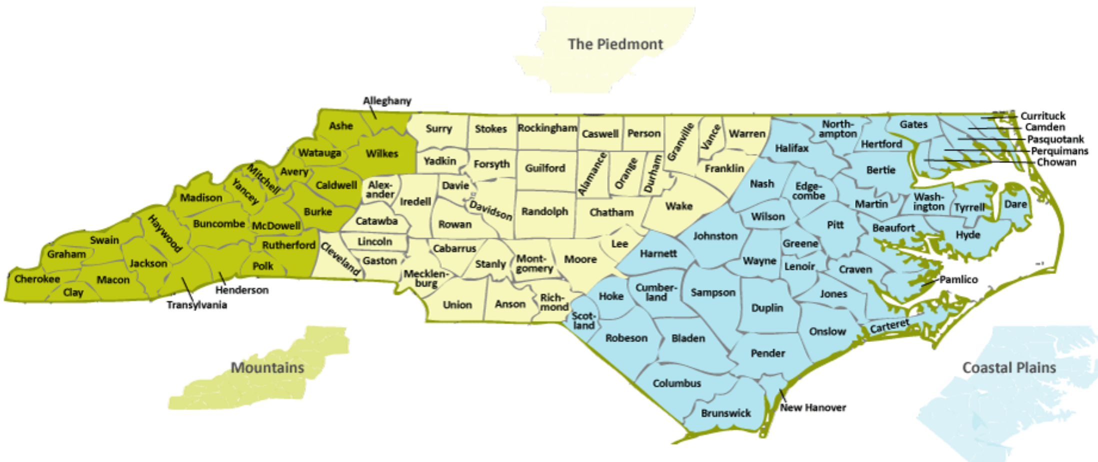

Figure A.1: North Carolina regional boundary map (NCPedia 2016)

A.2.5.1 Cementitious Materials

A.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) (NCDOT 2012). North Carolina currently does not have a cement manufacturing plant. 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 to achieve the required properties from the PCC.

A.2.5.1.2 Binary and Ternary Cement Blends

Supplementary cementitious materials (SCMs) such as fly ash, silica fume, slag and many others are used in concretes to replace a portion of the cement used in the mix. The SCMs typically have high-calcium elemental composition similar to portland cement, or are composed of aluminate or silicate minerals. Depending on their composition SCMs may have a cementitious characteristic due to the hydraulic activity of their constituent compounds and provide auxiliary strength to the concrete matrix. Other types of SCMs are capable of pozzolanic activity, which provides cementing action in the presence of portland cement. In addition to providing additional cementation, SCMs often provide benefits to the workability, durability, or cost economy of the concrete. Limestone powder provides only a mild contribution to the hydration reactions that lead to cementation, however, it impacts the performance of concrete in several other ways. The following sections describe the use of fly ash and limestone powder in concrete applications.

A.2.5.1.3 Fly Ash

Fly ash is an emissions control byproduct of coal combustion. The greatest source of fly ashes in the US originates from power generation facilities. On an annual basis, US facilities generate nearly 50 million tons of fly ash per year, although only a fraction of it is suitable and collected for recycled applications (American Coal Ash Association 2017). Fly ash is the portion of mineral particulate that is light enough to travel out of the combustion chamber with the flue gases. It is captured in emissions control equipment and stored for transfer to end users. The ash may be distributed for many geotechnical and structural applications as well as inclusion in specialized materials such as paints and polymers.

Because of their formation in flue gas, the fly ash particles are typically very fine, hollow, spherical shapes. Particles diameters range from 10 to 100 microns and are generally smaller than portland cement particles and limestone particles. Fly ash particles consist mainly of silicon oxide, aluminum oxide, iron oxide, and calcium oxide (American Coal Ash Association 2003). A typical X-Ray fluorescence oxide analysis of a low-calcium fly ash sample is shown in Table A.2. The predominant constituents are silica and alumina, however it is common to find other inclusions in the fly ash. One inclusion which sometimes limits the potential for use in concrete is carbon. Carbon can either be present in the ash as a residue of the combustion process or it may have been added as an emissions control strategy to adsorb mercury from the plant exhaust. In either case, carbon is typically measured as loss on ignition (LOI), and must be limited in concrete applications.

Table A.2: Example of composition of fly ash

Oxide	% by Mass
SiO_2	56.20
TiO_2	1.46
Al_2O_3	28.00
Fe_2O_3	5.22
MnO	0.02
MgO	1.00
CaO	1.52
Na_2O	0.21
K_2O	2.74
P_2O_5	0.18
Totals	96.55
LOI	3.32

Depending on the physical characteristics and chemical composition of the ash, it can be classified by the provisions of ASTM C 618: Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use in Concrete (ASTM 2015). Based upon these characteristics, two classifications for fly ash used in concrete are Class C and Class F. The most significant difference between the two ashes is the content of calcium oxide, or lime. Class C fly ash normally has twenty percent or more lime content by mass and is considered a high calcium fly ash. Because the lime is hydraulically active, Class C ash may develop cementitious characteristics through hydration reactions. The Class F fly ash is obtained from the burning of bituminous coal consisting of alumino-silcate glass, quartz, mullite, and magnetite. Class F ashes generally have less than ten percent lime and are considered a low calcium fly ash (American Coal Ash Association 2003). Both Class C and Class F fly ash can be used as mineral admixtures in a concrete mix design. Tables A.3 through A.5 present additional chemical, physical and reactivity requirements specified in ASTM C618 for classified ashes.

Table A.3: Chemical requirements for Class C and F fly ash (ASTM 2015)

Requirement	Class	
	F	C
Silicon dioxide (SiO_2) plus aluminum oxide (Al_2O_3) plus iron oxide (Fe_2O_3), min, %	70.0	50.0
Sulfur trioxide (SO_3), max, %	5.0	5.0
Moisture content, max, %	3.0	3.0
Loss on ignition, max, %	6.0*	6.0*

* NCDOT limits loss on ignition to 4%.

Table A.4: Physical requirements for Class C and F fly ash (ASTM 2015)

Requirement	Class	
Fineness:	F	C
Amount retained when wet-sieved on 45 µm (No. 325) sieve, max, %	34	34
Strength activity index: [A]		
With portland cement, at 7 days, min, percent of control	75	75
With portland cement, at 28 days, min, percent of control	75	75
Water requirement, max, percent of control	105	105
Soundness: [C]		
Autoclave expansion or contraction, max, %	0.8	0.8
Uniformity requirements:		
The density and fineness of individual samples shall not vary from the average established by the ten preceding tests, or by all preceding tests if the number is less than ten, by more than:		
Density, max variation from average, %	5	5
Percent retained on 45-µm (No. 325), max variation, percentage points from average	5	5

Table A.5: Additional requirements for Class C and F fly ash (ASTM 2015)

Requirement	Class	
	F	C
Increase of drying shrinkage of mortar bars at 28 days, max, difference, in %, over control[A]	0.03	0.03
Uniformity Requirements:		
In addition, when air-entraining concrete is specified, the quantity of air-entraining agent required to produce an air content of 18.0 vol % of mortar shall not vary from the average established by the ten preceding tests or by all preceding tests if less than ten, by more than, %	20	20
Effectiveness in Controlling Alkali-Silica Reaction:		
Expansion of test mixture as percentage of low-alkali cement control, at 14 days, max, %	100	100
Effectiveness in Contributing to Sulfate Resistance:		
Procedure A:		
Expansion of test mixture:		
For moderate sulfate exposure after 6 months exposure, max, %	0.10	0.10
For high sulfate exposure after 6 months exposure, max, %	0.05	0.05
Procedure B:		
Expansion of test mixture as a percentage of sulfate resistance cement control after at least 6 months exposure, max,%	100	100

Adding fly ash to concrete has been undertaken for many engineering reasons, and therefore, there is no encompassing guidance or standard practice. Generally, ash may be included to replace 10-40% of the portland cement in a batch of concrete. However, as the quantity of ash increases, the potential impacts to concrete properties are more substantially expressed. A typical replacement range is 15-25%, by weight. Because of the differences in specific gravity, and therefore, material density, a replacement rate of 1.2-1.5:1 may be used. NCDOT specifications allow 20% replacement rate with 1.2 lb of fly ash replacing 1 lb of portland cement (NCDOT 2012). Typical mixture design processes are used to develop proportions. As with mixtures that do not contain SCMs physical testing of trial batches is essential to accurately and economically specifying proportions.

Adding fly ash is associated with several benefits to concrete performance. Many of these benefits are related to the morphology of the ash particles. The cenospheres that comprise the ash add a substantial amount of very fine and rounded particles to the mixture. These affect workability by reducing water demand to achieve particular slump targets (Langan et al. 1990). This improved workability is also associated with enhanced pumpability and finishability. The additional fine material tends to block bleed water from traveling to the upper surface of the concrete during finishing and before initial set.

The chemical composition of the ash impacts the hydration processes of concrete (American Coal Ash Association 2003; Thomas et al. 2007). Typically, set times are found to be delayed as a result of adding fly ash. Because fly ash reduces the initial reaction rate, the heat of hydration is also typically lower for fly ash concrete. Strength development may also be slower than in equivalently treated mixtures with no preplacement of portland cement by fly ash. However, after long periods of curing, the gain of compressive and tensile strength may be greater in mixtures that contain fly ash (Neville 2011).

The primary reaction that leads to the development of additional compressive and tensile strength in mixtures that contain fly ash is between the hydration product, calcium hydroxide, and alumina-silicate material in the pozzolan. This reaction consumes the calcium hydroxide and generates more of the cementing agent, calcium-silicate-hydrate (C-S-H). This pozzolanic reaction can continue until there is very little calcium hydroxide left (Smith 1984). The benefits of reducing the quantity of calcium hydroxide are exhibited through enhanced durability. Because of its high solubility in water, calcium hydroxide leaves voids in the concrete as it is removed from the matrix by dissolution. This reduces strength and increase permeability (Smith 1984). The reduced permeability of fly ash containing concrete is beneficial to reducing the ingress of water that is laden with deleterious materials, such as sulfates and chlorides (American Coal Ash Association 2003). The pozzolanic reaction also reduces the availability of calcium hydroxide for participation in alkali silica reactions.

A.2.5.1.4 Portland Limestone Cement

Portland limestone cement is a blended cement which contains some proportion of ground limestone in addition to the typical clinker. Several benefits are associated with reducing the quantity of ground clinker in cement blends, such as lowered cost, reduced heat of hydration and lowered environmental impacts. Unlike the additional cementation that is associated with blending pozzolonic materials into cement, adding limestone primarily provides mechanical improvements during the clinker grinding process and during mixing and consolidation through better particle packing. Although limestone can be added separately to portland cement, PLCs are typically commercially produced by intergrinding limestone with the clinker. This process improves the particle size distribution of the blended cement. A typical limestone particle size distribution is shown in Figure A.2 (from Tennis et al. 2011).

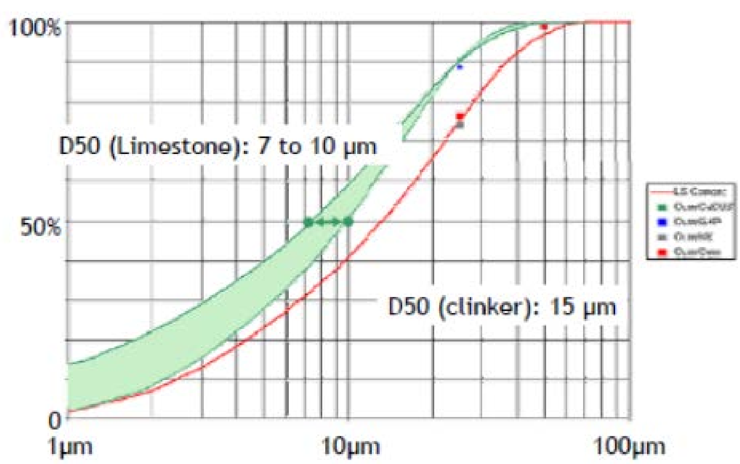

Figure A.2: Limestone particle size distribution (Tennis et al. 2011)

The environmental benefits to using PLC are substantial. The production of portland cement clinker is a key contributor to global greenhouse gas emissions. The carbon footprint of concrete is strongly linked to the calcination process that is inherent to clinker production. Because the limestone is minimally processed and is not calcined, intergrinding limestone with clinker results in a lower carbon footprint by way of reduced fuel consumption and avoidance of calcination-linked emissions (Tennis et al. 2011). Research into the use of PLC in conjunction with other supplementary cementitious materials (SCMs) has shown even more benefits related to sustainability. In a November 2012 presentation in a Transportation Research Board (TRB) sponsored webinar on PLC concrete, "Performance of PLC Concrete: Fresh, Hardened and Durability Properties," Michael D.A. Thomas of the University of New Brunswick (a leading PLC concrete researcher in Canada) reported that using a combination of PLC or blended PLC together with SCM additions at the concrete plant provides the opportunity to reduce the clinker content of paving mixes by up to 50%. The total impact of using these blends can translate to CO_2 reductions of the order of 1 to 1½ tons per truckload of concrete (Tennis et al 2011).

The limestone can improve the particle size distribution of the cement due to the fact that the limestone has a tendency to be softer than the clinker and grinds to a finer powder when inter-ground into the clinker (Tennis et al. 2011). The better particle size distribution from the inter-grinding of the limestone and clinker comes from the limestone making up the majority of the smaller particle sizes that range from 7 to 10 µm, and the clinker particle sizes being closer to 15 µm which means the concrete will exhibit a lower water demand (Tennis et al. 2011).

Limestone interground with OPC contributes to the hydration process through particle packing effects, nucleation effects, and chemical reactions (Tennis et al. 2011). During hydration, the presence of limestone particles offers several physical and chemical mechanisms to affect the development of the hardened microstructure. In the fresh state, the addition of limestone powder leads to a dilution of cement grains in a given volume of paste. However, due to particle packing, it also reduces the quantity of water in that same volume, which raises the effective *w/c* ratio and enables increased hydration. Further, the PSD of the powder fraction is smoother and contains few missing diameters. Therefore, the hydration products contain fewer pores to fill as they grow. This ultimately leads to increased strength and durability performance (Tennis et al. 2011). The improved particle size distribution allows for smaller limestone particles to intersperse in the void space between larger grains of cement and limestone. The limestone particles provide an increased number of nucleation sites. These additional surfaces for precipitation of hydration products, speed up hydration reactions, and result in higher early age strengths (Thomas and Hooton 2010). The presence of very fine limestone in the void spaces also tends to decrease water demand (Hawkins et al. 2003). The chemical composition of the limestone facilitates increased reaction with calcium aluminates in the OPC or SCM, forming calcium carboaluminates (Tennis et al. 2011).

<u>Experience with PLC in the Field</u>

Information on the performance of field installations of PLC concrete is becoming more readily available as the number of pilot projects increases, and as early pilot projects have been in service for several years. PLC concrete mixtures in Canada have been extensively studied, and a number of successful field trials have demonstrated suitable performance in pavements and other applications (Thomas et al. 2010a and 2010b). Data on the performance of field trials in Canada are available for privately-owned pavements at cement plants in the Canadian Provinces of Alberta, Quebec, and Nova Scotia (Thomas et al. 2010a and 2010b). PLC concrete mixtures used in a trial pavement in Gatineau, Quebec exhibited similar or slightly improved resistance to chloride permeability when compared to companion OPC mixtures (Tennis et al. 2011). Thomas reported that, based on the Canadian laboratory research and field studies, Portland-limestone cement (PLC) with 12% limestone, when optimized for equal strength, can provide equivalent performance to Portland cement.

PLCs have been successfully utilized in Europe for over 25 years (Hooton et al. 2007), and are being increasingly utilized in Canada, where PLCs with up to 15% limestone are allowed in all applications with the exception of sulfate-exposed applications (Thomas et al. 2010). International and domestic experience have resulted in several standards and specifications to govern the application of PLC. Currently, interground limestone is limited to 15% in Canada by Canadian Standard CSA A3001, although 35% interground limestone is permitted by European standard EN 197-1 (British Standards Institution 2000, European Committee for Standardization 2000, Canadian Standards Association 2013,). In a 2008 paper in Cement and Concrete Research, "Bridging the gap between research and standards," Hooton and others provided guidance in implementation of new Canadian standards based on research with Canadian materials. The 2010 National Building Code of Canada (NBCC) provides provisions for use of PLC, following the inclusion in CSA standard A3001-08, "Cementitious Materials Compendium" and CSA A23.1 "Concrete Materials and Methods of Concrete Construction" standard (Canadian Commission on Building and Fire Codes 2010).

The trend towards specifications allowing PLC concrete to be utilized in transportation applications is moving rapidly through the United States, with many state agencies either allowing or considering allowing PLC via provisions in specifications. In the United States, OPC may contain up to 5% limestone without additional labeling requirements. In cases that exceed this threshold, US specifications for PLC are outlined in AASHTO M 240 and ASTM C595, "Standard Specification for Blended Hydraulic Cements" (AASHTO 2017, ASTM 2015). These documents outline the Type IL Portland-limestone blended cement, which includes a designation for 10% limestone replacement, (e.g. Type IL(10) is PLC with a 10% limestone replacement). Requirements for the composition of limestone in binary and ternary blends including slag, fly ash, and other SCMs, are also provided. A summary of the physical and performance requirements for Type IL PLC that are outlined in ASTM C595 are presented in Table A.6.

Table A.6: Physical and performance requirements for Type IL cements

Property	Specification
Autoclave expansion, max, %	0.80
Autoclave contraction, max, %	0.20
Time of initial setting, Vicat test:	
Set, minutes, not less than Set, hours, not more than	45 7
Air content of mortar, volume %, max	12
Compressive strength, min, MPa [psi]:	
3 days 7 days 28 days	13.0 [1890] 20.0 [2900] 25.0 [3620]

Adoption of PLC in the US

In the United States, pilot projects using PLC concrete have been implemented in Utah and Colorado, with over 125 miles of PLC concrete pavement in place in these two states as of Fall 2012 (Laker and Smartz 2012). Utah DOT pilot projects include highway pavements in metropolitan areas (SR 20 and 104th South in Salt Lake City and UTA FrontRunner South from Salt Lake City to Provo) and rural county roads (Laker and Smartz 2012). Additionally, Utah DOT has experience in use of PLC in a segmental block retaining wall. Colorado DOT has utilized PLC concrete in a number of residential street projects and arterial roadway projects in Denver, as well as in highway pavements such as I-25 near Castle Rock and US 287 near Lamar (Laker and Smartz 2012).

Some US states have written specifications allowing use of PLC. Colorado DOT and Utah DOT currently allow PLC that meet ASTM C1157 performance specifications for GU (General Use), MS (Moderate Sulfate Resistance) and HS (High Sulfate Resistance) (ASTM 2011). Specifications for these two state agencies also require inclusion of SCMs in concrete mixtures used in applications that could be susceptible to alkali-silica reactivity (ASR) and/or sulfate attack (Laker and Smartz 2012). Louisiana DOTD currently allows use of PLC in concrete, and has published Special Provision HGR 05-07-13 for use on pilot concrete pavement projects to test new standard specifications. In Section 001.08.1 Cement of this Special Provision, "Type IL portland limestone cement" is listed as an allowable cement type for General Construction (Structural Class Concrete and Minor Structure Class Concrete), Concrete Pavement, and Prestressed or Precast Concrete. Oklahoma, Utah, Iowa, Missouri, and Louisiana also currently allow the use of PLCs, and it appears that additional states are either considering or planning on allowing use of PLC in the near future (Rupnow and Icenogle 2015).

NCDOT Standard Specifications permit the use of PLC with interground limestone at proportions up to 12% in type IL cement that meets AASHTO M240 requirements, fly ash up to 30% of cement content, and potentially greater amounts of both materials in Type IT cements. Thus, under the current prescriptive specification, a cement blend could contain only 58% portland cement and 42% of supplementary materials (30% fly ash and 12% interground limestone).

Performance Observations

Several field studies indicate that PLC concrete can exhibit performance similar to concrete produced with OPC. Laker and Smartz (2012) indicated that field experience on Colorado DOT and Utah DOT projects, PLC concrete shows strength gain, set time, water demand and compatibility with fly ash and admixtures that was similar to OPC concrete. However, Hawkins et al. (2003) and Vuk et al. (2001) have also found conditions in which the Vicat set times for PLC concretes are shortened by up to an hour. The reduced set times are linked to the increased availability of nucleation sites and smaller capillary pore diameters in PLC paste. For similar reasons, the addition of limestone has also been shown to increase early compressive strength as long as a lower percentage of limestone is added <8% replacement of cement and the limestone is ground finer than the cement (Tennis et al. 2011). When levels of limestone exceed 15%, it has been shown to have a negative impact on the compressive strength (Tennis et al. 2011).

Studies performed in the early 2000s by Dhir et al. (2007) on PLC concrete produced in the United Kingdom indicated that the permeability and durability of PLC, including initial surface absorption, carbonation resistance, chloride diffusion, freeze/thaw scaling and abrasion resistance, generally followed proportional relationships with strength for most properties. Extensive research into the durability performance of PLC Concrete has been conducted in Canada by Thomas and Hooton (2010). PLC with interground limestone replacements ranging from 3% to 19% were utilized in concrete mixtures with Canadian aggregates. Samples were prepared with varying *w/cm* ratios and a range of cement contents. Testing was performed to evaluate fresh properties, early age properties, and durability performance, along with methods to evaluate rapid chloride permeability, freeze/thaw resistance, salt scaling, shrinkage, sulfate resistance, and ASR expansion. This study concluded that for PLC with up to 15% limestone, equivalent performance as OPC from the same clinker was observed (Thomas and Hooton 2010). In tests performed for Colorado DOT and Utah DOT, Laker and Smartz (2012) indicate that in some tests for alkali-silica reactivity (ASR) (ASTM C1260 and ASTM C1567) and rapid chloride ion permeability (ASTM C1202), PLC concrete mixtures performed better (or slightly better) than OPC concrete mixtures.

Many durability characteristics of concrete are related to the permeability of the matrix. Tsivilis et al. (2003) conducted a study on the permeability of PLC concretes and found that the gas permeability of the concrete increased compared to ordinary portland cement concrete, while sorptivity and liquid permeability decreased with the addition of ground limestone. In this study, researchers analyzed concrete produced using limestone replacement amounts of 15% to 35%, showing that the increase in limestone resulted in a trend of increasing porosity. This complex relationship is due to the fact that permeability of concrete is a function of both porosity and the size, distribution, tortuosity, and continuity of the pores network. Gas permeability is more closely correlated to the overall porosity, while liquid permeability is affected by the size and kinds of pores in the concrete (Tsivilis et al. 2003). Ground limestone has a much smaller particle size, 7 to 10μm, which in turn increases the particle packing density and results in smaller capillary pores. As a result, permeability is reduced (Githachuri and Alexander 2013). The packing effect can be increased based on the how finely the limestone is ground. The limestone can range from a course limestone particle around 100 microns and down to the fine particle size of around 0.3 microns or lower for the powder limestone (Hooton et al., 2007).

The impact of limestone powder on the microstructure is complex. Mixtures with greater amounts of limesone addition without adjustment of water, also have a higher effective *w/c* ratio. This may lead to greater hydration, but also can create more capilary pores. Arora et al. (2016) added PLC and slag to concrete and found that the addition of the limestone gave the mixture a better particle packing. However, the group also discovered that both the pore size and porsity of the concrete mixtures was lower in the control OPC mixture than in the PLC-slag mixture at 28 days. However, the pore sizes were smaller and below a critical diameter that would impact durability. It was also found that the PLC-slag mixtures had evidence of pore size refinement by decreasing pore size with increasing levels of cement replacement due to pozzolanic activity.

There is a possibility that PLC concrete can be more susceptible to the thaumasite form of sulfate attack (TSA) due to the presence of carbonate ions that are present in the limestone. Wet, sulfate-rich conditions are required for TSA to occur and colder climates can increase the possibility of it initiating (Hartshorn et al. 1999). Similar to sulfate attack in warmer climates, damage originates from the formation of expansive thaumasite crystals, which are similar to ettringite, inside of a hardened concrete matrix. After the hardening of the concrete, an external sulfate source, such as from groundwater or seawater, can facilitate a reaction with carbonate ions to break down C-S-H and form thaumasite. This reaction leads to change in pH (lowers pH) and expansion of concrete takes place (Ramezanianpour 2012). Wet, sulfate-rich conditions are required for TSA to occur and colder climates can increase the possibility of it initiating. Hooton and Thomas (2002) studied the susceptibility of Canadian PLC concrete to TSA and found that there were no cases of TSA related to use of PLC in concrete in Canada.

The most commonly utilized test for TSA is CSA A3004-08 Procedure B, which is a modified version of the ASTM C1012 sulfate resistance test that is performed at low temperatures (5°C). A key study was conducted by University of Toronto on TSA in portland cement and PLC mortars exposed to sulfate solution. The mortar bars for this study were placed in two different temperatures, 5°C and 23°C. This study concluded that mortars containing PLC were more prone to sulfate attack at lower temperatures (such as 5°C) than mortars containing OPC. The initial expansion in mortar bars was due to ettringite formation and gypsum present in cement preceded thaumasite formation. Formation of the thaumasite crystals was confirmed by X-ray diffraction, and use of slag was effective in enhancing the sulfate resistance (Ramezanianpour and Hooton 2013a and 2013b).

Recently, leading researchers have criticized this test, calling it "overly severe" due to the potential for PLC and SCM combinations with a satisfactory history of field performance failing the test, conflicting results where mixtures with PLC perform better than non-PLC mixtures, and the "low maturity" of specimens when immersed in the sulfate solution at a low temperature (which stifles hydration reactions, particularly of SCMs). These researchers recommend modifying CSA A3004-08 Procedure B to "ensure sufficient hydration maturity" of the samples before subjecting them to the test (Barcelo et al. 2014)

A.2.5.1.5 Utilizing Portland Limestone Cement with Fly Ash

Ternary cement blends contain more than one mineral addition to the ground clinker. Often these additions may be fly ash and limestone powder. Yilmaz and Olgun (2008) conducted a study of cements and mortars containing fly ash and limestone replacements. Building upon previous research indicating that concrete with PLC cements can exhibit higher early strength compared to concrete with OPC, additional research was performed to include fly ash, which is well known to provide increases in later age strength. The materials used in the Yilmaz study were OPC, fly ash (Class F), limestone and dolomitic limestone. In this study, replacement rates of 5-40% fly ash, 5-15% limestone, and 5-15% dolomitic limestone were utilized in the concrete mixtures. The researchers concluded that an observed increase in the early strength of the concrete was an effect of the limestone having an active role in the hydration process. Findings from the study indicated that the addition of limestone to the mixtures helped increase the early compressive strengths compared to mixtures containing only fly ash (Yilmaz and Olgun 2008).

Other studies have confirmed that the limestone tends to increase the early age strength of the concrete and can be used to offset the delayed strength gain caused by the addition of fly ash. Yoshitake et al. (2013) prepared concrete specimens using Class F fly ash and belended limestone. The specimens were tested for uniaxial tensile and flexural strengths at 1, 2, 3, 5, 7 and 28 days. Results indicated that limestone fillers increase the strength at an early age even in high volume fly ash concrete (Yoshitake et al. 2013).

In summary, it has been demonstrated that use of PLC can support an increase in early age strength in concrete, and can be used to offset the delayed early age strength gain resulting from use of fly ash. The fly ash will allow these mixtures to continue to gain strength at later ages, so these ternary blends can provide a number of benefits associated with both strength and durability. Future research on concretes containing fly ash and limestone cements has been suggested to explore benefits associated with lower temperature rise, lower permeability, and a larger carbonation depth than pure cement mixtures (Jin and Mengyuan 2014).

A.2.5.2 Aggregates

The types of aggregates available in North Carolina vary greatly by region of the state. This is evident in Figure A.3, 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). As shown in Figure A.3, 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 primarily 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 areas 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.

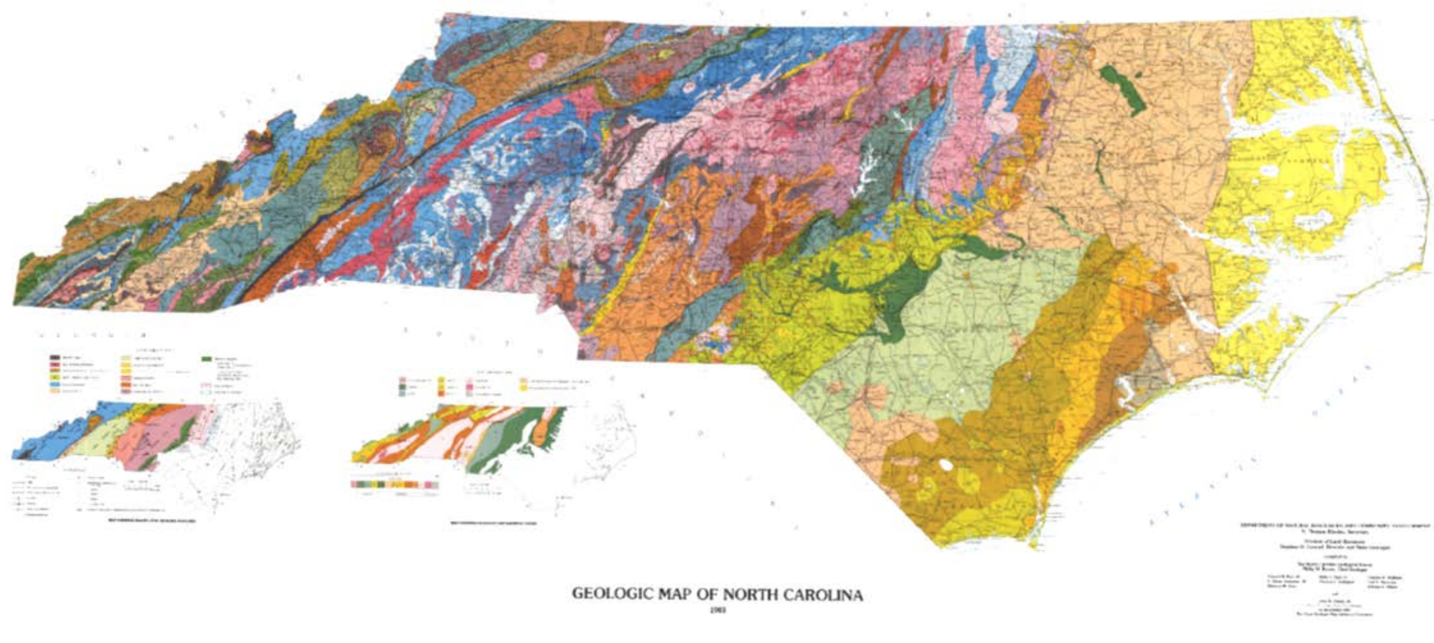

Figure A.3: Geological survey of North Carolina (NCDEQ 2015)

A.3 Sustainability and Lifecycle Assessment

Sustainable development can be defined as development where the needs of the present are met without compromising the ability of future generations to also meet their needs (WCED 1987). A pavement can be considered sustainable if it meets engineering goals, human needs, and will preserve the surrounding environment using financial, human, and environmental resources efficiently (WCED 1987). Portland cement production is a contributor to greenhouse gas (GHG) emissions, associated with both fossil fuel use and emissions associated with the calcination process. Concrete transportation infrastructure, including pavements, utilizes a large amount of portland cement, thus contributing to the release of those gases into the environment (Thomas et al. 2015).

The sustainability performance of a pavement, or of pavement designs under consideration, can be quantified by different methods. The four most preferred methods for measuring sustainability are performance assessment, life cycle cost analysis (LCCA), life cycle assessment (LCA), and sustainability rating systems (Van Dam et al. 2015). LCA is a technique which can be used to quantify environmental impacts of the pavement, whereas LCCA is a technique in which all the costs associated with each alternative are analyzed over the desired timeframe, but environmental and social impacts are not specifically addressed. Sustainability rating systems are often used to compare and contrast projects based on a scoring system, and ultimately provide a level of recognition for the stakeholders (Van Dam et al. 2015). Therefore, when interested in evaluating the environmental impacts of construction and use of pavements (and other types of infrastructure), LCA is the appropriate tool providing a quantitative approach to compare alternatives. Utilizing an LCA to evaluate pavement alternatives and to guide construction decisions can aid in reducing waste, GHG emissions, and usage of natural resources (Harvey et al. 2014).

The LCA process provides a means of assessment aimed at quantifying the materials and energy input and output flows of a project to assess its impacts on environment (Harvey et al. 2014). An LCA can be used to help in selecting the "best" option among different alternatives for the same project and can aid an agency in improving the environmental performance of a pavement over its complete life span. This process generally includes means of assessing the relative impacts of a project related to social, economic, and environmental factors. Indicators such as income, government tax, and injury are categorized under "social indicators" while GHG emissions, energy consumption, water footprint, and hazardous waste generation are categorized under "environmental indicators." Economic indicators include foreign purchase, business profit, and gross domestic product.

Early developers of the LCA aimed to incorporate the analysis of the three main elements of earth (air, land, and water) into the analysis, as these each are subjected to degradation due to human impacts (Harvey et al. 2014). By the start of the new millennium in 2000, LCA was typically broadened to include energy, use of available resources, and GHG emissions (Harvey et al. 2014). In recent years, the LCA process has been standardized by the International Standardization Organization (ISO) standardized assessment methods, and is detailed in ISO 14040 and ISO 14044 (ISO 2006). Some of the key steps in performing an LCA include identification of the required data, standardization of data collected, and updating and understanding impact assessment methodology (Muench et al. 2012). Per ISO 14040 LCA is divided into three important phases: goal and scope, life cycle inventory assessment, and impact assessment (ISO 2006).

Due to the detailed nature of the information required to support this type of analysis, LCA results are generally project specific and hence cannot be generalized for all the pavement projects around the country (Van Dam et al. 2015). By utilizing LCA, however, an agency can become aware of the impact of a project on the surrounding environment, compare alternatives, and make design and construction decisions aimed at lessening a project's impact on the existing environment. For example, LCA of a pavement evaluates the impact of construction of pavement on the environment while also considering factors such as raw material production, impact of the construction phase, impacts during use of the pavement, and the impact of the end use of the pavement. Therefore, the results of an LCA can be used to guide decisions impacting each of these areas during the pavements service life, as well as provide a tool to guide initial decision making during design.

Recently, research supported by the FHWA resulted in development of an LCA framework specifically for pavements. The interested reader is encouraged to review this recently published report (Pavement Life-Cycle Assessment Framework, Report No. FHWA-HIF-16-014), which "is an important first step in the implementation and adoption of principles in the pavement community within the United States (Harvey et al. 2016)." The product of years of development by renowned sustainability and pavements experts, this document provides "a framework for performing an LCA specific to pavement systems along with guidance on the overall approach, methodology, system boundaries, and current knowledge gaps (Harvey et al. 2016)." It is noted, however, that this framework was not available for use at the time of this study.

To date, LCA has been shown to be a valuable tool to allow stakeholders to quantify the environmental impact of cement and concrete production, as well as projects containing concrete. For example, from a study performed by the

Athena Sustainable Materials Institute for the Cement Association of Canada, it was observed that addition of limestone in portland cement reduces greenhouse emissions by about 9.6% (Athena 2005). This study also showed that production of PLC not only reduces greenhouse emissions but also supports improved industry performance across other environmental impact metrics, including reductions in ozone depletion potential and lower smog potential. Researchers suggested that agencies in the United States support the increase in the allowable percentage of limestone in cement from 15% to 35%. This allowable percentage of limestone inclusion would be similar to European standards, promoting reduction in environmental impacts. This recommended increase would be a marked change in American standards, which prior to the study, had restricted limestone content to up to 5%, primarily citing perceived reductions in the strength of the concrete as the reason for this relatively low limit (Athena 2005).

More recently, several research studies at the Massachusetts Institute of Technology (MIT) have focused on LCA specifically for pavements. In a study published in 2011 (Santero et al. 2011), twelve concrete pavements serving a range of uses (from rural roads to urban interstates) were considered. Pavements in the study were designed using 1993 AASHTO Design Method for design of pavements. For each of the pavements, the global warming potential (GWP) for each phase of pavement lifecycle was determined. Results from this study showed that the GWP of concrete pavements ranged from 600 tons CO_2 per mile (for rural roads) to 11,000 tons of CO_2 per mile for urban interstates per year. The production phase for most of the pavements constituted a large portion of overall GHG emissions, as cement production was associated with 45% of GHG emissions for urban interstates and 72% of GHG emissions for rural roads. Another important contributor of GWP for all pavements was fuel consumption, which is linked to roughness of pavements. Findings from this study showed that addition of fly ash (at replacement rates of about 10% to 30%) will reduce GWP of about 15% for urban interstates and 36% for rural roads. According to Santero et al. (2011), emissions due to rehabilitation activities are greater than fuel consumption due to roughness of roads if the traffic on road is less than approximately 2,500 vehicles per day. Hence rehabilitation strategies may increase GWP of rural roads by 10% and reduce about 13% for urban interstates. This study also showed that GWP in rural roads can be reduced up to 17% by using AASHTO M-EPDG for design, rather than the 1993 AASHTO Design Method. Santero et al. (2011) suggest that by using the strategies identified, GWP can be reduced to about 38% for urban interstates and 58% for urban roads.

In these times of increased effort to mitigate the environmental impact of infrastructure, as well as to responsibly utilize the limited amount of funds available for infrastructure maintenance and construction, there is great need for research to aid in decisions regarding pavement design, construction, and maintenance. To date, the cement used in rigid pavements in North Carolina is OPC. Considering the substantial amount of transportation infrastructure projects pending in North Carolina, an alternative to OPC that reduces environmental impact could be welcomed if acceptable performance is confirmed. As outlined in this literature review, the findings of studies in other countries and in the United States indicate that PLC requires less clinker for production of cement, energy can be preserved, raw materials can be saved, and fuel use can be reduced. It has also been shown that PLC concrete can provide equivalent performance to OPC concrete (Rupnow and Icenogle 2015), although this has not been confirmed using materials locally available for concrete produced in North Carolina. As shown by Santero et al. (2011), use of M-EPDG (presumably with local calibration) can also aid in improving the sustainability of pavements.

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APPENDIX B – SUPPORTING MATERIAL FOR LABORATORY TESTING (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 (SiO_2), %	Minimum	----	----	----	20.3
Aluminum Oxide (Al_2O_3), %	Maximum	----	6.0	----	4.7
Ferric Oxide (Fe_2O_3), %	Maximum	----	6.0	----	3.3
Calcium Oxide (CaO), %		----	----	----	64.1
Magnesium Oxide (MgO), %	Maximum	6.0	6.0	6.0	1.2
Sulfur Trioxide (SO_3), % **	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 (Na_2O equivalent), %	Maximum			0.60	0.54
Tricalcium Silicate (C_3S), %	Maximum	----	----	----	58
Tricalcium Aluminate (C_3A), %	Maximum	----	8	----	7
C_3S + 4.75(C_3A), %	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 SO_3	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 B.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 B.2: Mill report for OPC2 and PLC

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

REPORT OF FLY ASH TESTS			
Date Sampled: DS 11/23-12/11	Start Date:	November 23, 2015	
Manufacturer: Roxboro	End Date:	December 11, 2015	
	Date Received:	December 16, 2015	

Chemical Analysis**	Results (wt%)	Specification (Class F) ASTM C618-15	Specification (Class F) 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 B.3: Fly ash A 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

Figure B.4: Fly ash B report

Table B.1: Sieve analysis for Mountain coarse aggregate

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 B.2: Sieve analysis for Piedmont coarse aggregate

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 B.3: Sieve analysis for Coastal coarse aggregate

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 B.4: Sieve analysis for manufactured sand

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 B.5: Sieve analysis for natural sand

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%

Table B.6: Slump test results for each batch

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

Table B.7: Air content test results (ASTM C231) for each batch

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

Table B.8: Results of unit weight test for each batch

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

Table B.9: SAM test results for selected batches (batch 2)

Mixture	SAM 1 air content (%)	SAM 1 SAM number	SAM 2 air content (%)	SAM 2 SAM number	Average SAM air content (%)	Average SAM Number	C231 Air Content (%)
C.A.N.M	7.0	-	7.0	-	7.0	-	5.7
M.A.N.M	-	-	-	-	-		5.3
P.A.N.M	6.2	0.20	6.1	0.18	6.2	0.19	5.5
P.A.N.N	5.7	-	5.4	0.10	5.6	0.10	5.0
P.A.A.M	7.4	-	7.0	-	7.2	-	5.9
P.A.B.M	7.0	0.34	6.0	0.23	6.5	0.27	5.2
C.B.N.M	7.0	0.53	7.0	0.17	7.0	0.17	5.4
M.B.N.M	-	-	-	-	-	-	-
P.B.N.M	6.9	0.28	6.8	0.17	6.9	0.23	6.0
P.B.N.N	5.4	0.36	5.1	0.18	5.3	0.27	5.1
P.B.A.M	6.0	0.28	6.1	-	6.1	0.28	5.3
P.B.B.M	7.2	0.22	7.2	0.22	7.2	0.22	6.0
C.BL.N.M	7.4	0.07	7.4	0.31	7.4	0.19	5.6
M.BL.N.M	-	-	-	-	-	-	-
P.BL.N.M	6.6	0.27	6.5	0.28	6.6	0.28	5.8
P.BL.N.N	7.0	0.19	7.0	0.18	7.0	0.19	5.9
P.BL.A.M	6.4	0.37	6.3	0.21	6.4	0.29	5.3
P.BL.B.M	7.0	0.20	7.0	0.18	7.0	0.19	5.9

Table B.10: 28-day compressive strength results for each batch

Mixture	28-day Compressive Strength (psi)			Average Compressive Strength (psi)	Standard Deviation
	1	2	3		
C.A.N.M	5,432	5,405	5,233	5,360	108
M.A.N.M	5,060	5,151	4,882	5,030	137
P.A.N.M	5,130	5,207	5,338	5,220	105
P.A.N.N	5,245	5,584	5,378	5,400	171
P.A.A.M	4,445	4,026	4,352	4,270	220
P.A.B.M	3,911	3,372	3,702	3,780	113
C.B.N.M	5,743	6,272	5,856	5,960	279
M.B.N.M	4,941	5,272	5,077	5,100	166
P.B.N.M	4,899	4,783	4,856	4,850	59
P.B.N.N	4,220	4,458	4,484	4,390	145
P.B.A.M	4,295	4,115	3,745	4,050	280
P.B.B.M	3,138	3,222	3,045	3,140	89
C.BL.N.M	5,405	5,295	5,969	5,560	362
M.BL.N.M	4,727	5,008	4,636	4,790	194
P.BL.N.M	4,781	5,011	5,264	5,020	242
P.BL.N.N	5,196	5,352	5,024	5,190	164
P.BL.A.M	3,693	3,915	3,635	3,750	148
P.BL.B.M	3,616	3,211	4,501	3,780	660

Table B.11: 28-day modulus of rupture test results

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

Table B.12: 28-day modulus of elasticity test results

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

Table B.13: Poisson’s ratio test results

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

Table B.14: CTE test results

Mixture	28-day CTE (x10^{-6} in/in^{o}F)			Average CTE (x10^{-6} in/in^{o}F)	Standard Deviation
	1	2	3		
C.A.N.M	4.25	4.09	4.36	4.23	0.14
M.A.N.M	4.36	4.43	4.59	4.46	0.12
P.A.N.M	4.51	4.52	4.68	4.57	0.10
P.A.N.N	5.42	5.40	5.38	5.40	0.02
P.A.A.M	4.39	4.44	4.43	4.42	0.02
P.A.B.M	4.42	4.45	4.43	4.43	0.02
C.B.N.M	4.24	4.11	4.48	4.28	0.19
M.B.N.M	4.49	4.49	4.72	4.57	0.13
P.B.N.M	4.56	4.56	4.78	4.63	0.13
P.B.N.N	5.31	5.29	5.33	5.31	0.02
P.B.A.M	4.47	4.46	4.45	4.46	0.01
P.B.B.M	4.45	4.59	4.51	4.52	0.07
C.BL.N.M	4.16	4.21	4.53	4.30	0.20
M.BL.N.M	4.48	4.43	4.77	4.56	0.18
P.BL.N.M	4.47	4.53	4.62	4.54	0.08
P.BL.N.N	5.22	5.54	5.20	5.32	0.19
P.BL.A.M	4.55	4.58	4.56	4.56	0.01
P.BL.B.M	4.63	4.54	4.51	4.56	0.06

Table B.15: Thermal conductivity test results (50% RH condition)

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

Table B.16: Heat capacity test results (50% RH condition)

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

Table B.17: Thermal conductivity and heat capacity test results (oven dried, 50% RH and SSD conditions)

Mixture	Oven Dried	50% RH	SSD	Oven Dried	50% RH	SSD
	Thermal Conductivity (BTU/(ft·hr·°F))	Thermal Conductivity (BTU/(ft·hr·°F))	Thermal Conductivity (BTU/(ft·hr·°F))	Heat Capacity (by weight) (BTU/lb·°F)	Heat Capacity (by weight) (BTU/lb·°F)	Heat Capacity (by weight) (BTU/lb·°F)
C.A.N.M	0.651	0.812	0.896	0.173	0.219	0.196
M.A.N.M	0.714	0.869	0.968	0.161	0.199	0.184
P.A.N.M	0.697	0.923	0.830	0.162	0.203	0.181
P.A.N.N	0.981	1.246	1.302	0.160	0.202	0.184
P.A.A.M	0.687	0.889	0.945	0.160	0.201	0.182
P.A.B.M	0.701	0.896	0.977	0.160	0.201	0.183
C.B.N.M	0.741	0.894	0.914	0.172	0.216	0.197
M.B.N.M	0.760	0.948	1.052	0.159	0.206	0.192
P.B.N.M	0.706	0.947	0.949	0.165	0.203	0.187
P.B.N.N	0.889	1.117	1.174	0.162	0.202	0.184
P.B.A.M	0.703	0.905	0.926	0.159	0.203	0.185
P.B.B.M	0.707	0.900	0.917	0.160	0.202	0.182
C.BL.N.M	0.695	0.867	0.888	0.166	0.202	0.186
M.BL.N.M	0.751	0.909	1.028	0.160	0.203	0.184
P.BL.N.M	0.631	0.803	0.971	0.157	0.202	0.180
P.BL.N.N	0.861	1.178	1.338	0.156	0.196	0.191
P.BL.A.M	0.722	0.903	1.000	0.161	0.203	0.192
P.BL.B.M	0.713	0.884	0.957	0.157	0.196	0.182

* Note: each measurement is the average of three tests.

Table B.18: Recommended (grouped by aggregate type and averaged) inputs for thermal conductivity and heat capacity (oven dried, 50% RH, and SSD conditions)

	OD	50% RH	SSD	OD	50% RH	SSD
	Thermal Conductivity (BTU/(ft·hr·°F))	Thermal Conductivity (BTU/(ft·hr·°F))	Thermal Conductivity (BTU/(ft·hr·°F))	Heat Capacity (by weight) (BTU/lb·°F)	Heat Capacity (by weight) (BTU/lb·°F)	Heat Capacity (by weight) (BTU/lb·°F)
C.A.N.M M.A.N.M P.A.N.M P.A.A.M P.A.B.M C.B.N.M M.B.N.M P.B.N.M P.B.A.M P.B.B.M C.BL.N.M M.BL.N.M P.BL.N.M P.BL.A.M P.BL.B.M	0.705	0.890	0.948	0.162	0.203	0.186
P.A.N.N P.B.N.N P.BL.N.N	0.910	1.181	1.271			

Table B.19: ASTM C157 shrinkage test results

	Length change due to shrinkage (%)			
Mixture ID	4 Weeks	8 Weeks	32 Weeks	64 Weeks
C.A.N.M	0.0266	0.0418	0.0518	0.0569
M.A.N.M	0.0290	0.0412	0.0512	0.0569
P.A.N.M	0.0300	0.0436	0.0533	0.0569
P.A.N.N	0.0263	0.0336	0.0448	0.0530
P.A.A.M	0.0281	0.0409	0.0457	0.0530
P.A.B.M	0.0254	0.0381	0.0454	0.0530
C.B.N.M	0.0287	0.0409	0.0472	0.0509
M.B.N.M	0.0318	0.043	0.0527	0.0572
P.B.N.M	0.0345	0.0466	0.0545	0.0600
P.B.N.N	0.0181	0.0245	0.0321	0.0433
P.B.A.M	0.0239	0.0333	0.0393	0.0533
P.B.B.M	0.0257	0.0357	0.0457	0.0539
C.BL.N.M	0.0284	0.0372	0.0433	0.0460
M.BL.N.M	0.0363	0.0487	0.0566	0.0624
P.BL.N.M	0.0384	0.0500	0.0575	0.0606
P.BL.N.N	0.0203	0.0272	0.0360	0.0460
P.BL.A.M	0.0303	0.0409	0.0478	0.0557
P.BL.B.M	0.0306	0.0415	0.0500	0.0584

Table B.20: ASTM C1581 cracking potential test results

Mixture I.D.	Ring #	Time to Cracking (days)	Rate of Strain (psi/day)	Avg. Mixture Rate of Strain (psi/day)	Stress Rate (psi/day)	Avg. Mixture Stress Rate (psi/day)
C.A.N.M	1	10.72	8.142E-06	6.922E-06	85.24	72.47
	2	N.C.	6.855E-06		71.77	
	3	N.C.	5.769E-06		60.40	
M.A.N.M	1	13.73	9.980E-06	6.870E-06	104.49	71.93
	2	13.74	6.491E-06		67.96	
	3	N.C.	4.140E-06		43.34	
P.A.N.M	1	6.00	4.350E-06	4.816E-06	45.54	50.42
	2	N.C.	6.119E-06		64.07	
	3	N.C.	3.978E-06		41.65	
P.A.N.N	1	24.25	5.168E-06	4.430E-06	54.11	46.38
	2	22.03	3.435E-06		35.96	
	3	N.C.	4.686E-06		49.06	
P.A.A.M	1	18.75	2.746E-06	3.117E-06	28.75	32.64
	2	18.86	4.923E-06		51.55	
	3	N.C.	1.682E-06		17.61	
P.A.B.M	1	-	-	-	-	-
	2	-	-		-	
	3	-	-		-	
C.B.N.M	1	7.11	1.715E-05	1.326E-05	179.59	138.88
	2	14.65	8.150E-06		85.33	
	3	10.05	1.449E-05		151.71	
M.B.N.M	1	N.C.	6.833E-06	5.439E-06	71.54	56.95
	2	N.C.	4.281E-06		44.83	
	3	6.01	5.204E-06		54.49	
P.B.N.M	1	N.C.	1.091E-06	2.696E-06	11.42	28.23
	2	N.C.	3.926E-06		41.10	
	3	N.C.	3.072E-06		32.16	
P.B.N.N	1	20.28	7.252E-06	4.388E-06	75.93	45.94
	2	22.60	1.612E-06		16.88	
	3	15.85	4.298E-06		45.00	
P.B.A.M	1	N.C.	4.092E-06	3.401E-06	42.85	35.60
	2	N.C.	1.613E-06		16.89	
	3	20.02	4.496E-06		47.08	
P.B.B.M	1	-	-	-	-	-
	2	-	-		-	
	3	-	-		-	
C.BL.N.M	1	14.78	1.027E-05	1.191E-05	107.50	124.65
	2	7.51	1.354E-05		141.80	
	3	-	-		-	
M.BL.N.M	1	N.C.	7.147E-06	6.782E-06	74.83	71.01
	2	N.C.	6.418E-06		67.20	
	3	-	-		-	
P.BL.N.M	1	N.C.	6.656E-06	6.015E-06	69.69	62.98
	2	N.C.	5.374E-06		56.26	
	3	-	-		-	
P.BL.N.N	1	22.33	4.137E-06	5.092E-06	43.31	53.31
	2	29.28	6.254E-06		65.48	
	3	19.52	4.885E-06		51.14	
P.BL.A.M	1	18.17	4.081E-06	3.563E-06	42.72	37.30
	2	N.C.	4.462E-06		46.72	
	3	25.01	2.145E-06		22.46	
P.BL.B.M	1	-	-	-	-	-
	2	-	-		-	
	3	-	-		-	

Note: N.C. = no crack observed during testing period.

Table B.21: Conventional sulfate attack test results (CSA A3004-C8 Procedure A, specimens stored at 23°C)

Mixture ID	Average Expansion (%)					
	2 weeks	3 Weeks	4 Weeks	8 Weeks	15 Weeks	24 Weeks
A.N	0.0021	0.0033	0.0057	0.0072	0.0093	0.0163
B.N	0.0012	0.0039	0.0054	0.0103	0.0136	0.0190
BL.N	0.0024	0.0033	0.0015	0.0075	0.0106	0.0178
A.N.N	0.0049	0.0073	0.0073	0.0128	0.0190	0.0364
B.N.N	0.0003	0.0027	0.0039	0.0072	0.0115	0.0160
BL.N.N	0.0021	0.0015	0.0027	0.0075	0.0157	0.0430
A.A	0.0018	0.0033	0.0030	0.0084	0.0100	0.0109
B.A	0.0021	0.0041	0.0053	0.0094	0.0129	0.0278
BL.A	0.0018	0.0033	0.0039	0.0072	0.0000	0.0145
A.B	0.0057	0.0087	0.0103	0.0181	0.0303	0.0584
B.B	0.0012	0.0027	0.0042	0.0090	0.0166	0.0387
BL.B	0.0018	0.0036	0.0060	0.0081	0.0115	0.0196

Table B.22: Thaumasite attack test results (CSA A3004-C8 Procedure B, specimens stored at 5°C)

Mixture ID	Average Expansion (%)						
	2 weeks	3 Weeks	4 Weeks	8 Weeks	15 Weeks	24 Weeks	52 Weeks
A.N	0.0000	0.0021	0.0021	0.0084	0.0112	0.0193	0.106
B.N	0.0009	0.0009	0.0042	0.0069	0.0090	0.0172	0.384
BL.N	0.0006	0.003	0.0051	0.0042	0.0078	0.0142	0.314
A.N.N	0.0049	0.0073	0.0073	0.0128	0.0190	0.0364	0.372
B.N.N	0.0003	0.0027	0.0039	0.0072	0.0115	0.0160	0.053
BL.N.N	0.0021	0.0015	0.0027	0.0075	0.0157	0.0430	0.345
A.A	0.0018	0.0033	0.003	0.0084	0.0100	0.0109	0.078
B.A	0.0003	0.0021	0.0024	0.0045	0.0063	0.0075	0.016
BL.A	0.0018	0.0033	0.0039	0.0072	0.0000	0.0145	0.024
A.B	0.0036	0.0048	0.0060	0.0157	0.0372	0.0945	0.16
B.B	0.0069	0.0045	0.0084	0.0118	0.0184	0.0369	0.15
BL.B	0.0024	0.0033	0.0054	0.0069	0.0151	0.0306	0.054

APPENDIX C: SUPPLEMENTAL INFORMATION FOR EVALUATION OF RESULTS (Chapter 4)

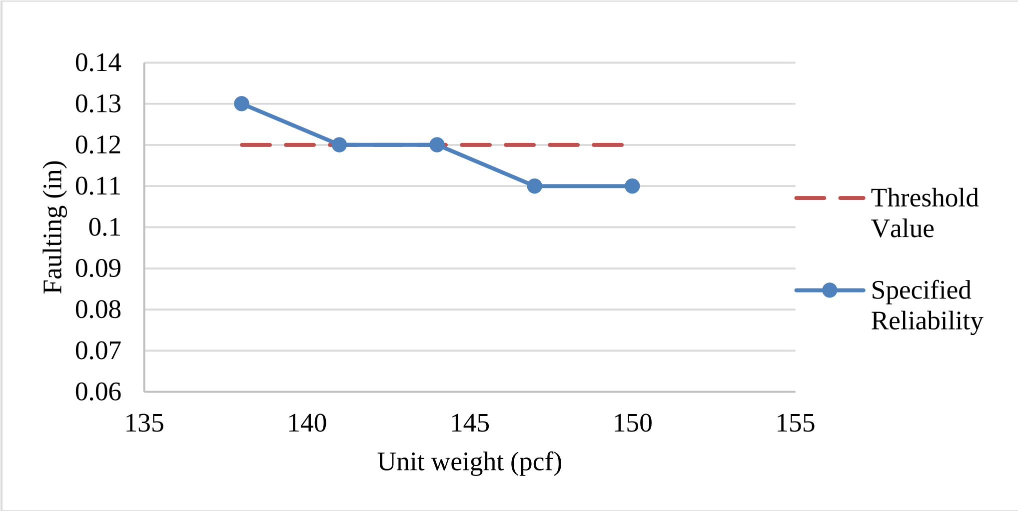

Figure C.1: Sensitivity of predicted faulting to changes in concrete unit weight

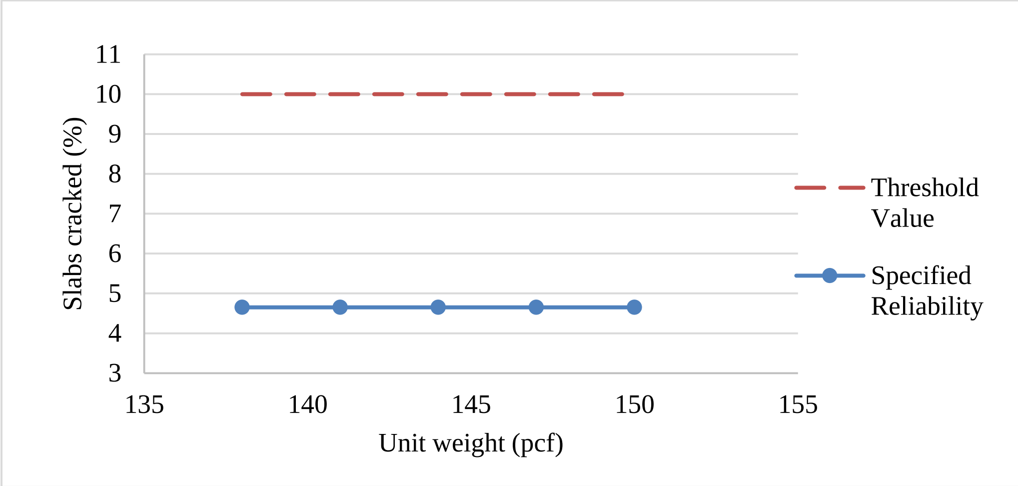

Figure C.2: Sensitivity of predicted slab cracking to changes in concrete unit weight

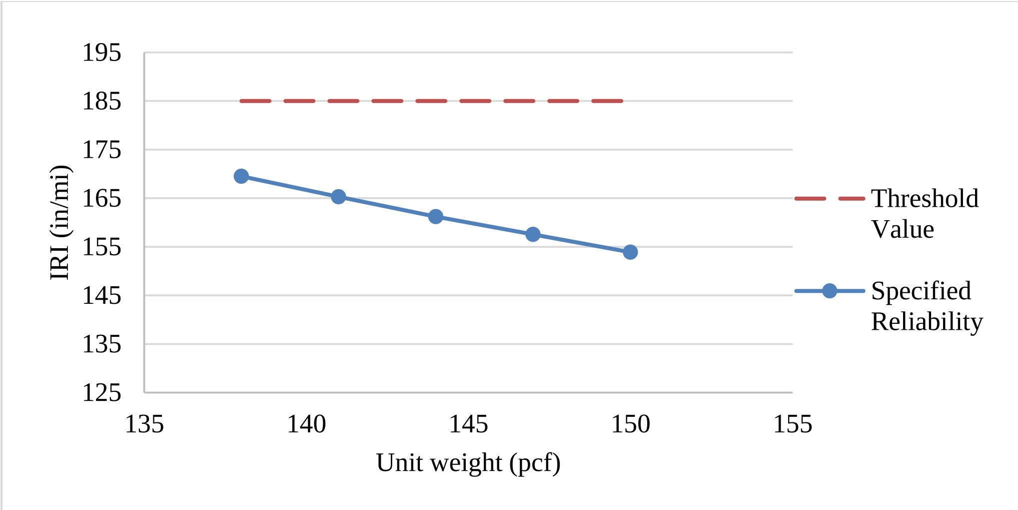

Figure C.3: Sensitivity of predicted IRI to changes in concrete unit weight

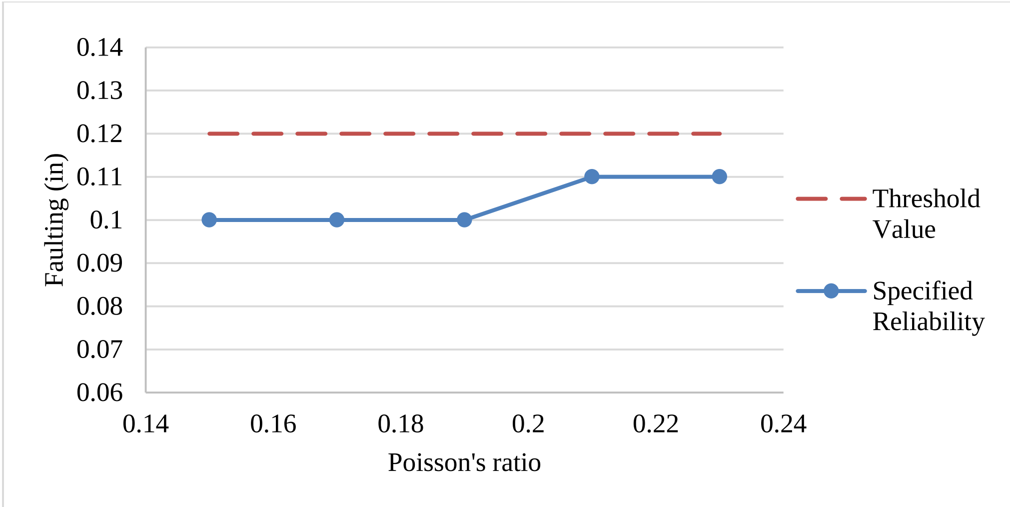

Figure C.4: Sensitivity of predicted faulting to changes in concrete Poisson's ratio

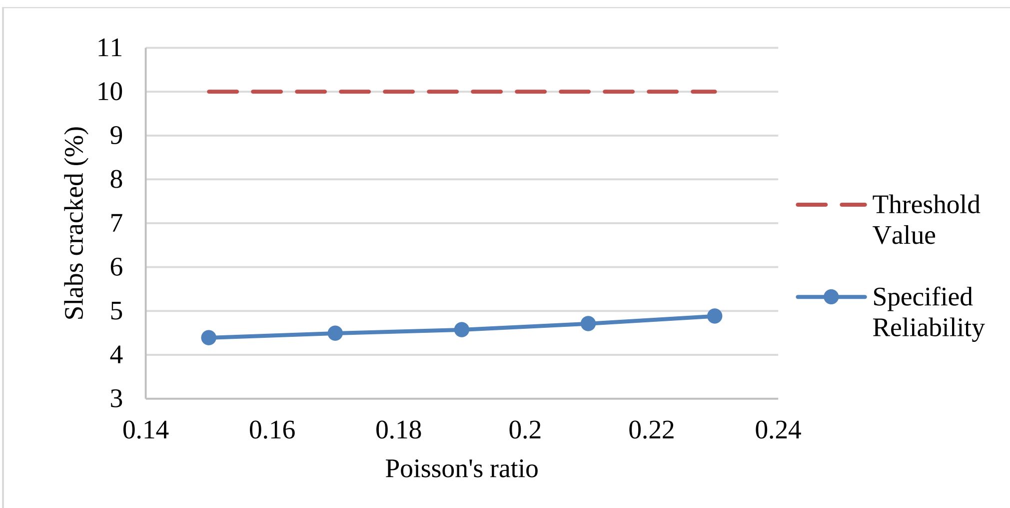

Figure C.5: Sensitivity of predicted slab cracking to changes in concrete Poisson's ratio

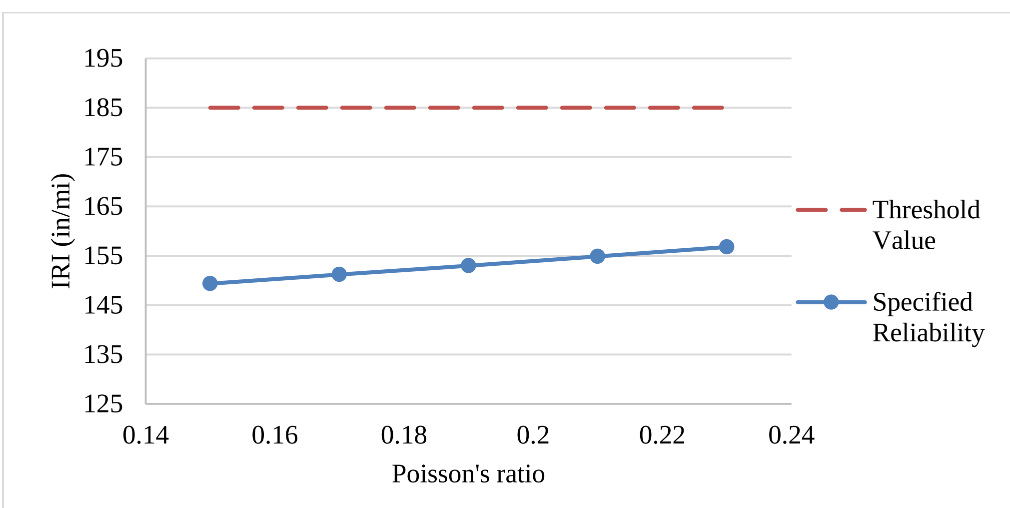

Figure C.6: Sensitivity of predicted IRI to changes in concrete Poisson's ratio

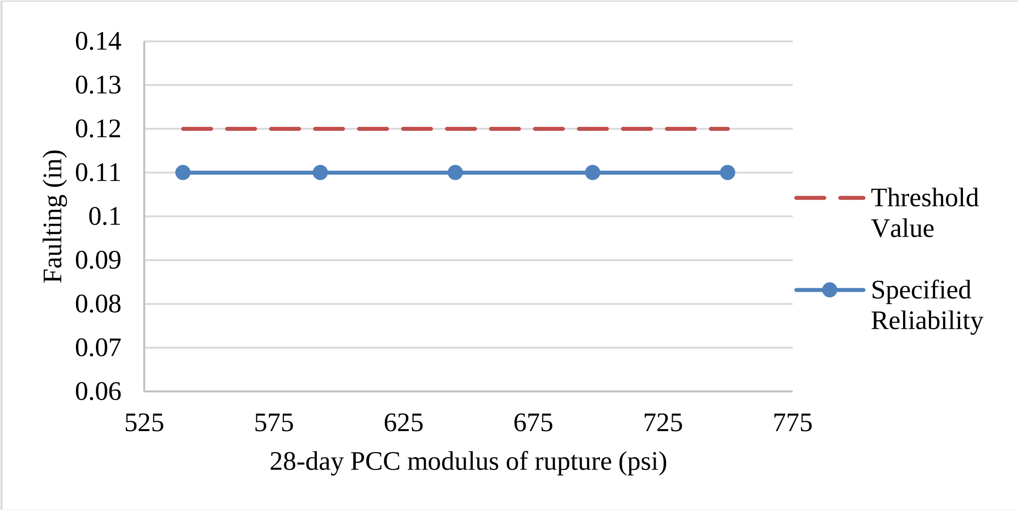

Figure C.7: Sensitivity of predicted faulting to changes in concrete MOR

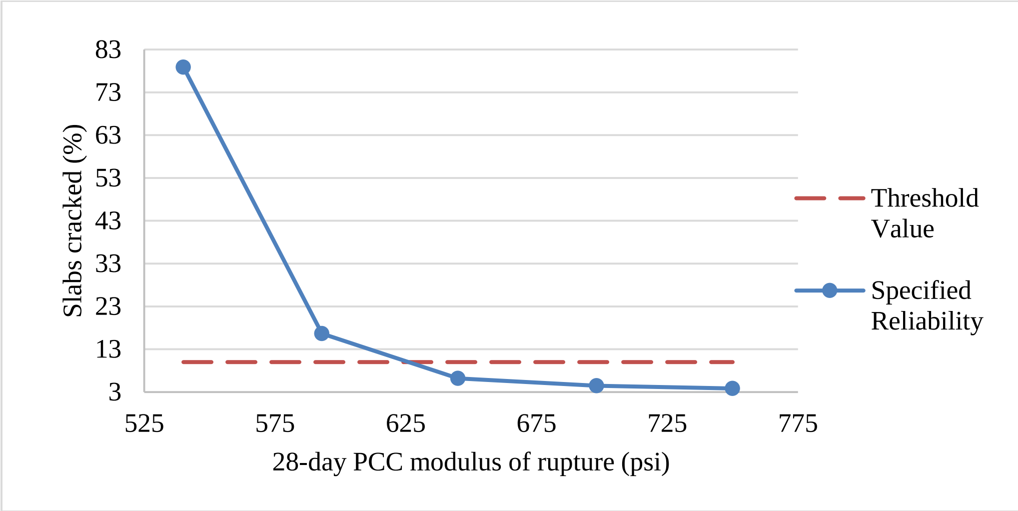

Figure C.8: Sensitivity of predicted slab cracking to changes in concrete MOR

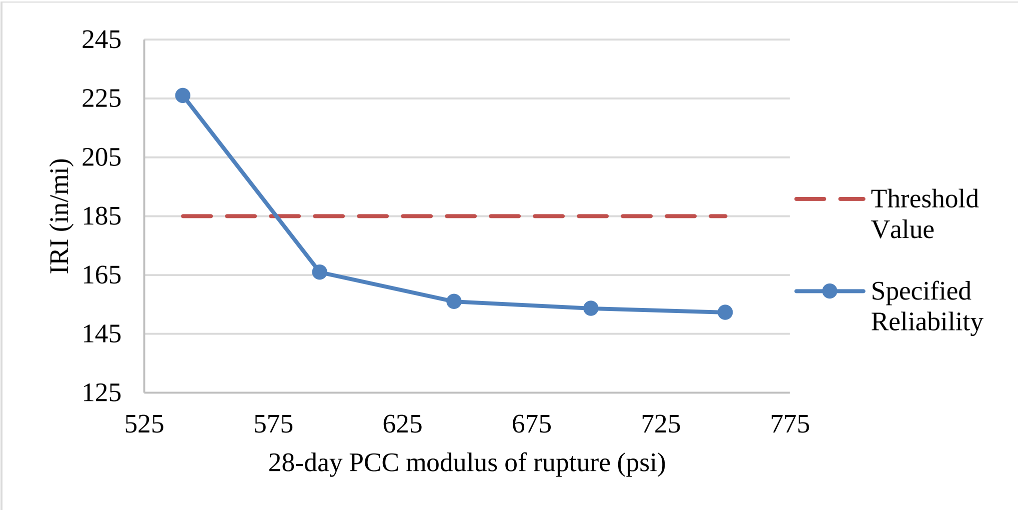

Figure C.9: Sensitivity of predicted IRI to changes in concrete MOR

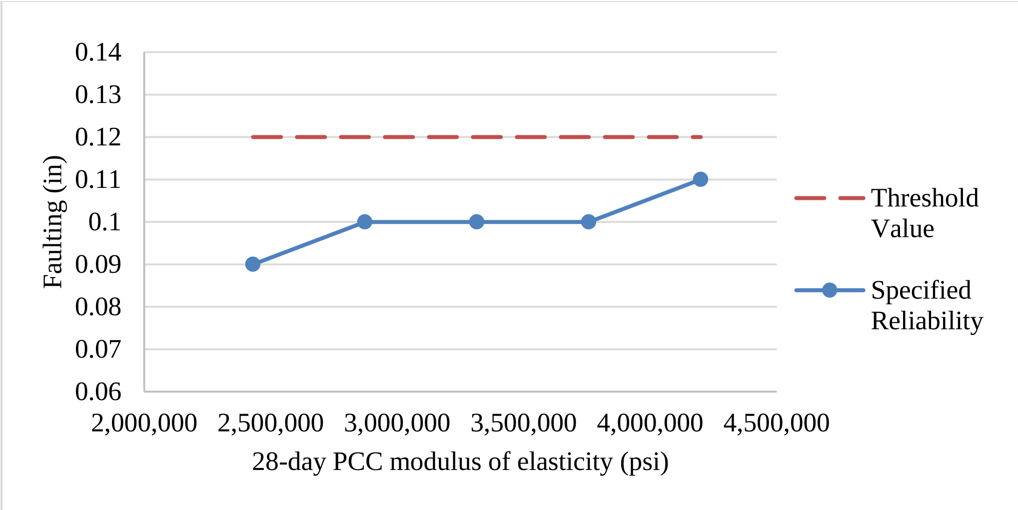

Figure C.10: Sensitivity of predicted faulting to changes in concrete 28-day MOE

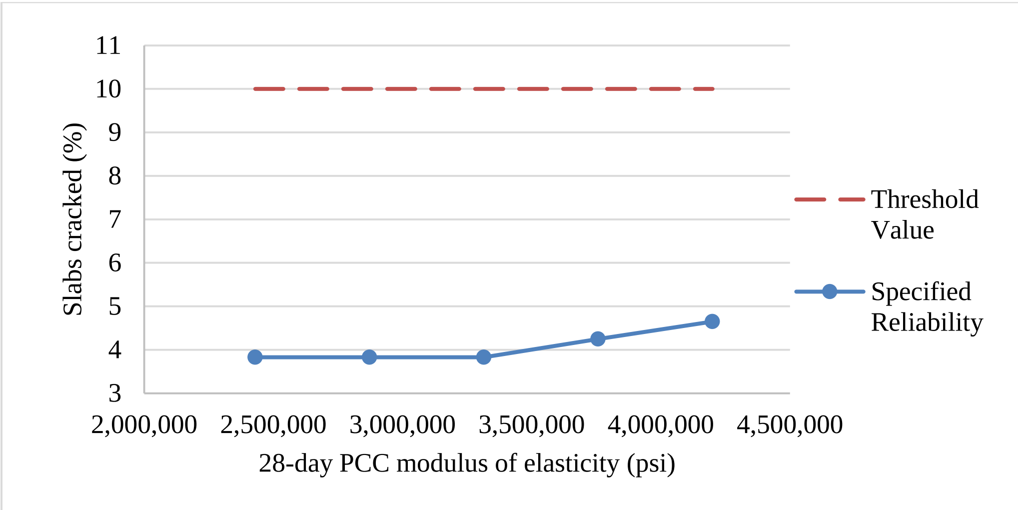

Figure C.11: Sensitivity of predicted slab cracking to changes in concrete 28-day MOE

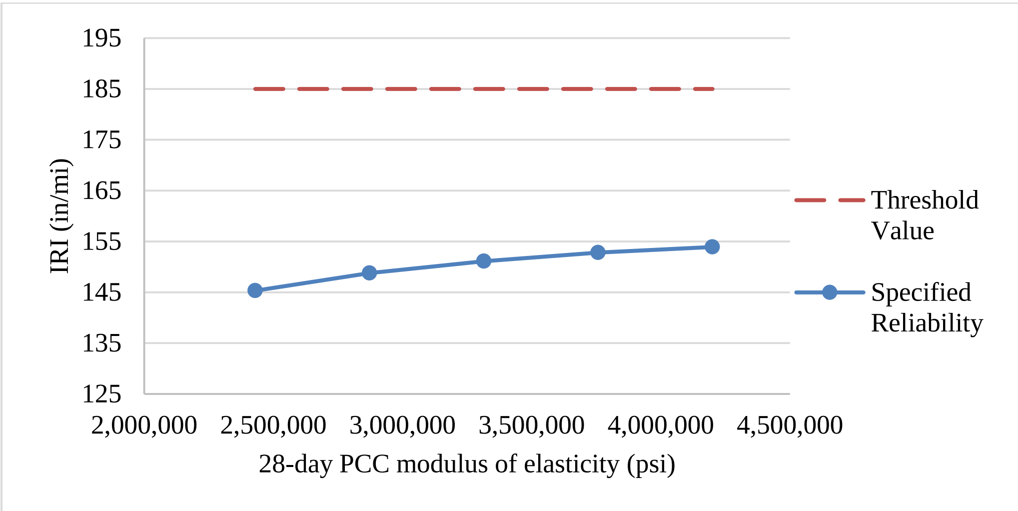

Figure C.12: Sensitivity of predicted IRI to changes in concrete 28-day MOE

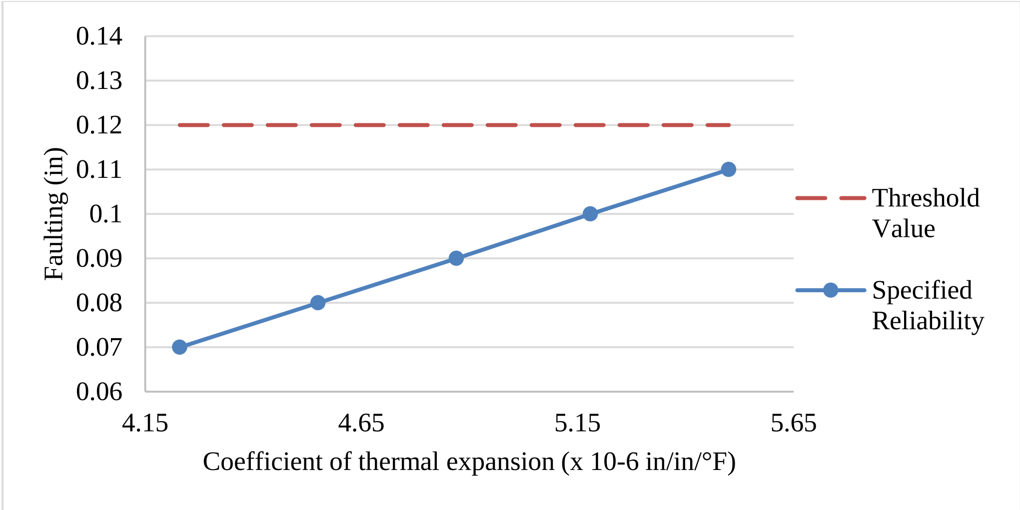

Figure C.13: Sensitivity of predicted faulting to changes in concrete CTE

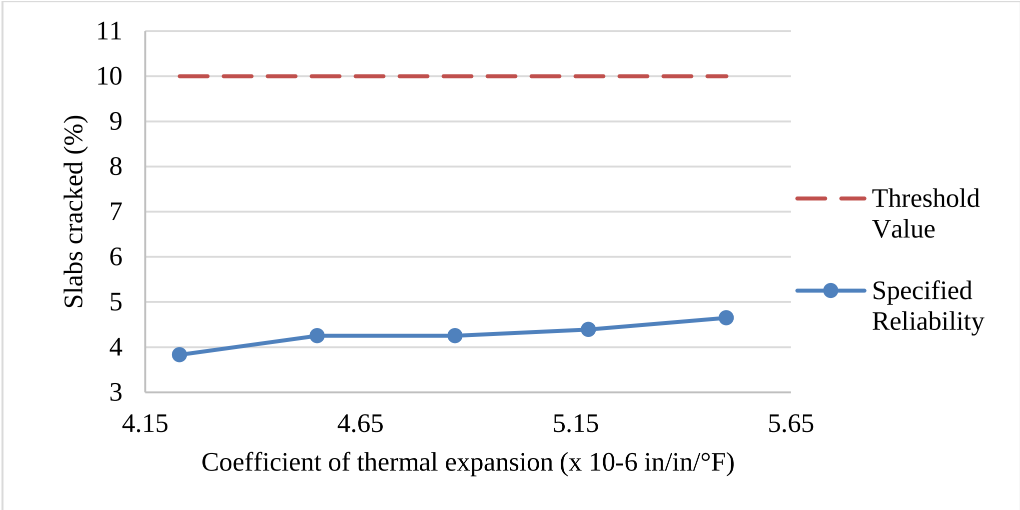

Figure C.14: Sensitivity of predicted slab cracking to changes in concrete CTE

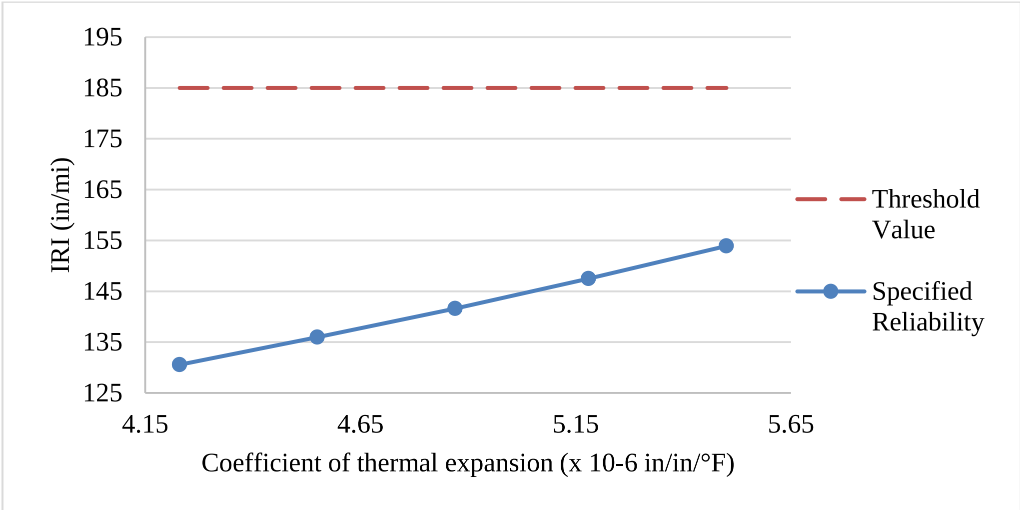

Figure C.15: Sensitivity of predicted IRI to changes in concrete CTE

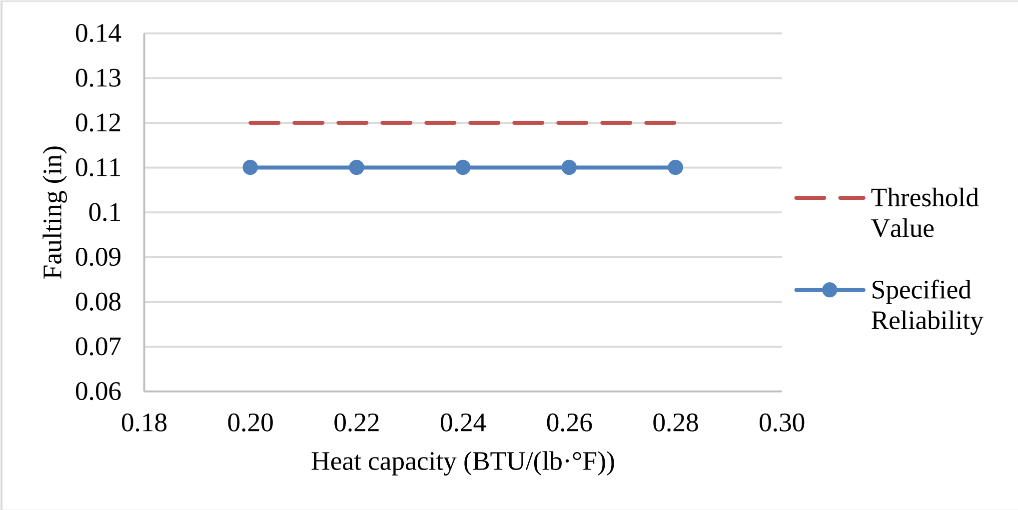

Figure C.16: Sensitivity of predicted faulting to changes in concrete heat capacity

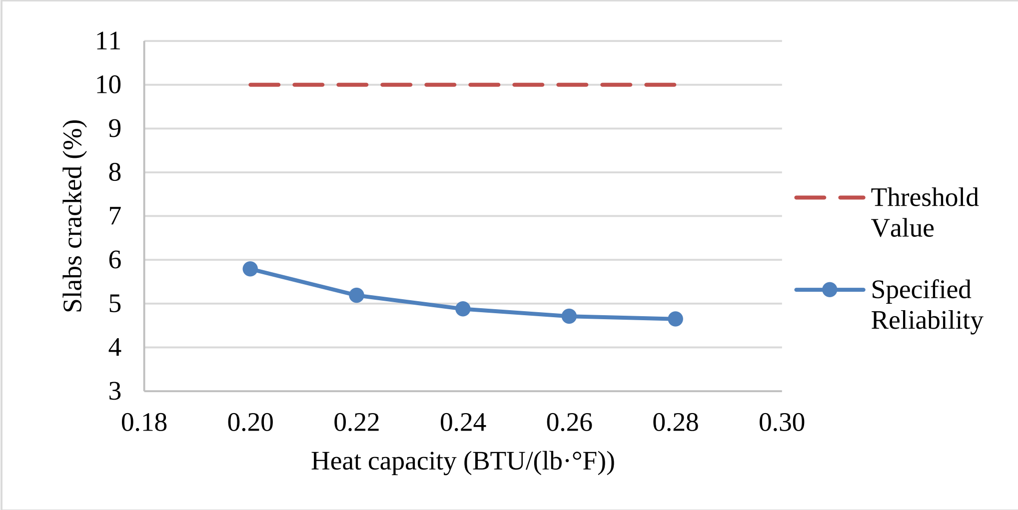

Figure C.17: Sensitivity of predicted slab cracking to changes in concrete heat capacity

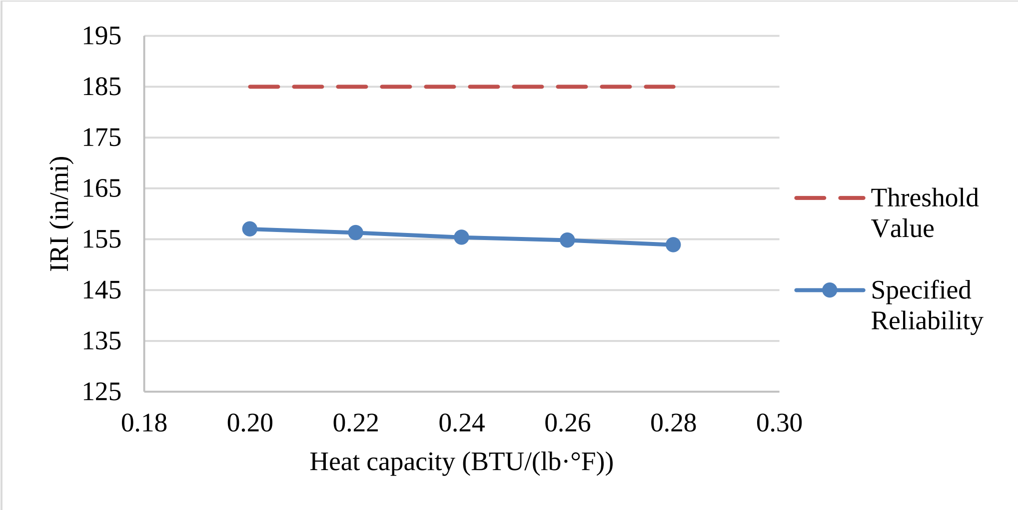

Figure C.18: Sensitivity of predicted IRI to changes in concrete heat capacity

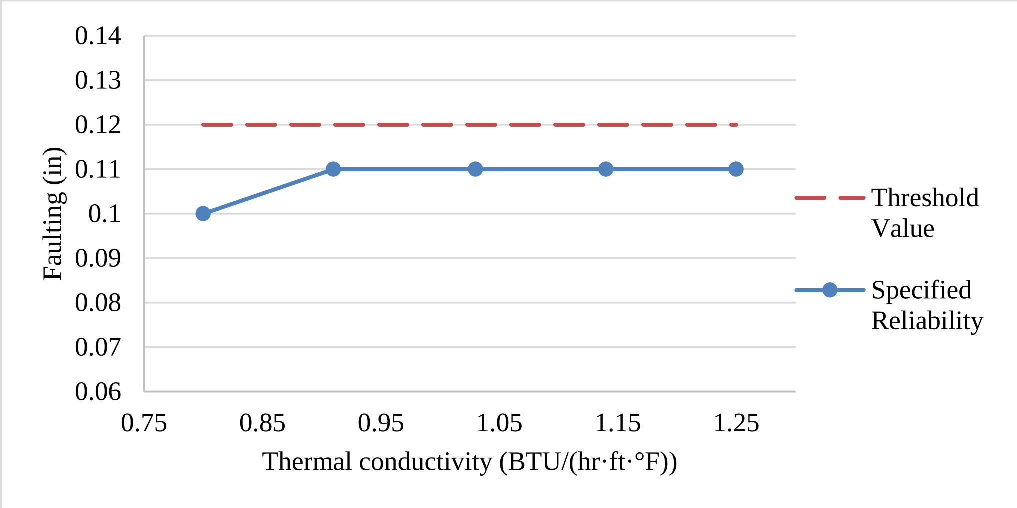

Figure C.19: Sensitivity of predicted faulting to changes in concrete thermal conductivity

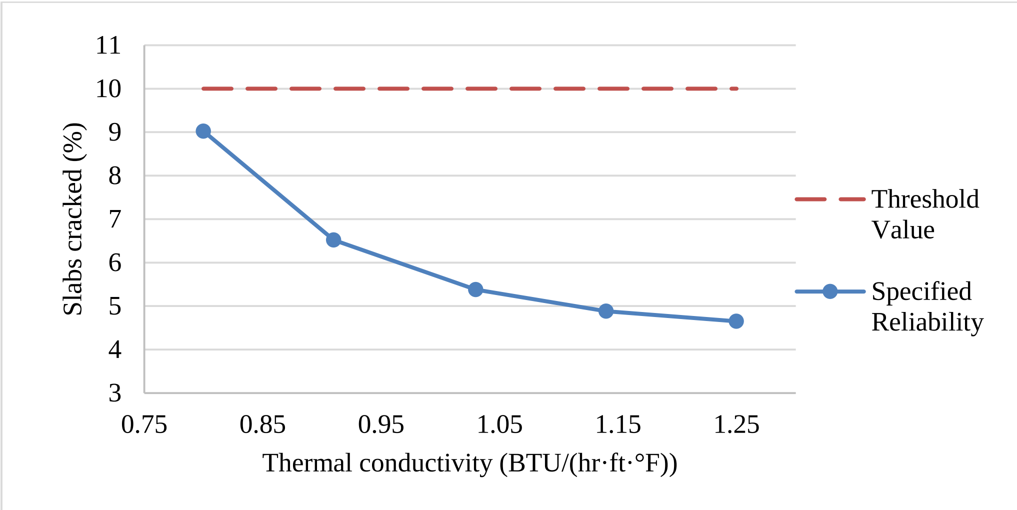

Figure C.20: Sensitivity of predicted slab cracking to changes in concrete thermal conductivity

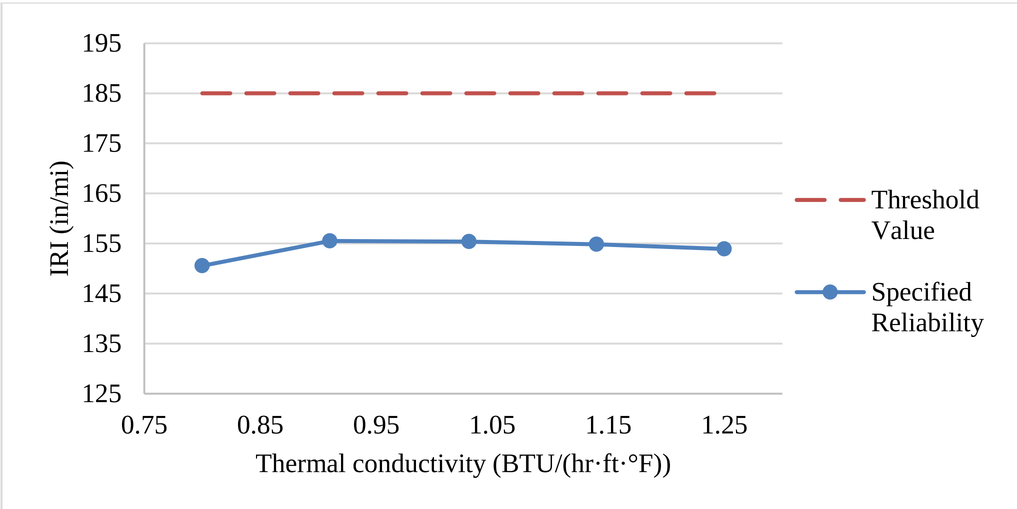

Figure C.21: Sensitivity of predicted IRI to changes in concrete thermal conductivity

Figure C.22: NC region map with project locations

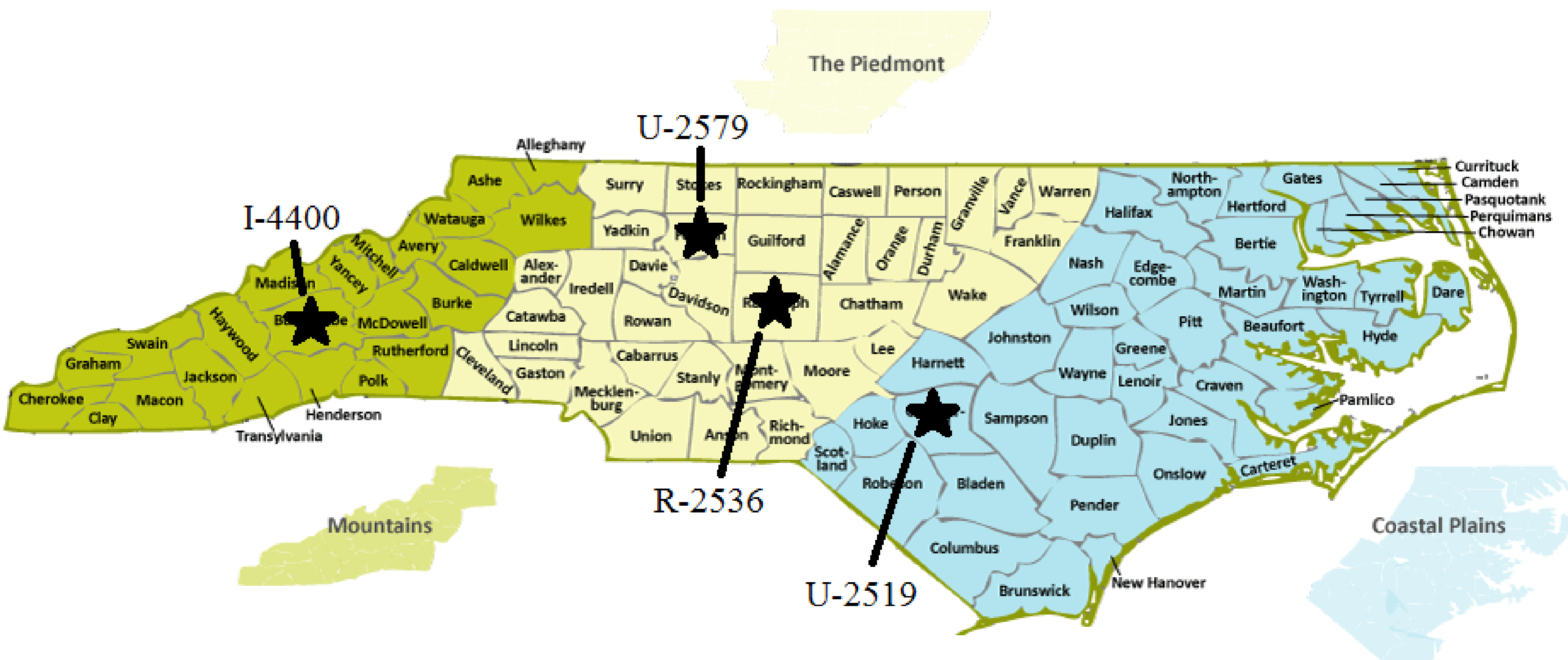

Table C.1: Analysis of project I-4400 (interstate) using previous inputs vs. new recommended inputs for Mountain Region

			NCDOT Project R-2536 Randolph Co.	NCDOT 2MS Manufactured Sand with A Cement	NCDOT 2MS Manufactured Sand with B Cement	NCDOT 2MS Manufactured Sand with BL Cement
Layer 1: PCC	Pavement Thickness (in)		10.5	10.5	10.5	10.5
	Cementitious Material Content (pcy)		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 Compressive Strength (psi)			5,030	5,100	4,790
	28 Day Modulus of Rupture (psi)		650	570	641	606
	28 Day Modulus of Elasticity (psi)		4,200,000	2,540,000	2,760,000	3,020,000
	Poisson's Ratio		0.17	0.18	0.20	0.20
	Coefficient of Thermal Expansion (x 10^{-6} in/(in°F))		6.00	4.46	4.46	4.56
	Heat Capacity (BTU/(lb·°F))		0.28	0.20	0.21	0.20
	Thermal Conductivity (BTU/(ft·hr·°F))		1.25	0.87	0.95	0.91
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	142.84	141.24	146.00
	Mean Joint Faulting (in)	0.12 (Target)	0.11	0.07	0.08	0.08
	JPCP Transverse Cracking (percent slabs)	10.00 (Target)	8.59	4.25	3.83	4.25
Reliability	Terminal IRI (in/mile)		96.88	99.42	99.51	99.18
	Mean Joint Faulting (in)		94.30	99.78	99.69	99.40
	JPCP Transverse Cracking (percent slabs)		93.88	99.87	99.96	99.87

Table C.2: Analysis of project U-2579C (urban freeway) using previous inputs vs. new recommended inputs for Piedmont Region

		Target		NCDOT Project U-2579C Forsyth 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)			11	11	11	11	11
	Cementitious Material Content (pcy)			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-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	117.80	112.06	126.66	121.81
	Mean Joint Faulting (in)	0.12	(Target)	0.08	0.06	0.05	0.07	0.07
	JPCP Transverse Cracking (percent slabs)	10.00	(Target)	4.39	3.83	3.83	3.83	3.83
Reliability	Terminal IRI (in/mile)			99.83	99.99	100.00	99.93	99.97
	Mean Joint Faulting (in)			99.34	99.98	100.00	99.76	99.92
	JPCP Transverse Cracking (percent slabs)			99.83	99.96	99.96	99.96	99.96

Table C.3: Analysis of project R-2536 (rural freeway) using previous inputs vs. new recommended inputs for 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 (pcy)		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 C.4: Analysis of project U-2519 (urban freeway) using previous inputs vs. new recommended inputs for 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 (pcy)		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

Table C.5: Analysis of project U-2519 (urban freeway) using previous inputs vs. new recommended inputs for 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 (pcy)			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

Table C.6: Evaluation of potential PCC thickness reduction for project I-4400 (interstate) using previous inputs vs. new recommended inputs for Mountain Region

			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 (pcy)		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

Table C.7: Evaluation of potential PCC thickness reduction for project U-2579C (urban freeway) using previous inputs vs. new recommended inputs for Piedmont Region – manufactured sand

			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 (pcy)		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

Table C.8: Evaluation of potential PCC thickness reduction for project R-2536 (rural freeway) using previous inputs vs. new recommended inputs for Piedmont Region – natural sand

				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 (pcy)			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

Table C.9: Evaluation of potential PCC thickness reduction for project U-2536 (rural freeway) using previous inputs vs. new recommended inputs for Piedmont Region – manufactured sand

				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 (pcy)			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

Table C.10: Evaluation of potential PCC thickness reduction for project U-2536 (rural freeway) using previous inputs vs. new recommended inputs for Piedmont Region – natural sand

				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 (pcy)			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

Table C.11: Evaluation of potential PCC thickness reduction for project U-2519 (urban freeway) using previous inputs vs. new recommended inputs for Piedmont Region – manufactured sand

			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 (pcy)		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

Table C.12: Evaluation of potential PCC thickness reduction for project U-2519 (urban freeway) using previous inputs vs. new recommended inputs for Piedmont Region – natural sand

				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
Layer 1: PCC	Dowel Diameter (in)			1.25	1.25	1.125	1.125
Layer 1: PCC	Cementitious Material Content (pcy)			600	550	550	550
Layer 1: PCC	Water to cement ratio			0.42	0.48	0.48	0.48
Layer 1: PCC	Unit Weight (pcf)			150	142	142	142
Layer 1: PCC	28 Day Modulus of Rupture (psi)			690	740	740	740
Layer 1: PCC	28 Day Modulus of Elasticity (psi)			4,200,000	3,300,000	3,300,000	3,300,000
Layer 1: PCC	Poisson's Ratio			0.20	0.16	0.16	0.16
Layer 1: PCC	Coefficient of Thermal Expansion (x 10^{-6} in/(in°F))			6.00	5.40	5.40	5.40
Layer 1: PCC	Heat Capacity (BTU/(lb·°F))			0.28	0.20	0.20	0.20
Layer 1: PCC	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
Distress	Mean Joint Faulting (in)	0.12	(Target)	0.10	0.09	0.14	0.13
Distress	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
Reliability	Mean Joint Faulting (in)			97.31	99.09	78.79	85.25
Reliability	JPCP Transverse Cracking (percent slabs)			99.87	99.96	99.96	99.96

Table C.11: Evaluation of potential PCC thickness reduction for project U-2519 (urban freeway) using previous inputs vs. new recommended inputs for Coastal Region

				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
Layer 1: PCC	Dowel Diameter (in)			1.25	1.25	1.125	1.125
Layer 1: PCC	Cementitious Material Content (pcy)			600	550	550	550
Layer 1: PCC	Water to cement ratio			0.42	0.48	0.48	0.48
Layer 1: PCC	Unit Weight (pcf)			150	139	139	139
Layer 1: PCC	28 Day Modulus of Rupture (psi)			690	660	660	660
Layer 1: PCC	28 Day Modulus of Elasticity (psi)			4,200,000	3,000,000	3,000,000	3,000,000
Layer 1: PCC	Poisson's Ratio			0.20	0.19	0.19	0.19
Layer 1: PCC	Coefficient of Thermal Expansion (x 10^{-6} in/(in°F))			6.00	4.30	4.30	4.30
Layer 1: PCC	Heat Capacity (BTU/(lb·°F))			0.28	0.22	0.22	0.22
Layer 1: PCC	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
Distress	Mean Joint Faulting (in)	0.12	(Target)	0.10	0.06	0.11	0.10
Distress	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
Reliability	Mean Joint Faulting (in)			97.31	99.94	94.38	96.86
Reliability	JPCP Transverse Cracking (percent slabs)			99.87	99.96	99.96	99.87

APPENDIX D – SUPPORTING INFORMATION FOR GREEN CONCRETE LCA (Chapter 4)

D.1 Data sources

D.1.1 Raw materials

Data on raw materials in production of cement were collected from a manufacturer using a confidential survey. This data included information on the amount of cement clinker, gypsum, and mode of transportation to the plant. Data on raw materials for concrete production were collected from the laboratory testing performed as part of this work and from available data from Green Concrete web tool. Green Concrete has data from various resources which are provided in at the Green Concrete website and within a thesis publication stemming from this work (Chimmula 2016).

D.1.1 Fuel and electricity

The Green Concrete web tool provided default information on fuel and electricity usage. Modes of fuel and electricity sources available for use in the analysis include: bituminous coal, lignite coal, distillate fuel oil, petroleum coke, residual fuel oil, natural gas, waste oil, waste solvent, waste tire (whole), waste tire (shredded), non-hazardous waste, waste paper, waste plastic, waste sewage sludge, and hazardous waste. Data used for fuels used for pyro processing of was clinker collected from a regional manufacturer supplying concrete to North Carolina through a confidential survey.

Electricity data for the concrete production plant was obtained from the Green Concrete web tool. For the purposes of this comparative analysis, the US average and respective state averages (North Carolina and South Carolina) were chosen depending on the location of the typical production of locally-utilized cements (South Carolina) and concrete batching plants (North Carolina). Electricity data for cement production, operation of the quarry, and concrete batch plant location were taken from default values provided in the tool. Data on pre-combustion fuel, combustion fuel and electricity were collected from various resources such as National Renewable Energy Laboratory, U.S. energy Information Administration (2011b), (2011c), U.S. Environmental Protection agency (1993), (1998a), etc. by the Green Concrete web tool designer. A more complete description of data sources is provided in Appendix E, and in Chimmula (2016).

D.1.2 Transportation

The Green Concrete web tool provided default modes of transportation (along with supporting data used in the analysis) which was collected from various resources. Information on these resources used for Green Concrete web tool are provided in (Chimmula 2016). Modes of transportation of raw materials available for use in the Green Concrete web tool include: truck class 8b (model 2005), truck class 5 (model 2005), truck class 2b (model 2005), rail, and water (inland barge). Transportation inputs for the transfer of raw materials to the cement plant and for the conveying distance of raw materials within the cement plant were collected from manufacturers through a confidential survey.

D.1.3 Technology

Technologies used in processing and handling of raw materials for cement production available for use in the Green Concrete web tool include: dry process raw storing (non-preblending), dry process raw storing preblending, wet process raw storing dry raw grinding (ball mill, tube mill, and vertical roller mill), wet raw grinding (tube mill and wash mill) raw meal homogenization (blending and storing), slurry blending homogenization and storing, preheater/precalciner kiln, wet kiln, long dry kiln, preheater kiln, US average kiln, rotary cooler, planetary cooler, reciprocating grate cooler (modern), reciprocating grate cooler (conventional), vertical gravity cooler with planetary cooler, grate cooler (recirculating excess air), ball mill, tube mill, vertical roller mill, roller press, and horizontal roller mill. Data used in selecting the technology for each phase of cement production and clinker cooling particulate matter (PM) control technology were obtained from manufacturers through a confidential survey.

D.1.4 Emissions

Emission are calculated in Green Concrete based on the other inputs such as raw materials, fuel and electricity, transportation, and technology used. Data required to support this analysis in Green Concrete were collected by the web tool designer from various sources. Additional information regarding the data sources used by the web tool is provided in (Chimmula 2016).

E.1.5 Cement Production Technologies and Plant Operation Assumptions

The Green Concrete web tool allows the user to select technology for production of cement and batching of concrete. For the purposes of the web tool analytical framework, cement production is performed in six phases. Each phase utilizes different technologies in order for the final product to be produced in that respective phase. The six phases considered in the Green Concrete web tool are raw materials prehomogenization, raw materials grinding, raw material blending/homogenization, pyroprocessing, clinker cooling, and finish milling/grinding/ blending with PC. A brief description of each phase is provided below, along with the technologies selected to be held constant for this analysis based on the results of a confidential survey of a southeast regional cement producer.

1. Raw materials prehomogenization: The end product in this phase is raw meal. The Green Concrete web tool allows the user to select one technology from three provided. The provided technologies are dry process raw storing (non-preblending), dry process raw storing (preblending), and wet process raw storing. Among these three alternatives dry process raw storing, non-preblending was utilized in this analysis.

2. Raw materials grinding: The end product in this phase is ground meal. The Green Concrete web tool provided five alternative technologies. They are: dry raw grinding (ball mill), dry raw grinding (tube mill), dry raw grinding (vertical roller mill), wet raw grinding (tube mill), and wet raw grinding (wash mill). Based on the results of the confidential survey, the technology selected for this phase (held constant for the analysis) was dry raw grinding, ball mill.

3. Raw meal blending/homogenization: The end product of this phase is blended meal. The alternatives provided by the Green Concrete web tool for this technology are raw meal homogenization (blending and storage), and slurry homogenization storage. Based on the results of the confidential survey, the technology utilized for this analysis (held constant) was homogenization, blending, and storage.

4. Pyroprocessing: The end product of this phase is clinker. Four technologies are provided in the Green Concrete web tool for the clinker production phase. These alternatives are preheater/precalciner kiln, wet kiln, long dry kiln, pre heater kiln. Based on the response to the confidential survey, the technology selected for use in this analysis was pre heater/ precalciner kiln.

5. Clinker cooling: The end product of this phase is cooled clinker. The Green Concrete web tool provides six alternative technologies for this phase. They are reciprocating grate cooler (modern), reciprocating grate cooler (conventional), rotary cooler, planetary cooler, vertical gravity cooler with planetary cooler, grate cooler (recirculating excess air). For this analysis, the technology used for this phase was again selected through the confidential survey. The technology held constant for this phase was the reciprocating grate cooler (conventional).

6. Finish milling/grinding/blending with PC: The end product of this phase is blended/traditional portland cement. The Green Concrete web tool provides five alternative technologies for this phase. These alternative technologies are ball mill, tube mill, vertical roller mill, roller press, and horizontal roller mill. Since production of PLC is highly dependent on the finish milling/grinding/blending of the limestone with the cement clinker, this technology was varied in the analysis. The purpose of varying this technology was to explore the environmental impact of the finish process used to produce the PLC. As part of this LCA, the five finish milling/grinding/blending technologies were varied.

Conveying of each product in the above phases can also be performed using several different technologies. The technologies available for use in Green Concrete web tool analysis methodology include conveyance by screw pump, airlift, dense phase pump, and bucket elevator. Based on the response to the confidential survey, an appropriate technology was used for different product, and was held constant through this LCA analysis.

1. Raw meal: This is the product from raw meal prehomogenization phase. The selected technology for conveyance was a bucket elevator, and the conveyance distance was held constant at 25 meters.

2. Ground meal: This is the product from the raw materials grinding phase. The selected conveyance technology and distance was selected to be a bucket elevator and 25 meters, respectively.

3. Blended meal: This is the end product from the raw material blending/homogenization phase. The conveyance mode and distance utilized in this analysis are the dense phase pump and 100 meters, respectively.

4. Clinker: This is the end product from the pyroprocessing phase. The technology used for conveyance is the bucket elevator, and the conveyance distance selected was 25 meters.

5. Clinker cooled: This is the end product from the clinker cooling phase. The conveyance technology used was the bucket elevator, and the conveyance distance used was 50 meters.

6. Blended/traditional portland cement: This is the final product in cement production, and is produced in finish milling/grinding/blending with portland cement (PC) phase. The conveyance technology used was a dense phase pump, and the conveyance distance considered in the analysis was 75 meters.

Two technologies are available in the Green Concrete webtool for clinker cooling and particulate matter (PM) control. They are fabric filter and electrostatic precipitators. Based on the confidential survey results, the technology selected for this analysis was fabric filter. The two alternatives that were provided in the Green Concrete web tool are controlled with fabric filter and uncontrolled. For this analysis, an uncontrolled PM emissions was utilized and held constant. Two alternative options were provided for mixing and plant loading were provided in the web tool. Mixer loading (central mix) and truck loading (truck mix) are the options. Mixer loading (central mix) was selected for the purpose of this analysis. Ultimately, the system boundary at the end of production is the gate of the concrete plant, with the truck ready to transport a batch of concrete to a jobsite (Celik et al. 2015).

D.1.6 Process for performing limited LCA for North Carolina concrete mixtures

1. Quarry and plant location, grid mix information: For the quarry and plant location, the electricity grid mix information US average was utilized in the analysis. This section of inputs consists of the electricity source (mix) proportions for raw materials mining, electricity mix for cement plant, electricity mix for gypsum quarrying and processing, electricity mix for fine and coarse aggregate quarrying and processing, electricity mix for limestone quarrying and processing, electricity mix for natural pozzolan quarrying and processing, electricity mix for fly ash processing plant, electricity mix for granulated blast furnace slag processing plant, and electricity grid mix for concrete batching plant.

2. Operation electricity mix: Alternatives were made in this section to evaluate changes in emissions and GWP by reduction non-renewable fossil fuels. In this section, four different grid electricity alternatives were considered. The first (base) analysis was performed by running the analysis with default values. The second analysis option included reducing fossil fuel by 3% and increasing the nuclear fuel by 3%. The third analysis option was to further reduce fossil fuel by 6% and increase nuclear fuel by 6%. The fourth analysis option was to reduce fossil fuel by 10%, and to increase nuclear fuel by 10%. Fuel options for pyroprocessing of cement were taken as 95% bituminous coal and 5% waste tire (whole) through information from the confidential survey.

3. Transportation input: In this section, distance travelled from raw materials to the cement plant were considered. Units for distance were taken as kilometer. For this analysis, the distance travelled from the cement raw materials to cement plant and the gypsum to cement plant were considered. The information about the distance travelled were again collected through a confidential survey from manufacturers. The distance travelled from cement raw materials to cement plant was input as 241.402 km (150 miles) and the distance travelled from the gypsum source to the cement plant was input as 0.4672 km (50 miles). The mode of transportation considered for both of these inputs was Truck Class 8b (model 2005).

4. Technology input: In this section, inputs regarding technology used for different phases of cement production, conveyance distance, and conveyance mode were input into the Green Concrete LCA web tool based on confidential surveys and assumptions. Details regarding this section have been explained the body of this report (Section 4.2.3), along with cement production technologies and plant operation.

5. Run analysis: Once all the inputs are provided in the respective sections run analysis option is selected, the Green Concrete web tool analysis wass performed, and the output graphs and table of emissions are displayed. The LCA results consist of resources use, energy usage, water consumption, and air emissions such as global warming potential (GWP) and air pollutants (CO, NOX, Lead, PM_{10}, SO_2, and volatile organic compounds (VOC)).

6. The results of the LCA analysis, in terms of the environmental impacts as computed by the Green Concrete LCA web tool, are described in Section 4.2.2 and in Chimmula (2012). Finish milling technology did not appear to have an impact on total air emissions for criteria air pollutants (shown in Figure E.7). A shift from fossil fuel sources to nuclear fuel sources also did not have an impact on total air emissions (shown in Figure E.8).

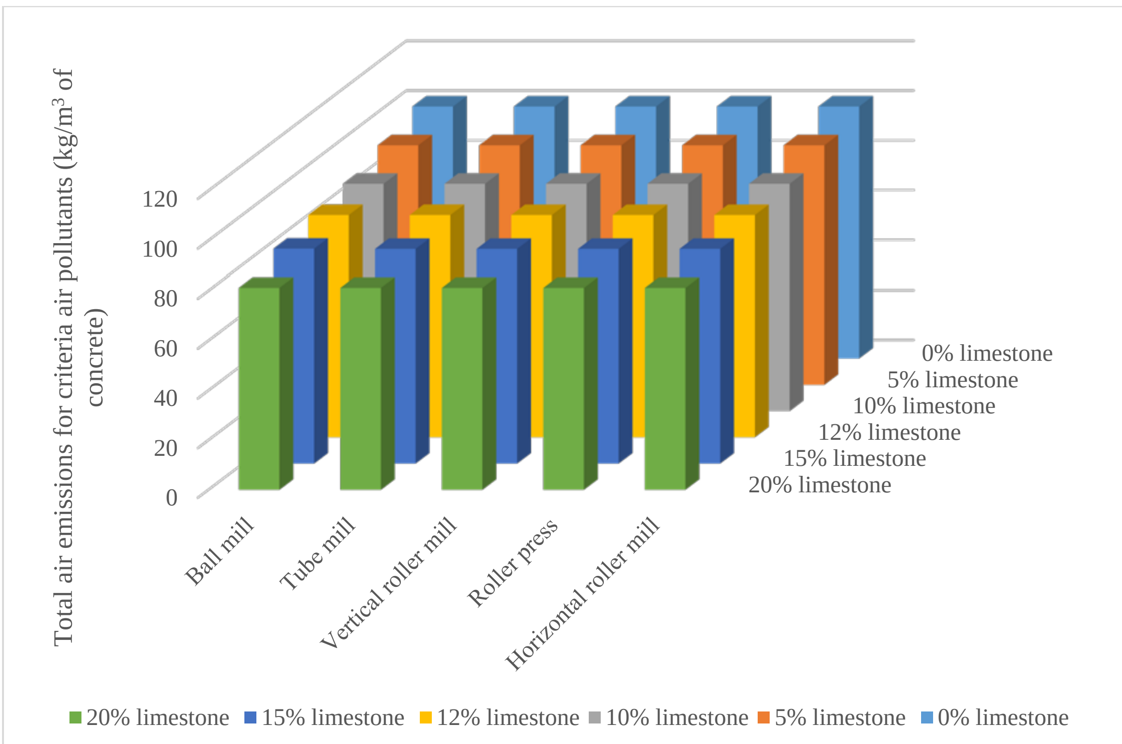

Figure D.1: Air emissions by change in finish milling technology (from Chimmula 2016)

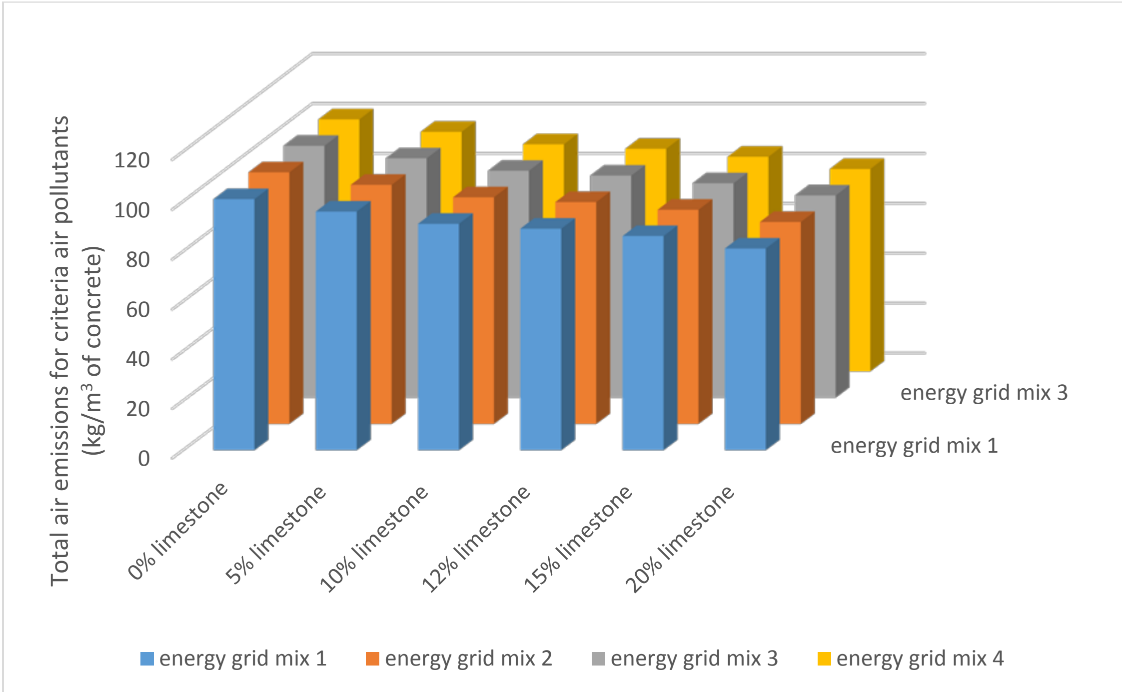

Figure D.2: Air emissions with change in energy source (from Chimmula 2016)

APPENDIX E – SUPPORTING INFORMATION FOR INDUSTRY FORECAST (Chapter 4)

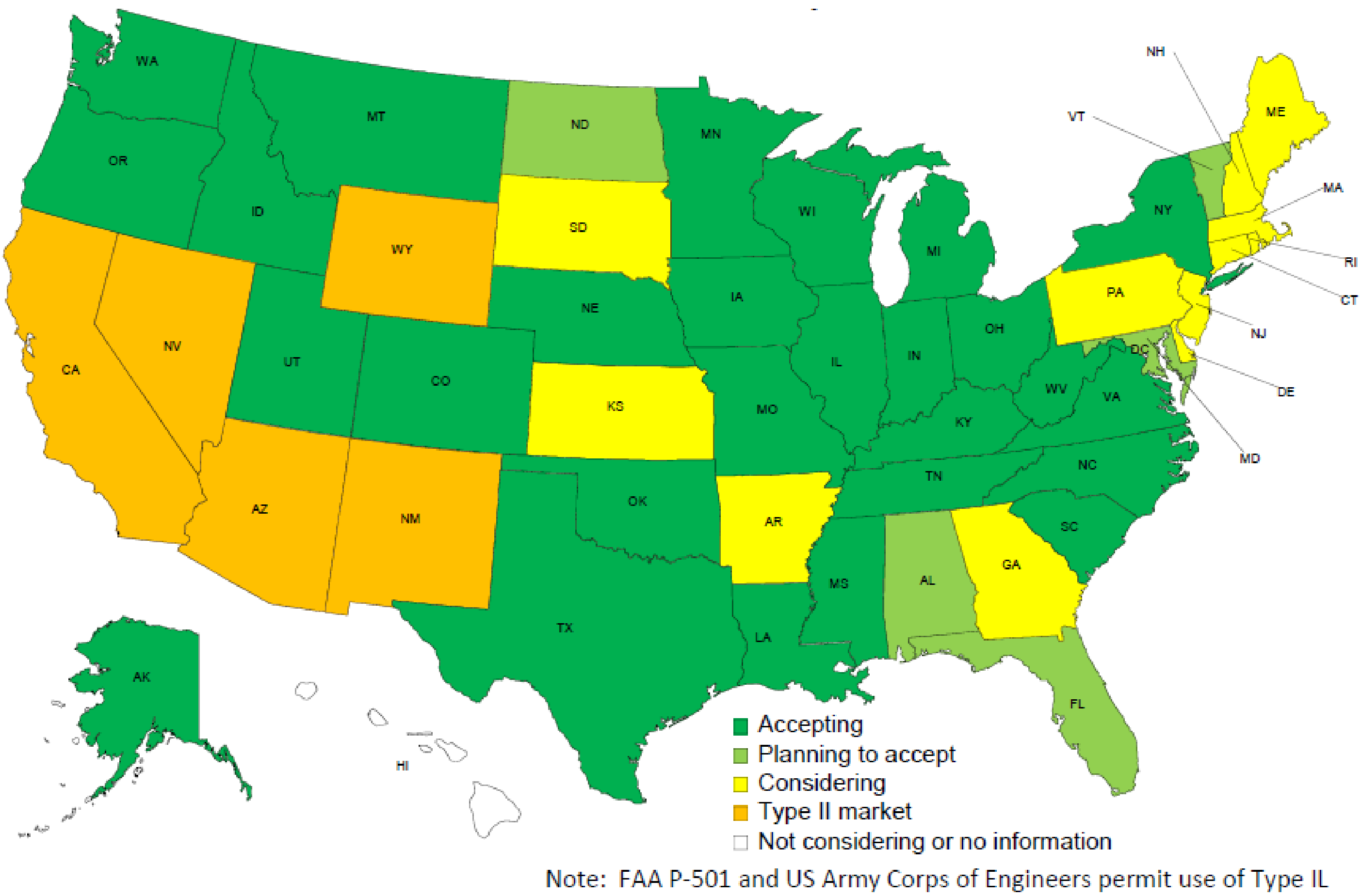

Figure E.1: Acceptance of PLC by State DOTs as of July 2017 (from Tennis 2017)