RESEARCH & DEVELOPMENT

Durable and Sustainable Concrete Through Performance Engineered Concrete Mixtures

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16. Abstract

As funds to construct, maintain, and preserve our highway infrastructure become increasingly stretched, it is imperative that the criteria for selection of concrete mixtures and acceptance of placed concrete reliably ensure durability. Over the past several decades, research has led to new understanding of deterioration mechanisms, advancements in concrete mixture design, and better field and laboratory tests to aid in QA/QC. AASHTO PP 84, "Standard Practice for Developing Performance Engineered Concrete Mixtures" provides guidance to agencies aiming to improve the durability, economy, and sustainability of their infrastructure using both prescriptive and performance specification provisions and emerging technologies. Although developed for pavement concrete mixtures, the approach outlined in AASHTO PP 84 can be extended to include specifications for performance-engineered concrete mixtures utilized for other infrastructure (bridges, lower grade uses, etc.) as well.

The scope of work in this project supports an initial effort to move towards a specification for performance-engineered concrete mixtures by 1) utilizing existing data to identify areas in need of improvement, 2) targeting the establishment of performance-related criteria using several PEM QA/QC tests and approaches of interest to NCDOT, and 3) providing a "roadmap," outlining recommended tasks towards implementation of performance measures, performance goals, and QA/QC protocol in other areas. Existing data from previous concrete research projects for NCDOT were leveraged with new, targeted data to efficiently develop proposed specifications for three areas of interest to improve durability of future concrete infrastructure: surface resistivity, early age strength for opening to traffic, and shrinkage. Guidance for other performance and prescriptive (including water-cementitious ratio and paste content) is also presented for consideration. Suggested specification provisions developed as part of this project are suitable for use by NCDOT on several upcoming projects targeted as pilot projects for PEM research.

Additional study to evaluate the performance of concrete mixtures that include the combination of both fly ash and portland limestone cement (PLC) was also performed. Findings of this work align with previous findings by the research team, indicating significant durability performance benefits in mixtures that contain the two materials. Performance improvements were particularly noted at the current allowable fly ash replacement (30%), which has been increased in recent years. The enhanced durability, sustainability and economy of these mixtures provides additional justification for use of PLC and higher fly ash contents. Findings of this study should allow NCDOT to specify and use of more durable, sustainable concrete mixtures in North Carolina highway infrastructure. Ultimately, paired with other efforts aligned with the national PEM initiative, findings this project should result in cost savings associated with construction costs, QA/QC costs, reduced maintenance costs, and extended life of concrete pavements and bridges.

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EXECUTIVE SUMMARY

As funds to construct, maintain, and preserve our highway infrastructure become increasingly limited, it is imperative that the criteria for selection of concrete mixtures and acceptance of placed concrete reliably ensure durability. The long service life expectations of pavements, bridges, and other components are often difficult to meet by using typical tests for specification and acceptance, which center around three criteria: slump, air content, and compressive strength. These three criteria are only loosely related to deterioration phenomena and so do not always ensure satisfactory field performance. Consistent with the focus of MAP-21 legislation on performance, there is a desire by the Federal Highway Administration (FHWA), public agencies, and industry to move towards performance-engineered construction materials. Performance-engineered concrete mixtures (PEM) include optimized mixture designs (materials selection, gradation, cement content, etc.) which, paired with advanced quality assurance methods, provide substantially improved durability, economy and sustainability.

Over the past several decades, research has led to new understanding of deterioration mechanisms, advancements in concrete mixture design, and better field and laboratory tests to aid in quality assurance/quality control (QA/QC). With this new knowledge, a FHWA initiative to move to performance-engineered concrete mixtures is underway. This initiative has resulted in development of AASHTO PP 84, " Standard Practice for Developing Performance Engineered Concrete Pavement Mixtures," which provides guidance to agencies aiming to improve the durability, economy, and sustainability of their infrastructure using both prescriptive and performance specification provisions along with emerging technologies. Although developed for pavement concrete mixtures, the approach outlined in AASHTO PP 84 can be extended to include specifications for performance-engineered concrete mixtures utilized for other infrastructure (bridges, lower grade uses, etc.) as well.

Development and implementation of performance-related specifications is an extensive undertaking, and the shift will impact all stakeholders in the construction process. Therefore, the scope of work in this project supports an initial effort to move towards a specification for performance-engineered concrete mixtures by 1) utilizing existing data to identify areas in need of improvement, 2) targeting the establishment of performance-related criteria using several QA/QC tests and approaches of interest to NCDOT, and 3) providing a "roadmap," outlining recommended tasks towards implementation of performance measures, performance goals, and QA/QC protocol in other areas.

Analysis of data from current and historically utilized concrete mixtures was performed, linking existing mixture characteristics, early-age test results, and field performance to identify trends associated with adequate and inadequate durability. Based on the results of data mining, literature review, and a survey of other existing state highway agency efforts to implement performance-related specifications for concrete mixtures, a laboratory program was developed and implemented to 1) provide data to support development of specification provisions for the surface resistivity meter, volumetric shrinkage, and early age strength for opening to construction equipment and traffic, and 2) provide preliminary data to support identification of additional performance and prescriptive measures that could be further explored for use in performance-engineered concrete specifications. A "roadmap" for additional tasks recommended to facilitate a move towards performance-engineered concrete specification provisions is proposed. Suggested specification provisions developed as part of this project are suitable for use by NCDOT on several upcoming projects targeted as pilot projects for PEM field implementation and study.

One notable component of the laboratory portion of this project was additional study to evaluate the performance of concrete mixtures that include the combination of both fly ash and portland limestone cement (PLC). Findings of this work align with previous findings by the research team on NCDOT Research Project (RP) 2015-03, indicating significant durability performance benefits in mixtures that contain the two materials. Performance improvements were particularly noted at the current allowable fly ash replacement (30%), which has been increased in recent years. The enhanced durability, sustainability and economy of these mixtures provides additional justification for use of PLC and higher fly ash contents.

Findings of this study should allow NCDOT to specify and use of more durable, sustainable concrete mixtures in North Carolina (NC) highway infrastructure. The roadmap provides a vision for the remainder of the development and implementation tasks for performance-related specification provisions. Ultimately, paired with other efforts aligned with the national PEM initiative, findings this project should result in cost savings for NCDOT, associated with construction costs, QA/QC costs, reduced maintenance costs, and extended life of concrete pavements and bridges.

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LIST OF ABBREVIATIONS

AAR	alkali-aggregate reactivity
AASHTO	American Association of State Highway and Transportation Officials
AC	alternating current
ACI	American Concrete Institute
ASR	alkali-silica reactivity
ASTM	American Society for Testing and Materials
BME	Bridge Management Elements
BMS	Bridge Management System
DOH	Division of Highways
DOT	Department of Transportation
FHWA	Federal Highway Administration
ft	foot
kg	kilogram
kΩ	kilo-ohm
ID	identification
LaDOTD	Louisiana Department of Transportation and Development
lb	pound
LCA	lifecycle assessment
LCCA	lifecycle cost analysis
LTPP	Long Term Pavement Performance
MOE	modulus of elasticity
MOR	modulus of rupture
NBE	National Bridge Elements
NC	North Carolina
NCC	National Concrete Consortium
NCDOT	North Carolina Department of Transportation
Ω	ohm
OPC	ordinary portland cement
PEM	performance engineered (concrete) mixtures
PCC	portland cement concrete
pcf	pounds per cubic foot
pcy	pounds per cubic yard
PLC	portland limestone cement
PMS	pavement management system
psi	pounds per square inch
PLC	portland limestone cement
QA	quality assurance
QC	quality control
RCPT	rapid chloride permeability test
RP	Research Project
SCM	supplementary cementitious material
SHA	state highway agency
SG	specific gravity
SHRP	Strategic Highway Research Program
TRB	Transportation Research Board
US	United States
VDOT	Virginia Department of Transportation
w/cm	water to cementitious materials ratio

1. INTRODUCTION AND RESEARCH OBJECTIVES

1.1 Introduction

Tests used for specification and acceptance of concrete mixtures have typically centered around three criteria (slump, air content, and strength) that are not always good predictors of long-term performance (Cackler et al. 2017). However, the current economic and policy environment has required highway engineers to focus on durability as a means of reducing maintenance and replacement costs. This durability must be imparted to concrete by careful selection of constituent ingredients as well as testing for enhanced acceptance criteria. Changes to concrete mixtures, such as use of new admixtures and the increased use of combinations of supplementary cementitious materials (SCMs) and exposure conditions (increased use of deicing chemicals), is resulting in a need to readdress the way that concrete mixtures are specified and tested (Ahlstrom 2016, Cavalline et al. 2016, Cackler et al. 2017). Consistent with the focus of MAP-21 legislation on performance, there is a desire by FHWA, public agencies, and industry to move towards performance-engineered construction materials. Performance-engineered concrete mixtures include optimized mixture designs (materials selection, gradation, cement content, etc.) that are engineered to meet or exceed design requirements, and are predictable, durable, and have increased sustainability (Ahlstrom 2016). The following are keys to implementation of performance engineered concrete (Taylor 2016):

- Design and field control of concrete mixtures around engineering properties related to performance
- Development of practical, performance-related specifications
- Incorporation of this knowledge into an implementation system (Design / Materials / Construction / Maintenance)
- Validation and refinement by performance monitoring

Performance-related specifications provide the ability for agencies to obtain the desired construction quality while allowing contractor greater control and flexibility (Ahlstrom 2016). For instance, current prescriptive specifications for minimum cement content and rate of strength gain may preclude the acceptance of mixtures that have superior economy, durability, and satisfactory mechanical performance, but contain high proportions of SCMs. AASHTO PP 84, “Standard Practice for Developing Performance Engineered Concrete Pavement Mixtures” was initially released as a provisional standard in 2016, and has since been revised yearly as FHWA’s PEM initiative evolves (AASHTO 2017, 2018, 2019). Although developed for pavement concrete mixtures, the approach outlined in AASHTO PP 84 can be extended to include specifications for performance-engineered concrete mixtures utilized for other infrastructure (bridges, lower grade uses, etc.) as well. The specification provides a framework and guidance for state highway agencies to develop specification provisions for performance engineered concrete mixtures that focuses on measurement and acceptance of concrete based on characteristics that have been linked to satisfactory long-term durability performance of the concrete. These parameters include (AASHTO 2017):

- *Sufficient strength*
- *Low risk of cracking and warping due to drying shrinkage*
- *Durable (freeze-thaw resistance)*
- *Durable (resistance to chemical deicers)*
- *Durable (low absorption, diffusion, and other transport related properties)*
- *Durable (aggregate stability)*
- *Workable*

Performance-related specifications require measurement of key properties and performance characteristics. For performance specifications to be successfully utilized, QA/QC tests should be rapid, effective, reliable, and inexpensive (Taylor 2016). A number of state agencies, including NCDOT, are using and evaluating new, rapid, early-age testing technologies such as resistivity, formation factor determination, and air void system analysis that support development and use of performance-engineered concrete mixtures. The capabilities of these tests to evaluate the durability performance of concrete mixtures is improving as state highway agencies build sufficient data to correlate the test results with durable field performance.

Ongoing concrete materials research is beginning to provide NCDOT data to support use of performance engineered concrete mixtures. However, additional work is needed to identify appropriate performance measures, performance goals, and QA/QC protocol. To support FHWA’s performance-engineered concrete initiative and implementation of AASHTO PP 84, ongoing research is being performed at other universities to enhance the knowledge of the basic science and emerging tests that form the foundation of the draft specification. NCDOT personnel have expressed a desire to concurrently perform research on North Carolina concrete mixtures to support implementation of certain targeted testing technologies suggested by AASHTO PP 84, identify areas of opportunity to improve specifications for concrete infrastructure, and move towards specification and use of performance engineered concrete mixtures.

Additionally, NCDOT has recently allowed use of portland limestone cements (PLC, or Type IL cements), which are blended cements containing up to 15% interground limestone, and has increased the allowable fly ash substitution rate from 20% to 30% (NCDOT 2018). The reduced greenhouse gas emissions and energy consumption of PLCs make them sustainable alternatives to ordinary portland cement (OPC). PLC has also been demonstrated to show improved performance linked to the production process in which both the clinker and limestone are ground together to finer sizes than in ordinary portland cement. This process supports improved particle packing and facilitates increased hydration of cement paste, ultimately improving strength and durability performance (Tennis et al. 2011). Research at UNC Charlotte from NCDOT Research Project (RP) 2015-03 (Cavalline et al. 2018) has confirmed additional durability benefits to cement blends that combine PLC and fly ash from sources supplying North Carolina. As measured by the ASTM C1202 Rapid Chloride Permeability Test (RCPT) and by surface resistivity, mixtures that contained fly ash and PLC were approximately 30% less permeable. This reduced permeability is strongly linked to increased durability in a wide range of aggressive environments such as those subject to chemical deicing or marine exposure. In the UNC Charlotte research, the combination of fly ash and PLC did not substantially affect the mechanical characteristics of the concrete. This initial work was performed at fly ash substitution rates of 20% (not the 30% currently allowable), and additional research performed in this study aims to further identify benefits of pairing of PLC with fly ash at the 30% substitution rate.

1.2 Research Objectives

Development and implementation of performance-related specifications is an extensive undertaking, and the shift will impact all stakeholders in the construction process. Therefore, the scope of work in this proposed project supports an <u>initial</u> effort to move towards a specification for performance-engineered concrete mixtures. The objectives of this proposed research project are as follows:

1) Utilize existing data on concrete materials, mixtures, and field performance, to identify trends in materials and proportions, and link to unacceptable, acceptable, and excellent performance,
2) Perform laboratory testing of a broad matrix of conventional highway concrete mixtures, to establish performance-related criteria using several rapid, early age QA/QC tests to assess durability currently of interest to NCDOT,
3) Produce additional performance data on concrete containing PLC and fly ash to support a better understanding the potential enhanced durability and economy of these mixtures and provide additional justification for use,
4) Develop specification provisions for surface resistivity, shrinkage, and early age strength for opening of pavements and bridge components to loads. Guide specifications or project special provisions will also be developed that could be utilized in pilot projects or other trial settings.
5) Provide NCDOT with a "roadmap," outlining recommended tasks towards implementation of performance measures, performance goals, and QA/QC protocol in other areas.

2. SUMMARY OF KEY LITERATURE FINDINGS

As demand for new infrastructure increases, along with an increased burden of maintenance and rehabilitation of existing infrastructure, resources required to support this work are becoming more limited. To address this need, researchers and industry leaders are increasingly investigating and developing sustainable solutions in a variety of areas, including construction materials and practices. Sustainability demands and resource scarcity are the root cause of the development of the Performance Engineered Mixtures (PEM) initiative, in which mixtures are designed to control and optimize the resources used in the process of making concrete (Ahlstrom and Richter 2018). By engaging engineering approaches during initial mixture development and qualification to determine the most efficient proportions of each material in concrete, PEMs are designed to last longer, have lower life cycle costs, and lower environmental impact amongst several other benefits. In addition, designers are able to enhance the quality of concrete by matching the properties of concrete to performance (Ahlstrom and Richter 2018).

Rather than specifying prescriptive requirements for concrete mixtures (such as materials and methods), performance based specifications are based on measurable aspects of performance that can allow concrete to be tailored to the environment and use in which it will serve (Cackler et al. 2017a, AASHTO 2017, 2018, 2019). These include performance requirements such as resistance to cracking and ingress of deleterious substances in specifications can result in concrete produced and constructed that is far more durable than concrete produced and constructed under prescriptive specification provisions (AASHTO 2017, 2018, 2019). Performance requirements allow concrete manufacturers to innovate and leverage their experience, adjusting mixture inputs with sustainability, economy, and constructability in mind, and producing concrete more finely tuned to perform under specific conditions.

Development and implementation of performance-related specifications is an extensive undertaking, and the shift will impact all stakeholders in the construction process. Therefore, the scope of work in this proposed project supports an <u>initial</u> effort by the North Carolina Department of Transportation (NCDOT) to move towards a specification for performance-engineered concrete mixtures. This literature review provides background information supporting the objectives and work performed, as outlined in the main body of this report.

2.1 Concrete Durability

In addition to mechanical sufficiency, the durability performance of concrete is an important consideration in the long-term success of the structure or pavement. In its 2013 circular on concrete durability, Transportation Research Board (TRB) Committee AFN30 Durability of Concrete stated "durability is not an intrinsic, measurable property of concrete. Instead it is a set of material properties that are required for the concrete to resist the particular environment in which it serves (Taylor et al. 2013)." To be durable, concrete must withstand distress from a variety of aggressive agents and environmental conditions, as well as service loads. Concrete durability has been defined as "the ability of concrete to resist weathering action, chemical attack, and abrasion while maintaining its desired engineering properties (Kosmatka et al. 2011)." The American Concrete Institute (ACI) Guide to Durable Concrete addresses fresh properties, resistance to freezing and thawing deterioration, resistance to alkali-aggregate reaction (AAR), resistance to chemical attack and corrosion, and resistance to abrasion (ACI 2008). The environment in which concrete is produced, placed, and maintained also plays a major part in the performance of the concrete. This is why identical mixtures placed in different climates can produce vastly different short and long-term performances (Kockal and Turcker 2007).

Weathering can be thought of as the effects of exposure to weather and climatic conditions on the concrete structure, along with other factors such as exposure to chemicals, storm water, or other elements. Wind, precipitation, temperature change, humidity, and other environmental factors can cause or contribute to deterioration of the concrete. Concrete is susceptible to attack from chemical substances introduced in the form of sulfates, chlorides, or other compounds. When these chemicals are introduced, reactions can occur producing new substances growing in the concrete structure. Secondary reactions, which often involve materials aside from those initially present during cement hydration, are generally not desirable once concrete hydration is essentially complete (ACI 2008).

Weathering and other mechanical distress can also be exacerbated by mechanical loads. Abrasion of concrete surfaces becomes a more prevalent issue as traffic loads on roadway systems increase. The demand for shipping of goods has resulted in heavier weights and increased passes of freight trucks, and concurrently greater wear on concrete pavements. As traffic loads and design expectations continue to rise, ability to mitigate deteriorating factors becomes increasingly important (ACI 2008).

2.2 Characteristics of Durable Concrete

<u>Materials</u>

Selecting appropriate and efficient materials for concrete is essential in having durable, sustainable concrete. The primary materials that impact concrete durability include aggregates, admixtures, and (the most expensive and highest environmental impactor) portland cement. Other cementitious materials, particularly SCMs, are often essential to ensuring concrete that is durable is batched and placed. Material selection should provide the prescribed mechanical performance while also considering durability goals. A concrete mixture design that combines the correct materials in proper proportions will succeed far more often than if either design aspect is neglected (Kosmatka and Wilson 2016).

<u>Mixture Proportions</u>

Mixture proportioning is important for a number of reasons, such as its influence on fresh properties, the ability of a mixture to meet the required mechanical properties and durability performance, and adhering to specification provisions and/or guidance limiting specific materials based on application (Kosmatka and Wilson 2016, ACI 2019). To optimize concrete performance, concrete mixtures must be designed with proportions that 1) are economical and 2) tend to exhibit the appropriate workability and ease of placement and finishing in the field (Kosmatka and Wilson 2016). Mixture characteristics should be selected based upon the environment of concrete placement and service, the size and shape of concrete members, the desired physical and chemical concrete properties, and anticipated exposure conditions (Taylor et al. 2013, ACI 2019). Also, concrete performance characteristics such as resistance to sulfate attack and resistance to chloride penetration should be considered (Cackler et al. 2017a).

Construction

Selecting quality materials and proper proportions are important to ensure the desired durability performance, but if the concrete is not batched, placed, and cured properly during construction, poor performance can result. Incorrect placement and/or curing methods can also compromise the concrete's mechanical properties. Many of the factors relating to concrete durability such as pore structure and air void systems are directly influenced by how the concrete is placed and finished (Hearn et al. 2006, Hover 2006). Material storage, batching, transport, placement consolidation, and curing methods each impact the quality of fresh and hardened concrete. Avoiding segregation between the coarse aggregate and the mortar is necessary, and any mixtures that will be pumped into place must have the air content change taken into account during the design phase. If workability of the concrete is of concern, retarders are often used to delay set time. These retarders can result in a lack of small entrained air bubbles (Du and Folliard 2005).

Materials must be stored, batched, mixed, and transported in a manner that supports successful placement. For example, the aggregate chosen for a mixture should be piled, transported, and stored properly to prevent segregation, prevent contamination, and ensure moisture content is controlled (Kosmatka and Wilson 2016). Methods utilized for batching, mixing and transporting concrete should comply with ASTM C94, "Standard Specification for Ready-Mix Concrete" (ASTM 2020). During concrete batching, each individual material must be added within tolerances provided by specifications. Concrete should be mixed thoroughly until all materials are uniformly distributed, and re-mixed within limits if mixture stiffens during transport (Kosmatka and Wilson 2016). Transportation from the mixer to the site of placement should be done with minimal impact to the original design conditions (such as slump, *w/cm* ratio, air content, etc.). For hot and cold weather concrete placements there are provisions that must be followed to guarantee expected properties after concrete has set (Taylor et al. 2013). Guidelines for proper concrete placement are outlined in ACI 304R (ACI 2009).

Construction factors such as water added on-site, placement, and curing measures have a direct influence on concrete pore structure. Much like pore structure, vibration can also have an effect on air void systems. Vibration can result in smaller air bubbles forming with larger ones, directly influencing the air void system (Du and Folliard 2005). Improper vibration techniques of concrete can lead to the destruction of the pore structure of concrete through thixotropy, or the lessening of viscosity (Chappuis 1990). By adding water on-site for workability purposes, a contractor can exceed the maximum *w/cm* which will affect pore structure. Proper consolidation helps remove excess voids in the concrete and increases its durability. Guidance on consolidation can be found in ACI 309R (ACI 2005).

Finishing and curing are the final steps in the construction process. Finishing should be performed with as little manipulation as possible, as overworking the concrete surface may reduce the surface air content and cause fine aggregates to rise to the top, increasing the cracking potential. Saw cutting of the groves and joints should be delayed until the concrete is strong enough to prevent coarse aggregate movement. If concrete is not properly cured, water at the surface can be evaporated and cement particles will not have the water needed for proper hydration (Kosmatka and Wilson 2016).

The PEM initiative aims to improve performance of concrete while also catering to the concerns of contractors performing the work. Without being able to effectively transport, place, and finish concrete, even a mixture that has characteristics to support durable performance will suffer from issues associated with improper construction. Workability, flow, pumpability, and finishability are characteristics considered in performance engineered concrete mixture designs (Ley et al. 2014, Cook et al 2014, Wang et al. 2017, Cackler et al. 2017a and 2017b). Durable concrete should be handled with care and placed/finished in accordance with project specifications and quality assurance provisions.

2.3 Performance Requirements and Tests for Durable Concrete

Performance-related specifications provide the ability for agencies to obtain the desired construction quality while allowing contractor greater control and flexibility (Ahlstrom 2016). For instance, current prescriptive specifications for minimum cement content and rate of strength gain may preclude the acceptance of mixtures that have superior economy, durability, and satisfactory mechanical performance, but contain high proportions of SCMs.

In September 2016, a proposed AASHTO provisional specification, AASHTO MP XX-17, "Standard Specification With Commentary for Performance Engineered Concrete Pavement Mixtures" was submitted to AASHTO member states for balloting (AASHTO 2016). This specification, which has been updated and reissued yearly between 2020, has become AASHTO PP 84, "Standard Practice for Developing Performance Engineered Concrete Pavement Mixtures." At the time of the start of this work, AASHTO PP 84-17 was the current standard, with PP 84-18 and PP 84-19 released during the course of the project. AASHTO PP 84 provides a framework and guidance for state highway agencies to develop a specification for performance engineered concrete mixtures that focuses on measurement and acceptance of concrete based on characteristics that have been linked to satisfactory long-term durability performance of the concrete. As stated in

AASHTO PP 84-19, "A significant barrier to adoption of pure performance-based specifications, or even performance-related specifications, is the lack of effective test methods that assess the ability of a concrete mixture to resist the environment to which it is exposed. To address this barrier, new testing methods that measure performance-related parameters have been developed and are being evaluated in the field, while other advancements are emerging." Per AASHTO PP 84, Concrete performance parameters that should be addressed in a materials specification include:

Sufficient strength

- Low risk of cracking and warping due to drying shrinkage
- Resistant to damage from freeze-thaw stresses
- Resistant to ingress by chemical deicers and other deleterious agents
- Low absorption, diffusion, and other transport related properties
- Aggregate stability to resist D-cracking and alkali-aggregate reactivity (AAR)
- Workable

Performance-related specifications require measurement of key properties and performance characteristics. In order for performance specifications to be successfully utilized, QA/QC tests should be rapid, effective, reliable, and inexpensive (Taylor et al. 2014, Cackler et al. 2017b). A number of state agencies, including NCDOT, are using and evaluating new, rapid, early-age testing technologies such as resistivity, sorptivity, and air void system analysis that support development and use of performance-engineered concrete mixtures. The capabilities of these tests to evaluate the durability performance of concrete mixtures is improving as state highway agencies build sufficient data to correlate the test results with durable field performance. Ongoing concrete materials research is beginning to provide NCDOT data to support use of performance engineered concrete mixtures.

Durability tests traditionally aim to evaluate concrete's ability to conduct three mediums: air, water, and electricity (Milla et al. 2020). The less conductive concrete is to these mediums, the more likely the concrete can resist the ingress of harmful substances and deterioration. Water permeability can cause poor freezing and thawing results as well as allow the ingress of chlorides, sulfates, and other deleterious substances into the concrete. It is important to note that the conditioning of the concrete test specimens (particularly moisture saturation level, pore chemistry, and temperature) can greatly influence durability testing results (Hearn et al. 2006).

Water and Air Permeability

Permeability of concrete is directly related to pore structure and concrete density. The denser the paste matrix is developed within the concrete, the less permeable the concrete mixture. Water absorption testing is used to determine the permeability of concrete in regard to water. ASTM C1585 "Measurement of Rate of Absorption of Water by Hydraulic-Cement Concretes" outlines the test procedures for finding the concrete's sorptivity by means of ponding water on one surface of a specimen (ASTM 2013). Sorptivity is defined as the action of absorbing and transmitting water by means of capillary force in a porous material (Hall 1989). The test procedures in ASTM C1585 are used to evaluate concrete absorption with only one face exposed to water. This method provides an accurate representation of the surface exposure of a concrete structure or pavement. Sorptivity is influenced by a number of factors including the mixture proportions of the concrete (as well as presence or absence of chemical admixtures or SCMs), physical characteristics and chemical composition of mixture inputs, content of entrained air, methods of concrete placement, methods of finishing, curing quality, age, microcracking in the concrete, surface treatments, and moisture conditions (ASTM 2013).

Electrical Tests

Over the past decades, electrical tests have emerged as an effective method to evaluate the resistivity (and hence permeability) of concrete. In 1982, the AASHTO standard for RCPT was presented and approved as AASHTO T 277, "Standard Method of Test for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration" (AASHTO 2015). ASTM also endorsed the test method in 1991, producing ASTM C1202 or AASHTO T 277, "Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration" (ASTM 2018, AASHTO 2015). The test method presented in both of these standards is commonly known as the rapid chloride permeability test, or RCPT. Surface resistivity testing using the Wenner probe offers a vast improvement in time and cost, requiring minutes to obtain results, compared to the several days required to perform the RCPT (AASHTO 2015). Surface resistivity tests measure the electrical resistivity of water-saturated concrete, and can be used to evaluate a wide array of concrete characteristics (Morris et al. 1996). In fact, Polder's study was able to relate the likelihood of steel reinforcement corrosion to the resistivity of various concrete samples and structures (Polder 2001).

The test method for measuring surface resistivity is outlined in AASHTO T 358. A four-pin Wenner probe is used to pass an alternating current (AC) across the surface of a concrete structure or specimen. The two outer pins provide the current flow, and the potential difference is measured between the two inner pins. The results are presented in kΩ-cm (AASHTO 2017). Recently, results obtained through surface resistivity testing have been shown to correlate well with test results from RCPT (Rupnow and Icenogle 2012). Factors that influence surface resistivity results include moisture content, composition of the concrete, permeability, age, and temperature (Morris et al. 1996, Polder 2001, Presuel-Moreno et al. 2010, Liu et al. 2010).

Shrinkage Tests

AASHTO PP 84 identifies volume of paste, unrestrained volume change testing, restrained shrinkage, and cracking potential as influencing factors for the pavement's ability to resist warping and cracking caused by shrinkage. Of the suggested tests, only one should be selected for project specifications. AASHTO PP 84 includes prescriptive options for reducing shrinkage, which are limiting paste content of the concrete to 25%, or testing for unrestrained volume change (AASHTO 2019). AASHTO T 160, "Standard Method of Test for Length Change of Hardened Hydraulic Cement Mortar and Concrete" (also ASTM C157, "Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete") is the specified test method for unrestrained volume testing, with a target value of less than 420 microstrain at 28 days (AASHTO 2017, ASTM 2017). AASHTO PP 84 Section 6.4.2 provides alternative performance specifications for testing. As opposed to the prescriptive specification for the same test, target values are 360, 420, or 480 microstrain at 91 days depending on the application. This linear (volumetric) shrinkage test is commonly utilized and easily performed, although in recent years the shortcomings of this test (including its inability to capture early age autogenous and chemical shrinkage potential) have led many agencies to question its usefulness (ASTM 2019, Weiss 2017).

Previous editions of AASHTO PP 84 (2017) provided provisions for use of restrained shrinkage testing, although this has been removed from the 2019 edition of AASHTO PP 84 as additional research in this area is ongoing. Restrained shrinkage testing was specified using either AASHTO T 334, "Standard Method of Test for Estimating the Cracking Tendency of Concrete", or AASHTO T 363, "Standard Method of Test for Evaluating Stress Development and Cracking Potential due to Restrained Volume Change Using a Dual Ring Test" (AASHTO 2017). These tests are similar to ASTM C1581, "Standard Test Method for Determining Age at Cracking and Induced Tensile Stress Characteristics of Mortar and Concrete under Restrained Shrinkage" (ASTM 2018). If specifying using AASHTO T 334, the suggested target value was no cracking at 180 days (AASHTO 2017). AASHTO T 363 should have stress results less than 60% of splitting tensile strength for 7 days. Computational programs can also be used to evaluate cracking potential (AASHTO 2008). Computational programs should have a determined cracking probability of less than 5%, 20%, or 50% depending on curing conditions and the application.

3.0 DATA ANALYSIS OF NORTH CAROLINA MIXTURES

Many thousands of mixture designs have been approved by the NCDOT to perform in specific applications, geographic locations and construction conditions. In many cases, details from these mixture designs can be linked to the record of condition assessments and maintenance records of facilities that have been built of these concrete mixtures. The analysis presented in this chapter presents correlations between mixture design details, early-age test results and long-term performance. Identifying trends in performance can lead to improved specifications which increase the lifespan of concrete bridge elements and pavements, reduce maintenance costs, and allow for safer roadways.

3.1 Data Sources

A variety of identifiers, such as mixture design IDs, project contract numbers, structure ID numbers and pavement locations were linked to combine the contents of several datasets as shown in Figure 3.1, and are described below.

Figure 3.1: Data linking sequence

Concrete Mixture Database- The concrete mixture database contains details of all designs submitted to for approval since 1999. This includes the proportions of materials used, the producer of the materials and the concrete itself, the class of the concrete, the mixture ID, the status of the mixture design (active or expired), dates accepted/expiring, etc.

Concrete Early Age Test Data- For each project related to NC bridges, a record of test results for fresh and hardened concrete is included in a central concrete test data spreadsheet. This dataset included the early age test data (slump, air content, and 28-day compressive strength), as well as information such as the contract number, material description, sample data, site description, county, and route number for bridges between 1999 and 2018. Due to the large number of projects (nearly 4000 unique site descriptions), as well as multiple tests performed on each, the original version of this spreadsheet was large (over 200,000 records). For the purposes of this study, this database will be referred to as the "*Bridge Early Age Data*" database. A similar version of this database was created for the concrete pavement test data. For this database, in addition to the results of fresh concrete testing, the early age test data includes both flexural strength and compressive strength, with records spanning from 2000-2018. Because rigid concrete pavements are not as frequently constructed as asphalt pavement in NC, and the propensity to use a common mixture design for a single project, the original spreadsheet included 29 individual roadways, and only had around 7,400 records for concrete pavement projects. For the purposes of this study, this database will be referred to as the "*Pavement Early Age Data*" database.

NCDOT also utilizes several databases to house records on bridge and pavement condition and maintenance, repair, and rehabilitation (MR&R) actions performed.

BMS Network Master- The Network Master contains information about the location (county, intersection features, latitude, and longitude), structure number, and route number for each NCDOT-maintained bridge in the state. This dataset is updated annually to ensure accuracy. The version of the Network Master used for this project was exported in February 2018 to an Excel spreadsheet. At the time of export, it included 21,835 records, divided among 3,965 bridges and culverts.

BMS Element Data- To create uniformity in the way that inspection data was recorded and reported, AASHTO created the Bridge Element Inspection Guide Manual. The goal of the manual is to "completely capture the condition of bridges in a simple was that can be standardized across the nation while providing the flexibility to be adapted to both large and small agency setting" (AASHTO 2010). The element set includes two different element types, the National Bridge Elements (NBE) and the Bridge Management Elements (BME). NBE represents the primary structural components of bridges, such as the deck, superstructure, substructure, bridge rail, bearings, etc. BME includes the other bridge components such as joints, wearing surfaces, protective systems, and other non-structural elements. All elements, no matter what type, have a standard number of condition states, comprised of good, fair, poor, and severe.

Pavement Management System (PMS)- The pavement management system (PMS) database is one of three parts in the Pavement Management Unit (the other two being Data Collection and Pavement Design & Analysis), a unit responsible for the design, testing, and monitoring of the pavements in the NCDOT network. The PMS is utilized to store and analyze pavement condition data, maintain records for the construction or maintenance of NCDOT roadways, and support analysis of pavement data to assist in the optimization of department funds. For this study, the PMS Network Master was used. Within the Network Master roadways are divided into smaller sections. For each section, details such as location, length, surface type, and condition are provided. The version of the Network Master used for this project was exported in February 2019 to an Excel spreadsheet, and was pre-screened to remove sections of roadway that do not have concrete as the upper layer. At the time of export, it included 3,610 records, divided amongst 29 roadways.

Long Term Pavement Performance (LTPP) Database- As pavement performance data is one of the major research areas of the Strategic Highway Research Program (SHRP), the Long-Term Pavement Performance (LTPP) program was established to aid in this effort. Fifty sections of roadway involved in this study exist in North Carolina, located in areas throughout the state. The database includes information such as average climate, weather, traffic estimates, signs of distress, cracking, fracture, deflection, etc. for most if not all of those sections of pavement. This database provides detail regarding the specific distresses observed in the concrete pavement, and assisted in the evaluation of their long term performance.

3.2 Compliance with Existing Prescriptive Specifications

Compliance of concretes used in bridge decks and pavements with their required early age performance on, air content, slump and compressive strength tests was evaluated. Deviations from the required performance are correlated with the target specified value and the mixture design details to detect patterns that can inform future specifications.

3.2.1 Bridge Deck Concrete

Acceptance of concretes that test outside of the specified air content, maximum slump, and minimum compressive strength requirements can lead to future performance issues because there is an increased likelihood that the concrete will not be adequately placed and consolidated (for low slumps), or could exhibit segregation or other issues (for high slumps). Chapter 10 of NCDOT's specifications contains the requirements for typical early age concrete tests (NCDOT 2018). A summary of the relevant components of that table are shown in Table 3.1.

Table 3.1: Excerpt from NCDOT Table 1000-1

Type of Concrete	Air (%)		Max Slump (in)		Strength (psi)	
	Between X of target 5.0%		Vibrated	Non	Day	Min f'c
Class AA	-1.5	1.5	3.5	-	28	4500
Class AA, Slip-form Barrier Rail	-1.5	1.5	1.5	-	28	4500
Drilled Shaft	-1.5	1.5	7	9	28	4500
Class A	-1.5	1.5	3.5	-	28	3000
Latex Modified Concrete	-1.5	1.5	6	-	7	3000
Flowable Fill	-1.5	1.5	-	-	56	150
Pavement	-1.5	1.5	1.5	-	28	4500
Prestress	-1.5	1.5	8	-	-	-
High Early-strength Patching Mix	-1.5	1.5	-	-	-	-
Class AAA	-1.5	1.5	-	-	-	-

Analysis of early age test results determined that most of the accepted and placed mixtures comply with the standards and specifications described in Table 3.1. Table 3.2 provides a summary of the data and shows the number of data points that were available in the dataset for each class of concrete and each early age test.

Table 3.1: Percent of mixtures meeting early age test targets from NCDOT specifications

	Concrete Type		Total Number of Records	% Within Target Range
Air	1	Class AA	15,073	97.5
	2	Class AA, Slip-form Barrier	1,878	99.1
	3	Drilled Shaft	8,581	89.1
	4	Class A	13,913	98.4
	5	Latex Modified Concrete	949	94.0
	6	Flowable Fill	2	100.0
	7	Pavement	102	100.0
	8	Prestress	7	100.0
	9	High Early-strength Patching	18	100.0
	10	Class AAA	28	67.9
Slump	1	Class AA	15,073	79.8
	2	Class AA, Slip-form Barrier	1,878	90.4
	3	Drilled Shaft	8,581	98.2
	4	Class A	13,913	86.9
	5	Latex Modified Concrete	949	86.2
	7	Pavement	102	42.7
Compressive Strength	1	Class AA	15,073	92.5
	2	Class AA, Slip-form Barrier	1,878	97.4
	3	Drilled Shaft	8,581	96.5
	4	Class A	13,913	99.1
	5	Latex Modified Concrete	949	99.4
	6	Flowable Fill	2	100.0
	7	Pavement	102	95.1

Influence of Mixture Design Characteristics on Early Age Test Results

Relationships between the following mixture design characteristics and early age test result outcomes were analyzed using a canonical correlation technique (Lukavsky 2019):

- Class of concrete
- Mortar content
- Cement approved producer
- Cement amount
- Pozzolan amount
- Pozzolan approved producer
- Pozzolan type
- Fine aggregate SG
- Fine aggregate amount
- Coarse aggregate amount
- Coarse aggregate SG
- Water amount
- Latex modifier amount
- Specified air content
- Specified slump
- *w/cm* ratio
- Yield
- Paste content
- Aggregate content

Since data on incidental conditions such as mixing time, ambient temperature, etc., were not available for inclusion in the analysis, the analysis assumes these were each within acceptable ranges. Correlation results are presented in Table 3.3. They can be interpreted as describing three potential types of relationships between variables:

1. -1 to 0: Correlations falling in this range indicates a negative relationship between the variables. This means that as one increases, the other decreases.
2. 0: A correlation value of 0 indicates no relationship at all.
3. 0 to 1: Correlation coefficients in this range indicate a positive relationship between the variables. This means that as one increases, the other increases as well.

Greater absolute values of the correlation imply stronger relationships between the two variables. For instance, in Table 3.3, “Paste Content” has a stronger correlation to the difference in air content than “Yield” does.

Table 3.3: Correlation and regression results for difference in air content

Correlation Rank	Variable	Correlation to Air Content Difference
1	Design Air Content	-0.1417
2	Design Slump	0.1282
3	Paste Content	-0.1217
4	Fine Aggregate Amount	0.1108
5	Aggregate Content	0.0812
6	Coarse Aggregate Amount	-0.0786
7	Mortar Content	0.0630
8	Latex Amount	0.0536
9	Coarse Aggregate Specific Gravity	-0.0440
10	Water Amount	-0.0425
11	Yield	-0.0308
12	W/CM Ratio	-0.0122
13	Cement Amount	0.0089
14	Pozzolan Amount	-0.0025
15	Fine Aggregate Specific Gravity	0.0013

Discussion of analysis results:

1. The design air content has a negative relationship with the difference between the design and actual air content values. This relationship may indicate that the higher the design air content, the harder it is to actually achieve.
2. The design slump value has a positive relationship with the difference between the design and actual air content values. Practically, this means that when the design slump is increased, there is greater likelihood for the air content within the concrete to be larger than design value.
3. The paste content, a combination of the design cement, pozzolan, water, and air amounts, has a negative relationship with the difference between design and actual air contents. This agrees with the first factor (as air content is a component in the paste content).
4. The increase in fine aggregate amount has a positive relationship with the difference between design and actual air content. Since well-rounded fine aggregate particles can lead to higher air entrainment, increasing the amount of fine aggregate should positively increase the amount of air in the concrete. This relationship also indicates that suppliers tend to provide fine aggregate more in the middle size fractions (passing through the

No. 30 sieves but retained on the No. 50), as testing has shown that in general, increasing the amount retained at the No. 100 sieve instead of the No. 50 sieve leads to decreased air content (Malisch 1996).

Table 3.4 provides a similar analysis with relation to the slump of fresh concrete. The following observations arise from the values in the table.

1. The design air content had a relatively strong positive correlation with the difference between design and actual slump values. This agrees with results from the difference in air content values, and practically means that an increase in air content (whether or not it was intended) leads to a higher slump, sometimes higher than desired.
2. The design slump has a negative relationship with the difference between the design and actual slump value, indicating the same principle as shown with the relationship between design air and the difference between design and actual air content: if the design value is increased, the likelihood of that value not being met is also increased. Practically, this is not a currently a major concern for NC bridges since, for most concrete types, at least 80% of the accepted mixtures are within the required range.
3. The cement content has a negative relationship with the difference between design and actual slump, meaning that adding more cement to the mixture decreases the slump.
4. The *w/cm* ratio has positive relationship with the difference between design and actual slump amount, meaning that as the ratio is increase (either by water content increasing or cement value decreasing), the mixture becomes more workable and the slump itself increases.

Table 3.4: Correlation and regression results for difference in slump

Correlation Rank	Variable	Correlation to Slump Difference
1	Design Air Content	0.3956
2	Design Slump	-0.2347
3	Cement Amount	-0.2154
4	*w/cm* Ratio	0.0920
5	Water Amount	-0.0912
6	Paste Content	0.0907
7	Yield	0.0788
8	Latex Amount	-0.0691
9	Fine Aggregate Amount	-0.0676
10	Aggregate Content	-0.0669
11	Fine Aggregate Specific Gravity	-0.0428
12	Coarse Aggregate Specific Gravity	-0.0422
13	Pozzolan Amount	0.0309
14	Mortar Content	-0.0305
15	Coarse Aggregate Amount	0.0043

Correlations between concrete mix design characteristics and compressive strength are shown in Table 3.5. Within the dataset, most of the deviations from the specified strength are positive, indicating the minimum strength capacity is usually exceeded. Therefore, when there is a negative correlation, it does not always indicate that increasing one variable or another will cause the compressive strength to fail to match the minimum requirement; instead, it might simply indicate that it will reduce the compressive strength to a level that is still acceptable. Observations from Table 3.5 are described below:

1. The amount of fine aggregate in the mixture design has a relatively strong positive correlation to the change in compressive strength, indicating that increasing the amount of fines in a mixture tends to increase the strength, in many cases above the design compressive strength. The SG of the fine aggregate, and thus the size of the fine aggregate, is not as important to the strength capacity, but increasing the total volume of fines tends to lead to higher increases in compressive strength.
2. Paste content has a negative relationship with the difference in compressive strength, so increasing the paste content increases the chance that the compressive strength will not be as high. Paste content is a combination of cement content (positive), pozzolan amount (negative), water amount (negative), and latex (positive). The

two positives, cement content and latex, are either very lowly correlated or not present in many of the mixtures, which is why the negatives control this variable.

3. While mortar content is a combination of cement amount and pozzolan amount, the relationship between this variable and the difference in compressive strength is positive. This indicates that increasing the mortar content, by increasing the cement amount and not the pozzolan amount, tends to lead to higher compressive strengths.
4. The coarse aggregate amount has a negative relationship with the difference between design and actual compressive strength, which is logical because too many large particles can increase the likelihood of void spaces, as well as decrease the space available for the binding paste.

Table 3.5: Correlation and regression results for difference in compressive strength

Correlation Rank	Variable	Correlation to Compressive Strength Difference
1	Fine Aggregate Amount	0.2819
2	Paste Content	-0.2526
3	Mortar Content	0.2189
4	Coarse Aggregate Amount	-0.2018
5	Latex Amount	0.2000
6	Pozzolan Amount	-0.1733
7	*w/cm* Ratio	0.1535
8	Water Amount	-0.1210
9	Design Slump	0.1092
10	Aggregate Content	0.1051
11	Fine Aggregate SG	0.0805
12	Design Air Content	-0.0651
13	Yield	-0.0558
14	Coarse Aggregate SG	-0.0296
15	Cement Amount	0.0273

3.2.2 Pavement Concrete

While pavement mixtures have similar categories for early age requirements as bridge mixtures, not all of them are recorded in the database. In Division 10 of the Standard Specifications for Roads and Structures, NCDOT states that for pavements, “Use a mix that contains a minimum of 526 pounds of cement per cubic yard, a maximum water cement ratio of 0.559, an air content in the range of 4.5 to 5.5 percent, a maximum slump of 1.5", a minimum flexural strength of 650 psi at 28 days and a minimum compressive strength of 4,500 psi at 28 days (NCDOT 2018).” This is consistent with the NCDOT minimum compressive strength for Concrete Class AA, Concrete Class AA Slip-Form, and Drilled Pier. The following are the minimum required values for early age pavement concrete:

- Flexural Strength = 650 psi (minimum)
- Compressive Strength = 4500 psi (minimum).

Several of the mixtures in the dataset were not designed to meet the initial threshold value of 650 psi, with only 405 out of 4,942 recorded mixtures designed for this threshold. Through personal communication with Brian Hunter, the State Laboratory Operations Manager for the NCDOT Materials & Tests unit, this is due to the fact that prior to 2002, the specifications called for 550 psi at 14 days, which was then raised to 600 psi before settling at the current target of 650 psi at 28 days (Hunter 2019). Since the age of testing is not included in the database, it will be assumed that these mixtures that require a lower flexural strength are measured at 14 days.

For mixtures that required a flexural strength of lower than 550 psi, many records indicated that the purpose was for concrete repair, or something similar. In this case, the strength requirement is a 3-day strength requirement, since the roadway needs to be operational as soon as possible. Therefore, because of the variance in strength requirements and the unknown date of testing, the listed required strength will also be the assumed NCDOT required strength for flexural strength tests. A large number of records listing flexural strength value of 0 psi were removed from the dataset.

Table 3.6 displays a summary of the analysis. This summary is presented in a manner that shows the number of data points for each concrete type for each early age test. Just over 750 records for Concrete Pavement – E have a reported flexural strength value of 0. There are two different “types” of pavement represented here, “M” and “E”. These stand for

“Metric” and “English” and refer to other components of the contract. The mixture design and test results are all in US customary units, so while they are separated

Table 3.6: Percentage of mixtures meeting early age test targets from NCDOT specifications

	Concrete Type		Total Number	% Within Target Range
Flexural Strength	1	Concrete Pavement - M	2,564	94.3
	2	Concrete Pavement - E	1,596	91.3
	3	Concrete Pavement- Beams - E	25	100.0
Compressive Strength	2	Concrete Pavement - E	1,545	96.8
	3	Concrete Pavement- Beams - E	938	99.0

Statistical regression analysis for the correlation of mixture design details to each of the early age components (flexural strength and compressive strength) was performed to identify which components of the pavement mixture were most influential to the important early age concrete properties. In multiple linear regression modeling, there are the two sets of variables: the independent variables, represented by the mixture design information, and the dependent variables, represented by the early age properties. The data pertaining to this was imported from Excel into Minitab, to allow for the performance of multiple linear regression. To determine the most influential mixture design variables for each of the early age properties, the following mixture design components were evaluated.

- Class of concrete
- Mortar content
- Cement amount
- Pozzolan amount
- Pozzolan type
- Fine aggregate SG
- Fine aggregate amount
- Coarse aggregate SG
- Coarse aggregate amount
- Water amount
- Air content
- Slump
- *w/cm* ratio
- Yield
- Paste content
- Aggregate content

To account for mixing conditions that are not available in the dataset, the following assumptions were made:

- The prescribed mixing times are followed during the mixing of the concrete
- The surrounding air temperature during the mixing process was between 50°F and 95°F (10°C to 35°C) (except where other temperatures are required by Articles 420-8, 420-9 and 420-15) (NCDOT 2018)
- The concrete is mixed using the prescribed amounts, and the number of times that was exceed for water content (i.e. approaching maximum water content) is minimal and can be disregarded.

To determine which parameters of the pavement mixture designs were most influential to the concrete strength (flexural and compressive), and how those variables interact to either cause the strength to be more or less than designed, canonical correlation was performed using the SAS software package, while stepwise regression was performed in Minitab. The following section presents the results separated by the early age test variable examined (difference in flexural and different in compressive strength) and shows the condensed version of the results from canonical correlation and stepwise regression, combining them into a single table (Table 3.7) for each independent variable. Interpretation of the correlation results is as follows:

1. -1 to 0: Correlations falling in this range indicates a negative relationship between the variables. This means that as one increases, the other decreases.
2. 0: A correlation value of 0 indicates no relationship at all
3. 0 to 1: Correlation coefficients in this range indicate a positive relationship between the variables. This means that as one increases, the other increases as well.

The closer to 1 in the positive and -1 in the negative region indicates a stronger relationship between the two: i.e. “Mortar Content” has a stronger correlation to the difference in flexural strength than “Yield” does.

Table 3.7: Correlation results for difference in flexural strength

Correlation Rank	Variable	Correlation to Flexural Strength Difference
1	*w/cm* ratio	0.158
2	Pozzolan amount	-0.153
3	Water amount	0.133
4	Yield	-0.122
5	Fine aggregate amount	0.122
6	Mortar aontent	0.114
7	Coarse aggregate amount	-0.107
8	Cement amount	0.105
9	Fine Aggregate SG	0.103
10	Design slump	-0.017
11	Aggregate content	-0.008
12	Coarse Aggregate SG	-0.007
13	Paste content	0.007
14	Design air content	0.005

Observations regarding this analysis are as follows:

1. *w/cm* ratio is the most correlated variable, with a positive relationship to the difference in flexural strength. This finding is supported by the fact that water amount is also positively correlated, as an increase in the *w/cm* ratio indicates either an increase in water amount or a decrease in cement content.
2. Pozzolans have been shown by numerous studies to increase the compressive strength of concrete (as confirmed in the difference in compressive strength column). Their impact on flexural strength has not been studied as widely, but some research suggests that it can increase the flexural strength in laboratory conditions (Akbulut and Akoz 2006). For NC concrete, only Class C Fly Ash and Class F Fly Ash are used. The negative correlation results indicate that for NC mixtures, increasing the pozzolan amount leads to lowered (but not necessarily lower than required) flexural strength. Given that pozzolans hydrate more slowly than cement, and the test date is not currently adjusted for fly ash mixtures, this finding could be expected.
3. Yield has a negative relationship with the change in flexural strength. Yield is positively correlated with another negative variable, coarse aggregate amount, indicating the two may be associated: as design amount of coarse aggregate amount increases, the yield also increases, and the overall difference in flexural strength decreases.
4. Fine aggregate amount has a positive correlation with flexural strength, indicating that increasing the fine aggregate content typically increases the strength above the design amount. This, combined with the fine aggregate SG (positively correlated at #9 in the list, and also included in the regression equation) indicate that for NC mixtures, increasing the fine aggregate content as well as ensuring good gradation increase the concretes flexural strength.

Of these top three variables, two exhibit positive correlations and one exhibits a negative correlation. Increasing the water amount, and thus the *w/cm* ratio, tends to increase the difference in flexural strength. However, increasing the pozzolan amount does not tend to lead towards higher flexural strength for NC mixtures. This could be due to the fact that fly ash begins to show its full impact on the strength of concrete after the initial 28-day strength test (Mehta and Montiero 2014, Harison et al. 2014) so its full impact cannot be seen at the age which the concrete is typically tested.

About 97.6% of the mixtures included in the data set either met or exceeded minimum compressive strength requirements. Therefore, while a negative correlation indicates that as one variable increases the other decreases, it does not indicate that increasing a variable like slump (which has a negative correlation to the difference between design and actual) will cause the compressive strength to be below the required amount. It may be more important to look at the factor that causes the strength to decrease, as excessive strength is not always useful.

Because the use of rigid concrete as pavement is not as common as flexible pavements, there were a relatively small number of datapoints available for study. Compressive strength test results in the early age database are entirely from only four Piedmont divisions and so the results are not representative of the variety of materials used across the state. The results, presented in Table 3.8, can be interpreted as follows:

1. Fine aggregate SG has a negative relationship with the difference between actual and required compressive strength. A negative correlation here means that while the compressive strength can still be larger than the design value (as it is in most cases), use of higher SG fine aggregate leads to an overall lowered compressive strength. There can

be many reasons for the difference in SG of the fine aggregates used in the state, including the producer who obtains it, or if it is natural or manufactured sand. The SG of manufactured sand is higher than that of natural sand (Megashree et al. 2016), so the results indicate that the use of manufactured sand leads to a lower difference in compressive strength. Manufactured sand has been shown to lead to higher compressive strengths (Vijayaraghavan and Wayal 2013), and these results do not contradict that observation.

2. Similar to common research findings, pozzolan amount has a positive correlation to the difference between design and actual compressive strength. Fly ash is the only pozzolan present in the pavement mixture designs covered by this dataset. As with much available research on fly ash (Harison et al. 2014) in concrete, NCDOT pavements that included it showed increases in the later-age strength of concrete.
3. Increasing the air content has a positive relationship with the difference amount, indicating that concrete designed for higher air content levels tend to have higher strength capacities. The design air contents for the mixtures used in this study ranged from 4.9% to 6.1%, which is not a large spread. Since over 95% of the data falls within the 5.0 ± 0.1%, the cases where the air content was higher are very few, and other components of the mixture that the air content correlates to could have also aided in increasing the compressive strength.

Table 3.8: Correlation and regression results for difference in compressive strength

Correlation Rank	Variable	Correlation to Compressive Strength Difference
1	Fine Aggregate Specific Gravity	-0.273
2	Pozzolan Amount	0.217
3	Design Air Content	0.185
4	Aggregate Content	-0.171
5	Coarse Aggregate Specific Gravity	0.168
6	Yield	0.157
7	Fine Aggregate Amount	-0.153
8	*w/cm* Ratio	-0.147
9	Paste Content	0.146
10	Cement Amount	-0.103
11	Mortar Content	0.094
12	Design Slump	-0.065
13	Coarse Aggregate Amount	0.041
14	Water Amount	0.005

3.3 Correlations Between Concrete Mixtures and Long Term Performance

In this portion of the analysis, the qualities of concrete mixtures were correlated with long-term performance. This analysis was only possible with concrete from bridges because the number of pavements available for consideration was too small to provide reliable results. The long-term performance of bridge decks, as indicated by their condition rating in the NBI, was compared to a general deterministic deterioration model prepared and reported by Goyal (2015) for NCDOT 2014-07 (Cavalline et al. 2015, Goyal et al. 2016, Goyal et al. 2019). This model, shown as the black trace in Figure 3.2, represents the average deterioration of condition rating for bridges with concrete decks in NC along with high and low limits that represent a range of time that each deck may remain within the same condition rating. In comparison to the general deterministic deterioration model for NC bridges, individual bridges were separated into groups labeled as underperforming or overperforming based on whether their condition rating after a period of service was above the high limit or below the low limit estimated by the model and shown in Figure 3.2.

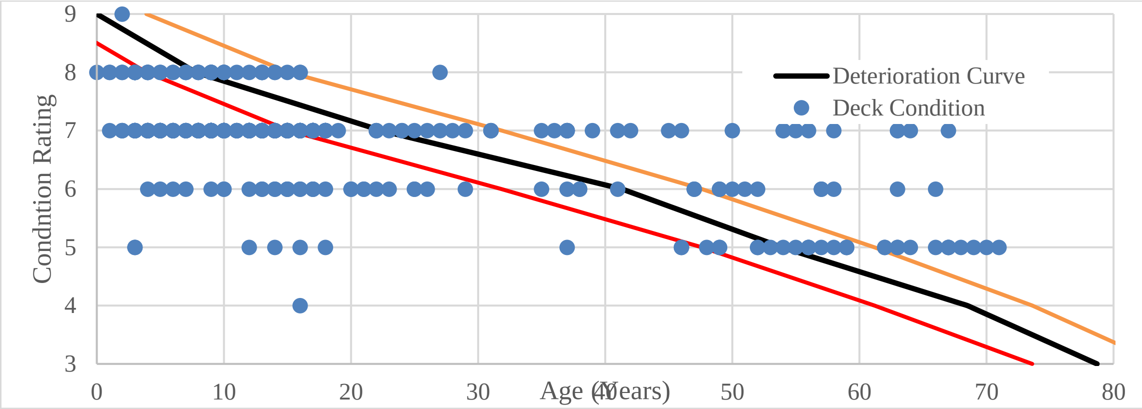

Figure 3.2: Bridge deck deterioration model

When applied to the data, the amount of data in each category is shown in Table 3.9, and the number of bridges per condition rating that fall into those categories is shown in Table 3.10.

Table 3.9: Number of bridge decks within specified ranges

Range	# of Bridges	# of Records
Under	283	12,077
Inside	120	4,783
Over	49	861

Table 3.10: Number of bridges per condition rating within specified ranges

Rating	# of Bridges		
	Under	Inside	Over
9	0	1	0
8	12	34	3
7	230	62	22
6	31	7	11
5	9	16	13
4	1	0	0
3	0	0	0

Table 3.11 lists the variables that were evaluated as potentially significant factors in bridge longevity. Variables identified as being important are ones for which the difference in the mean value of the characteristic between the under-performing and over-performing bridge groups is statistically significant. Therefore, a significant variable is one that the difference in the mean is significant. Significance was determined using the t-test. Also, a one-way ANOVA was used to determine if the group difference between the underperforming and overperforming bridges was significant with 80%, 90% and 95% levels of confidence.

Table 3.11: Characteristics of durable bridge decks

Variable	Mean Value		Significant at confidence level:		
	Under-Performing	Over-Performing	95%?	90%?	80%?
Mortar content (cu.ft.)	16.30	16.27	NO	NO	NO
Cement amount (lbs)	574.7	554.6	NO	NO	NO
Pozzolan amount (lbs)	142.5	164.0	NO	NO	YES
Class C fly ash (lbs)	---	110	---	---	---
Class F fly ash (lbs)	162.6	168.8	NO	YES	YES
GGBFS (lbs)	192	319	---	---	---
Fine aggregate (lbs)	1056.8	1028.5	NO	NO	YES
Fine aggregate SG	2.64	2.64	NO	NO	YES
Coarse aggregate (lbs)	1826.8	1805.7	NO	NO	NO
Coarse aggregate SG	2.74	2.70	NO	YES	YES
Water content (lbs)	32.59	33.37	YES	YES	YES
w/cm ratio	0.38	0.39	NO	YES	YES
Yield (cu.ft.)	27.01	27.04	NO	NO	NO
Paste content (%)	36.67	37.20	NO	YES	YES
Aggregate content (%)	63.33	62.80	NO	YES	YES

Only one variable, "Water content" is significant at a 95% confidence level. At the 90% confidence level, five more become significant: "Class F fly ash", "Coarse aggregate SG", "*w/cm* ratio", "Paste content", and "Aggregate content." An elaboration of these primary findings is below:

- The amount of water present in the mixture (variable "Water content") is highlighted by the data as the most important variable, with the mean increasing from under-performing to over-performing. The difference in water amount between the under and over performing groups, however, the variable is strongly linked to several other important factors, such as paste content, the *w/cm* ratio and fly ash content (all of which are significant at the 90% confidence level, and cause an increase in water content). This difference, although small, is illustrated in Table 3.12, which shows that overperforming decks with fly ash use slightly more water than those without.

Table 3.12: Comparison of mean water amounts for under/over-performing bridges that contain/do not contain fly ash

Cementitious Material	Mean Water Amount (lbs)	
	Under	Over
With Fly Ash	32.55	33.49
Without Fly Ash	32.88	33.00

- In laboratory and field settings, increasing the fly ash content has been proven to increase the overall durability of concrete by decreasing the permeability and reducing the alkali-silica reaction (Shafaatian et al. 2012, Taylor et al. 2013). The results of this study confirm this, as the mean amount of fly ash is greater for the over performing bridges. This indicates that increasing the fly ash amount does help improve durability of concrete not only in the lab, but also over the lifetime of a bridge while in service.
- The SG of the coarse aggregate is a significant variable at the 90% confidence level (and also at the 80% confidence level, where the variable no longer has equal variance). The difference in the mean of the SG is not large (only a decrease of 0.04 from under to over).
- AASHTO PP 84-19 lists several strategies for improving concrete durability (AASHTO 2019). While this document is written primarily for pavements, the ideas presented in it are generally applicable to all concrete.
    1. If shrinkage cracking caused by volume change due to changes of moisture (hygral volume change), then either the volume of paste should be limited to 25% or the unrestrained volume change should be less than 420 microstrain at 28 days.

2. For freeze-thaw durability, the water to cementitious material ratio (*w/cm*) should be less than 0.45, and the air content should be either between 5 and 8 percent or greater than 4 percent with a SAM number less than 0.20 using TP 118.
3. To reduce joint damage due to deicing chemicals when $CaCl_2$ or $MgCl_2$ is used, either SCM's should replace at least 35% of the cement by volume, or a sealer should be used consistent with AASHTO M 224.

For the bridge decks included in this study:

1. The volume of paste was consistently higher than this recommended minimum (typically in the low 30% range rather than around 25%). While this is typically the case for bridges (with higher slumps and a decreased maximum coarse aggregate size), this value is still high. Furthermore, the mean of the paste content for the over performing bridges was higher than for the under-performing bridges, and the mode value is also much higher. At a 90% confidence level this difference between the means became significant. As such, for the NC bridges that were included in this study, statistical findings indicated that lowering the paste content may not be beneficial for performance considerations. However, it may be that from a statistical perspective paste content is acting as a proxy for workability, and lower paste contents could still be suggested in order to improve durability, but producers should still endeavor to maintain adequate workability.
2. For Class AA mixtures, the air content is designed for either 5% or 6% air content, and with a tolerance of ±1.5%, 97.5% of the early age tests confirmed that this range was met. Therefore, the majority would fit either the greater than 4% or the between 5% and 8% requirement. For the *w/cm* ratio, the maximum included in this study was 0.43, falling below the recommended maximum. With a mean of 0.38 for the under-performing bridges and a mean of 0.39 for the over-performing bridges, at a 90% confidence level the difference in means is significant, but does not indicate that continuing to elevate the *w/cm* ratio will lead to better performance.
3. The mean difference between the amount of Class F fly ash used increased from under to over performing bridge decks (from 162.6 lbs to 168.8 lbs), and this difference is significant at the 90% confidence level. 25% of the mixtures in the under-performing category are below 163 lbs. As mentioned previously, at least 75% of the mixtures use fly ash, and since it has been shown that use of Class F fly ash above the recommended minimum, as well as an increase between the means from under to over performing bridges, continuing to add at least 35% replacement of cement with SCM's is recommended.

4. LABORATORY TESTING PROGRAM AND RESULTS

4.1 Materials Description and Characterization

Materials used were selected due to their common use, particularly in the Piedmont region of the state, their physical properties and performance characteristics being representative of other materials, and their selection for use in previous research studies (providing continuity in datasets across several research studies).

4.1.1 Cementitous Materials

A Type I/II ordinary portland cement (OPC) meeting ASTM C150 was used in this research. This cement was produced in Holly Hill, South Carolina, and is a commonly used cement for the Coastal and Piedmont regions of NC. This OPC is sourced from the same production facility as the OPC used in RP 2016-06 and Cement B used in RP 2015-03. A mill reports for the cement is provided in Appendix B as Figure B.1. The PLC used in this research is a Type IL cement meeting ASTM C595 and AASHTO M 240, that was produced at the same Holly Hill, South Carolina facility. The PLC was produced using the same clinker as the OPC, with less than 15% limestone added per ASTM C595. The mill report for this PLC is also provided in Figure B.2 in Appendix B.

Fly ash used in this study was a Class F ash sourced from the Roxboro Power station in Roxboro, NC. NCDOT Standard Specifications allow for substitution of 1 pound of Class F fly ash per pound of cement replaced up to 30%. Additional information is provided in Figure B.3 in Appendix B.

4.1.2 Aggregates

Aggregates for this project met the requirements of ASTM C33. The coarse aggregate was a granitic gneiss (SG of 2.63 and absorption of 0.40%) sourced from a quarry near Cary, NC. The coarse aggregate met the gradation requirements of No. 67 stone. The fine aggregate used for this study was a natural sand (SG 2.61, absorption 0.40%, and a fineness modulus of 2.65). Sieve analysis results for the coarse and fine aggregates are shown in Appendix B, Tables B.1 and B.2.

4.1.3 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 for mixtures was 3.5 inches, although reasonable variations to this target slump were accepted in order to achieve the target *w/cm* while maintaining the selected material proportions. NCDOT specifications allow an air content for pavement mixtures of 5.0% ± 1.5%, and for structural concrete 6.0% ± 1.5%. However, 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.

4.2 Concrete Mixtures

The testing program for this work was designed to support development of recommended specification provisions for surface resistivity, early age opening to traffic, and shrinkage. Highly influential in development of the integrity of the paste structure of concrete are the *w/cm* ratio, total cement or cement/SCM content, and fly ash replacement percentage. These parameters were the primary focus in developing the mixture matrix for this work, shown in Figure 4.1.

- Three *w/cm* ratios – 0.37, 0.42, and 0.47
- Three cement contents – 700 pcy, 650 pcy, and 600 pcy
- Two fly ash replacement levels – 20% and 30% by weight

Higher cementitious content mixtures (700 pcy and 650 pcy, shown in orange and yellow, respectively) are typical of bridge mixtures (NCDOT Class AA). Lower cementitious content mixtures (600 pcy, shown in green) are typical of lower cementitious content AA mixtures and pavement mixtures. Twenty-one of the 24 mixtures utilized OPC, while 3 lower cementitious content (low AA and pavement) mixtures utilized PLC.

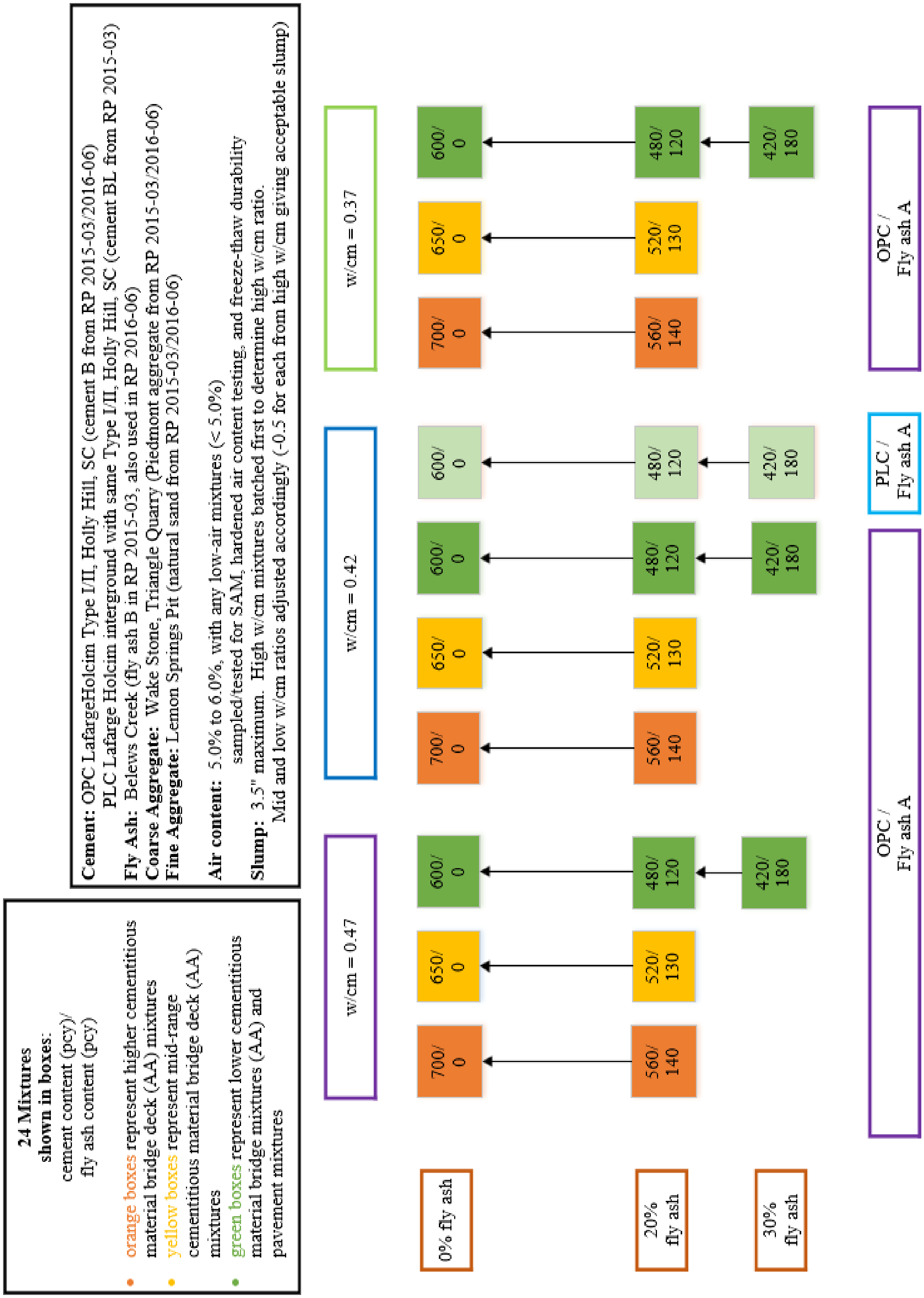

Figure 4.1: Concrete mixture matrix with supporting details

Utilizing the established *w/cm* and cementitious materials contents, along with the aggregate properties measured, the remainder of mixture proportions were computed using the ACI 211.1 methodology. Mixture materials and proportions are shown in Table 4.1, with colors shown in the table corresponding to the colors of the boxes in Figure 4.1. The concrete mixtures developed for this project were each given a mixture ID, with the convention W-XXX-YYY, which is summarized as follows:

- W is the *w/cm* ratio (H = high = 0.47, M = medium = 0.42, L = low = 0.37)
- XXX is the cement content in pcy
- YYY is the fly ash content in pcy

Table 4.1: Concrete mixtures with material proportions

Mixture ID **W-XXX-YYY,** where **W** is *w/cm* ratio, **XXX** is cement content, **YYY** is fly ash content	**Mixture Characteristics**			**Mixture Proportions, pcy**					
	Mixture type	**Cement type**	***w/cm***	**Fly ash replacement (%)**	**Cement**	**Fly ash**	**Coarse aggregate**	**Fine aggregate**	**Water**
H-700-0	AA (high and medium cm content)	OPC	0.47	0	700	0	1659	1072	329.0
H-560-140				20	560	140	1659	1022	329.0
H-650-0				0	650	0	1659	1175	305.5
H-520-130				20	520	130	1659	1129	305.5
H-600-0				0	600	0	1659	1277	282.0
H-480-120				20	480	120	1659	1235	282.0
H-420-180				30	420	180	1659	1214	282.0
M-700-0			0.42	0	700	0	1659	1163	294.0
M-560-140				20	560	140	1659	1114	294.0
M-650-0				0	650	0	1659	1259	273.0
M-520-130				20	520	130	1659	1214	273.0
M-600-0				0	600	0	1659	1356	252.0
M-480-120				20	480	120	1659	1313	252.0
M-420-180				30	420	180	1659	1292	252.0
M-600P-0		PLC		0	600	0	1659	1356	252.0
M-480P-120				20	480	120	1659	1313	252.0
M-420P-180				30	420	180	1659	1292	252.0
L-700-0	AA (low cm content) and Pavement	OPC	0.37	0	700	0	1659	1254	259.0
L-560-140				20	560	140	1659	1205	259.0
L-650-0				0	650	0	1659	1344	240.0
L-520-130				20	520	130	1659	1298	240.0
L-600-0				0	600	0	1659	1434	222.0
L-480-120				20	480	120	1659	1392	222.0
L-420-180				30	420	180	1659	1370	222.0

4.3 Testing Program and Results

The testing program for fresh and hardened concrete properties performed is summarized in Table 4.2. In addition to tests to confirm the fresh properties met the targets described previously, tests were performed to evaluate the fresh mechanical properties, thermal properties, and durability performance of each of the eighteen mixtures.

Table 4.2: Testing program

	Test name	Standard	Testing age(s) in days	Replicates
Fresh	Air content	ASTM C231	Fresh	1
	SAM number	AASHTO TP 118	Fresh	2
	Slump	ASTM C143	Fresh	1
	Fresh density (unit weight)	ASTM C138	Fresh	1
	Temperature	AASHTO T 309	Fresh	1
Hardened	Compressive strength	ASTM C39	3, 7, 28, 56, 90	3 each age
	Modulus of rupture (MOR, or flexural strength)	ASTM C78	28	2
	Modulus of elasticity (MOE) and Poisson's ratio	ASTM C469	28	2
	Resistivity	AASHTO T 358	3, 7, 28, 56, 90	3 each age
	Formation factor (via Bucket Test)	Protocol by J. Weiss, Oregon State University (Weiss 2018)	35	2
	Shrinkage	ASTM C157	Per standard	3
	Rapid chloride permeability	ASTM C1202	28, 90	2

Each mixture was prepared in two batches in accordance with ASTM C685, allowing the research team to mix adequate quantities of concrete for groups of tests. Fresh properties were measured for both batches to verify consistency. Batch 1 was used to cast test specimens for compressive strength test, hardened air content measurement, MOE and Poisson's ratio tests, RCPT, and formation factor tests. Batch 2 was used to cast beams for MOR and shrinkage tests. A set of compressive strength cylinders was also cast from Batch 2 in order to verify consistency between batches.

4.3.1 Fresh Concrete Properties

To mitigate the influence of a wide range of air contents on the test results, air content of all batches was restricted to a range between 5.0% to 6.0%. Batches not meeting this range (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 tests (average of the two batches for each mixture) is presented in Appendix B, Table B.3, along with a graph of unit weights shown in Figure B.4.

4.3.2 Mechanical Properties

Mechanical property tests were performed using the methods listed in Table 4.2. A summary of these results (typically the average of two or three specimens) is provided in Table 4.3. Supporting data providing the result of each test and averages/standard deviations is provided in Appendix B. Test results for each specimen are shown for compressive strength in Table B.4, MOR in Table B.5, MOE in Table B.6, and Poisson's ratio in Table B.7. Figure B.5 in Appendix B provides a graph of 28-day compressive strength test results with variability shown.

Table 4.3: Results of laboratory testing for mechanical properties

Mixture ID	Compressive strength (psi)					MOE (psi)	Poisson's ratio	MOR* (psi)
	3 day	7 day	28 day	56 day	90 day			
H-700-0	3,810	4,394	5,379	6,140	6,381	3,040,000	0.21	-
H-560-140	3,461	3,950	4,994	5,961	6,087	2,670,000	0.20	-
H-650-0	4,276	5,232	6,256	7,135	7,556	3,650,000	0.21	-
H-520-130	3,705	4,323	5,319	6,921	7,233	3,060,000	0.23	-
H-600-0	3,750	4,309	5,494	5,887	6,302	2,980,000	0.19	745
H-480-120	2,784	3,150	3,982	4,418	5,148	2,530,000	0.20	808
H-420-180	2,446	3,417	4,328	4,869	5,521	2,460,000	0.22	724
M-700-0	5,088	5,679	6,688	7,531	8,168	3,570,000	0.24	-
M-560-140	4,019	4,854	5,688	6,114	6,322	3,360,000	0.18	-
M-650-0	5,192	5,935	6,739	7,223	8,221	3,710,000	0.20	-
M-520-130	4,258	5,129	6,375	7,705	8,416	3,620,000	0.20	-
M-600-0	4,526	5,362	5,873	6,418	7,995	3,400,000	0.21	822
M-480-120	4,167	4,895	5,390	5,832	6,483	3,350,000	0.19	726
M-420-180	3,991	4,260	5,007	5,590	6,216	3,080,000	0.20	727
M-600P-0	4,661	5,212	6,284	6,841	7,098	3,450,000	0.23	809
M-480P-120	4,249	5,314	6,415	6,967	7,215	3,130,000	0.19	720
M-420P-180	3,852	4,288	5,091	5,418	6,004	3,000,000	0.20	681
L-700-0	5,921	7,550	7,856	8,762	9,237	3,830,000	0.17	-
L-560-140	5,045	5,267	6,729	7,316	7,808	3,660,000	0.20	-
L-650-0	6,984	7,367	7,991	8,251	9,113	4,320,000	0.19	-
L-520-130	5,194	6,005	7,203	7,591	8,062	3,630,000	0.21	-
L-600-0	5,698	6,471	7,010	7,427	7,936	3,760,000	0.19	817
L-480-120	5,510	6,184	6,814	7,107	7,650	3,090,000	0.22	718
L-420-180	5,264	5,716	6,228	6,693	7,063	3,240,000	0.20	815

* MOR was tested for pavement-type (lower cementitious content) mixtures only

Compressive Strength

To be in compliance with NCDOT's 2018 Standard Specifications, both paving and Type AA (bridge) mixtures must have a minimum compressive strength of 4,500 at 28 days (NCDOT 2018). Of the 24 mixture designs, all but two (H-480-120 and H-420-180) met NCDOT's required minimum compressive strength of 4,500 psi by 28 days (for both Type AA and pavement mixtures). Hydration of fly ash occurs at later ages than cement, allowing the H-420-180 mixture to meet the minimum requirement at 56 days, and mixture H-480-120 to meet the requirement by 90 days. Compressive strength test results are graphically displayed in Figures 4.2 and 4.3.

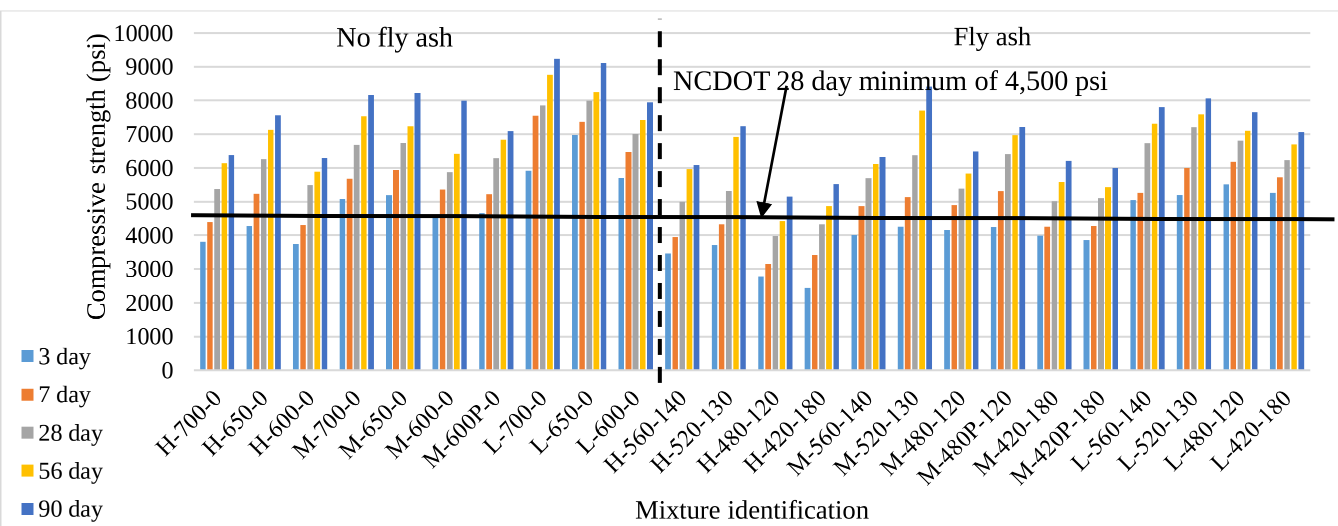

Figure 4.2: Compressive strengths with mixtures sorted by non-fly ash and fly ash mixtures

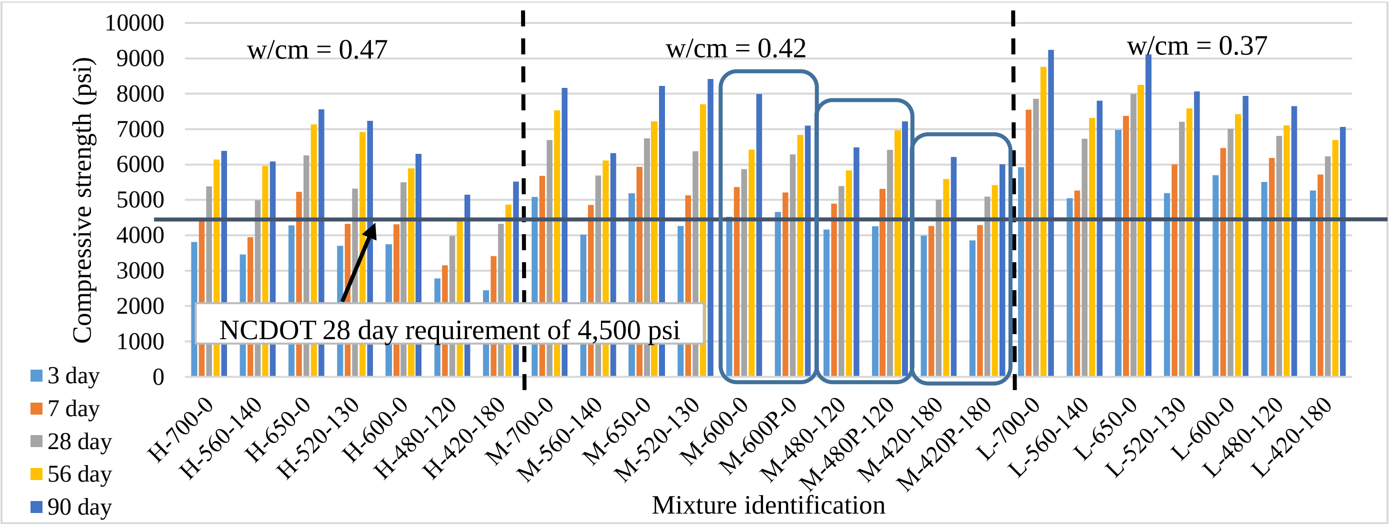

Figure 4.3: Compressive strength test results with mixtures sorted by *w/cm*. Blue rectangles denote pairs of OPC and PLC mixtures

As expected, mixtures with the 0.37 *w/cm* had the highest compressive strength performance when compared to the 0.42 and 0.47 *w/cm*. The five mixtures with the highest 28-day compressive strengths were low *w/cm* (0.37) mixtures. The use of increasing water contents in higher *w/cm* mixtures resulted in a reduction in compressive strength, also as expected. This is likely a reason for H-480-120 and H-420-180 not meeting the 28-day NCDOT requirement, although the majority of the high (0.47) *w/cm* mixtures still met the 28-day NCDOT minimum compressive strength of 4,500 psi. Not only did 22 of the 24 mixtures produced meet this 28-day minimum, but they far exceeded the required results, providing insight into the excessive cementitious contents often utilized. Excessive cement cementitious contents can result in both economic issues and poor durability performance. High *w/cm* mixtures (0.47) accounted for six of the nine lowest average 28-day compressive strengths, with only one mixture (H-650-0) above the bottom 50% of all mixtures.

Mixtures with the higher cementitious material contents (700 pcy and 650 pcy) outperformed the 600 pcy mixtures. As shown in Figure 4.2, the 700 pcy and 650 pcy mixtures were relatively comparable at each testing date. The 650 pcy straight cement mixtures had the highest compressive strengths at most test dates for the 0.47 and 0.37 *w/cm*, with the 0.42 *w/cm* 650 pcy straight cement mixture performing in the top few mixtures with the 0.42 *w/cm*.

Straight cement mixtures typically had the highest compressive strengths. Of the ten groups of mixtures (as grouped by cementitious material content and *w/cm*), in all but one instance (M-600-0 vs. M-480P-120), straight cement mixtures outperformed their companions fly ash mixtures. These nine non-fly ash mixtures had superior compressive strengths across all five test ages. Since fly ash hydrates more slowly than portland cement, compressive strength testing at even later ages could have resulted in fly ash mixtures performing even more comparable to the straight cement mixtures. Of note, the 0.47 *w/cm* mixtures with a fly ash replacement performed similar to the non-fly ash mixtures, and M-520-130 had the highest compressive strength of all mixtures with the 0.42 *w/cm*.

Of the 24 mixtures, three of the paving mixtures were batched using PLC. Each of these three mixtures had a companion OPC mixture with the same mixture proportions, with blue rectangles shown in Figure 4.3 denoting pairs of OPC and PLC mixtures. Of interest to stakeholders is the relative performance of the PLC compared to the OPC, if used in the same mixtures/proportions. The M-480P-120 mixture significantly outperformed its companion mixture at all five test dates, with compressive strengths 12.1% higher on average. The two other PLC mixtures, M-600P-0 and M-420P-180 outperformed their companion mixtures on three and two of the test dates respectively. At 28 days, which is the primary focus of NCDOT testing, all three PLC mixtures had higher compressive strengths than their OPC companion mixtures. This could be attributable to fineness differences between the OPC and PLC (PLC is often ground finer to aid in hydration reactions), or due to particle packing effects.

Modulus of Rupture

Modulus of Rupture testing was performed at 28 days for lower (600 pcy) cementitious content mixtures only, with three beams being tested for each and the results averaged. Mixtures for which MOR testing was performed are colored green in Figure 4.1 and Table 4.1. Graphical depictions of MOR results are shown in Figures 4.4 and 4.5, color coded by *w/cm* as shown in Figure 4.1.

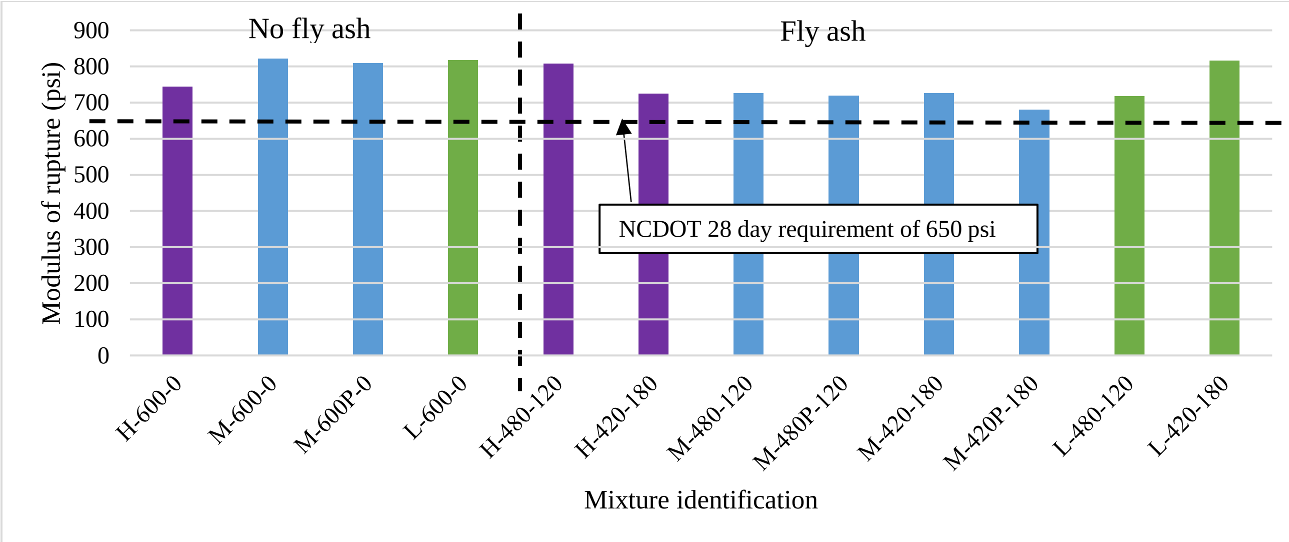

Figure 4.4: MOR test results with mixtures sorted by non-fly ash and fly ash mixtures

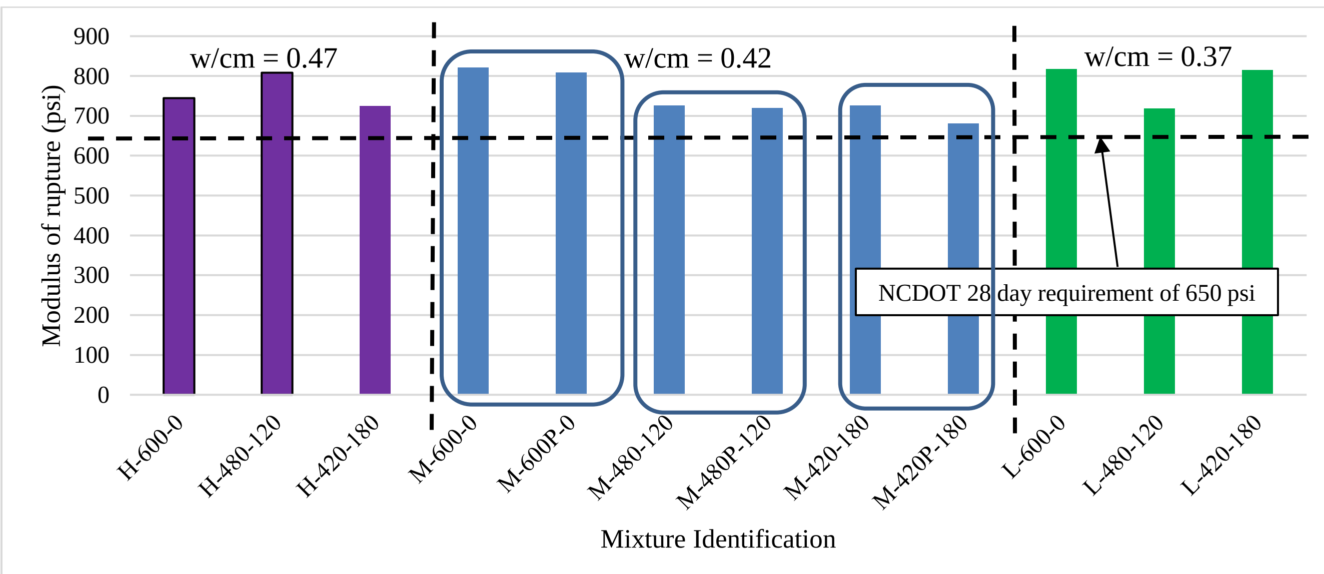

Figure 4.5: MOR test results sorted by *w/cm*

NCDOT's specifications require a minimum flexural strength of 650 psi at 28 days for paving applications (NCDOT 2018). All twelve of the paving mixtures reached this minimum requirement. Similar to results for compressive strength tests, the mixtures without fly ash those with lower *w/cm* typically exhibited the highest MOR. The four mixtures without fly ash accounted for four of the six highest test results, including the two highest test results (M-600-0 and L-600-0). Of the twelve mixtures tested, the four highest test results (M-600-0, L-600-0, L-420-180, and M-600P-0) were medium and low (0.42 and 0.37, respectively) *w/cm*. Pairs of OPC and companion PLC mixtures are shown with a blue rectangle in Figure 4.5. As can be observed, PLC mixtures exhibited similar MOR to their OPC counterpart mixtures.

Modulus of Elasticity and Poisson's Ratio

As evident in Figure 4.6 (which is color coded by total cementitious materials content per Figure 4.1 and separated by fly ash and non-fly ash mixtures), non-fly ash mixtures showed higher MOE test results than their fly ash companion mixtures. The MOE values for lower *w/cm* mixtures were lower than the higher *w/cm* mixtures. Mixtures with higher cementitious material contents also typically had higher MOE than those with lower contents. These trends can be seen in Figure 4.7, which is grouped by *w/cm* and color coded by total cementitious material content. Rectangles are also used in Figure 4.6 and 4.7 to group OPC and PLC companion mixtures. Pairs of OPC/PLC mixtures tended to exhibit similar MOE, and no trend was evident.

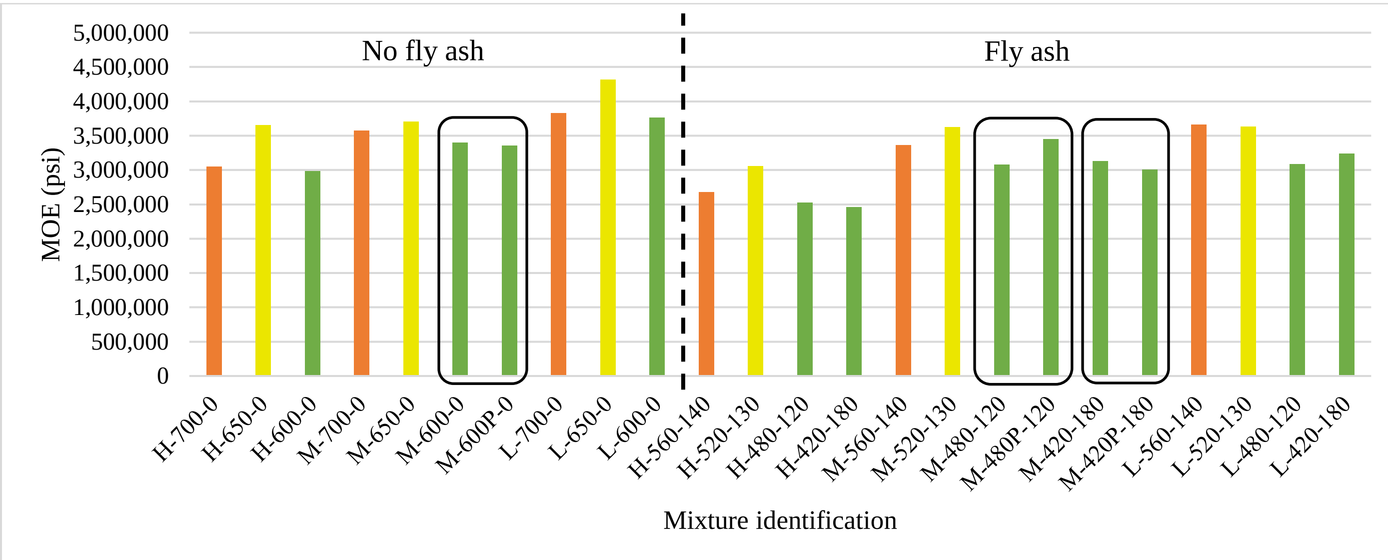

Figure 4.6: MOE test results sorted by non-fly ash and fly ash mixtures

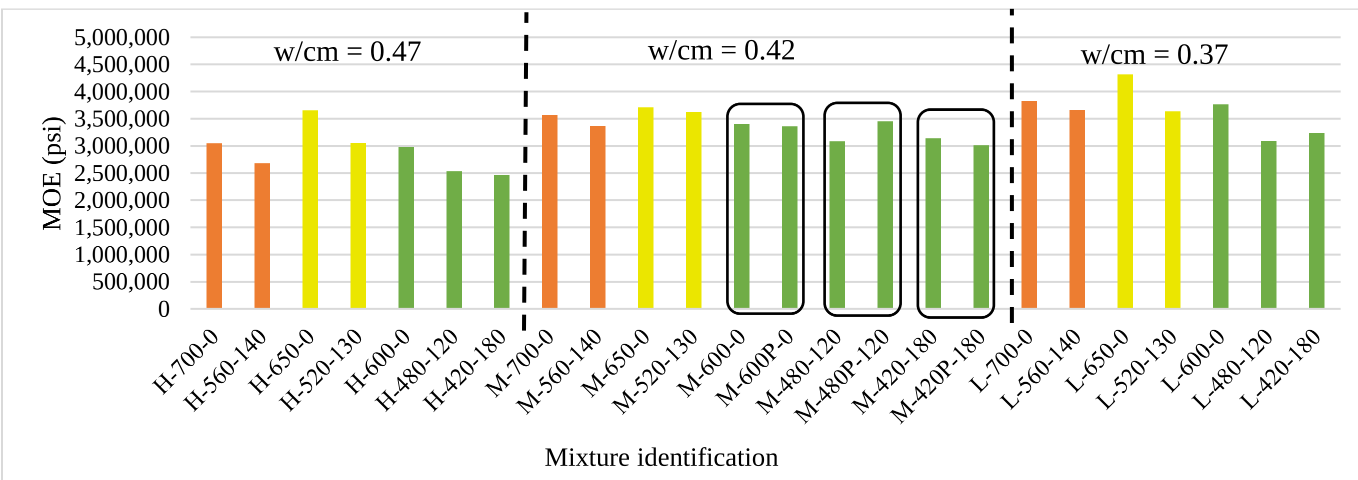

Figure 4.7: MOE test results sorted by *w/cm*

A graphical representation of the calculated and measured MOE values is presented in Figure 4.8. The measured MOE test results, which ranged from 2,460,000 psi to 4,320,000 psi, showed a very consistent trend (similar slope) with values predicted based upon mixture unit weight per the equation provided in ACI 318 (2014), which ranged from 3,430,000 psi to 5,050,000 psi. Although the measured MOE values exhibited a similar slope to those predicted by the ACI 318 equation commonly used by structural designers, all measured values were notably lower (roughly 13-33%) than the predicted counterparts. This trend of lower-than-predicted MOE values measured from concrete with NC materials was a trend evident in previous studies by this research team (Cavalline et al. 2018 and 2019), and should be of interest to NCDOT because of the potential for lower cracking tendency as well as the potential implications for designers.

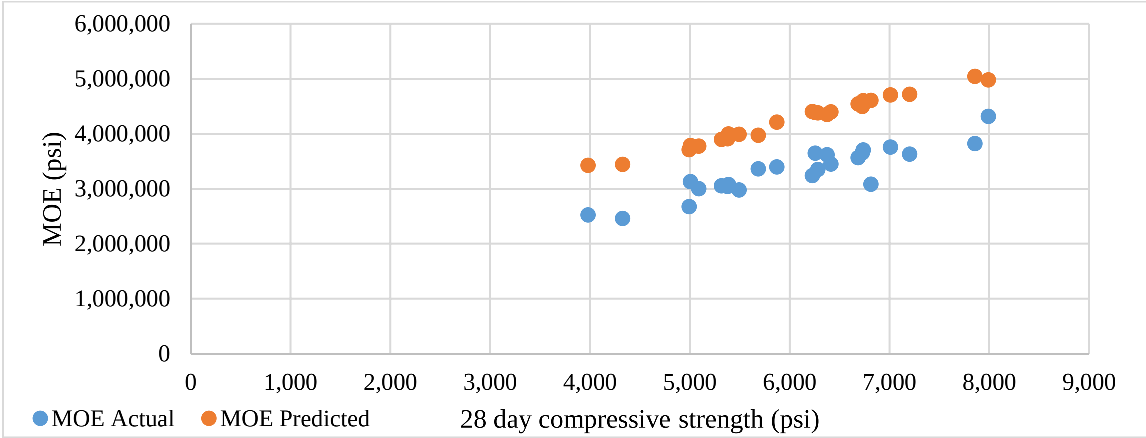

Figure 4.8: Predicted and measured MOE results

Poisson's Ratio

The trends observed when reviewing results of testing for Poisson's ratio were similar to those found in MOE testing, with a range of 0.17 to 0.24. No trends stood out when the data for Poisson's ratio was plotted. Figures 4.9 and 4.10 show this data color coded by total cementitious material content and separated by fly ash vs. non-fly ash mixtures and *w/cm*. Pairs of OPC/PLC mixtures are again denoted with a rectangle.

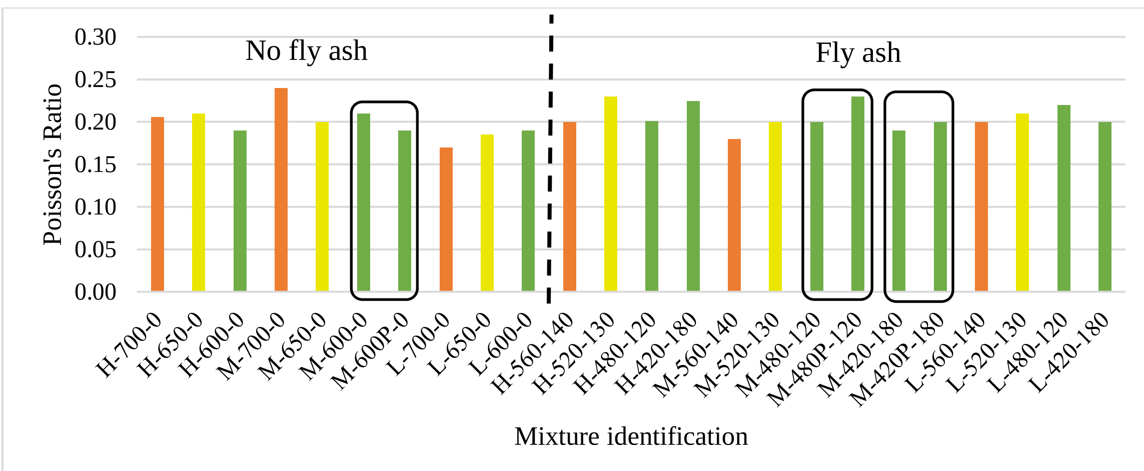

Figure 4.9: Poisson's Ratio test results sorted by fly ash and non-fly ash mixtures

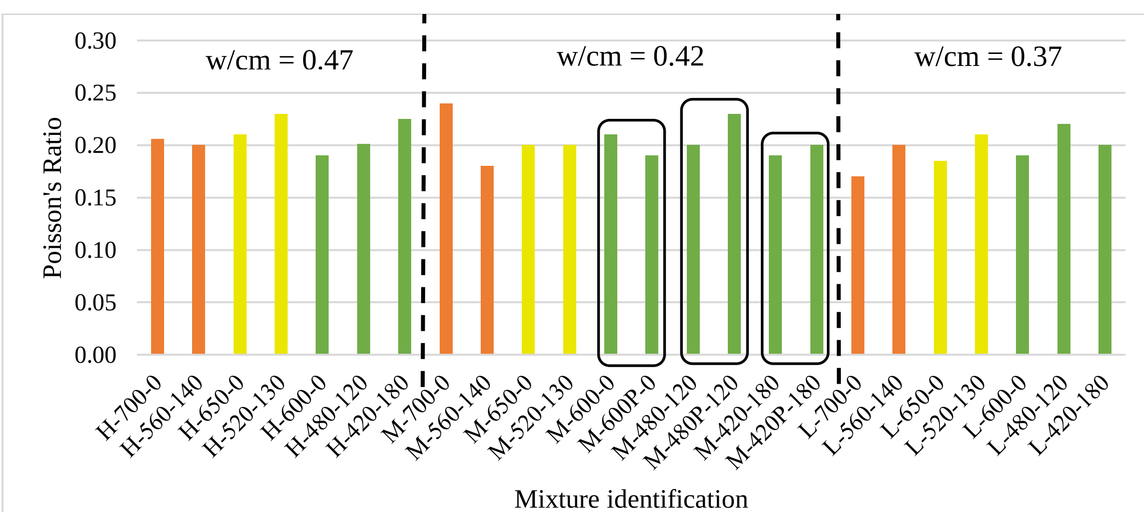

Figure 4.10: Poisson's Ratio sorted by *w/cm*

4.3.3 Durability Performance

Durability performance tests were performed using the methods listed in Table 4.4. A summary of these results (typically the average of two or three specimens) is provided in Table 4.3. Supporting data providing the result of each test and averages/standard deviations is provided in Appendix B. Test results for each specimen are shown for surface resistivity test results in Table B.8, bulk resistivity in Table B.9, and RCPT in Table B.10.

Table 4.4: Summary of durability performance test results

Mixture ID	Surface resistivity (kΩ-cm)					Bulk resistivity (kΩ-cm)		RCPT (coulombs)	
	3 day	7 day	28 day	56 day	90 day	28 day	90 day	28 day	90 day
H-700-0	6.1	6.4	7.3	12.1	14.0	5.1	14.1	4,253	3,070
H-560-140	5.1	5.7	6.6	14.1	18.8	4.9	15.2	3,860	2,118
H-650-0	5.7	6.8	8.7	9.7	9.8	5.0	8.9	4,687	4,018
H-520-130	4.8	6.3	10.6	18.0	21.8	6.8	17.1	4,480	2,879
H-600-0	6.9	7.3	8.1	11.2	17.6	5.2	11.9	4,159	3,439
H-480-120	5.4	5.8	9.5	12.0	17.1	7.3	11.6	3,766	2,266
H-420-180	4.2	6.9	11.2	16.3	20.7	9.7	19.2	3,571	1,980
M-700-0	7.1	8.1	10.9	10.9	12.5	7.2	9.2	4,479	3,822
M-560-140	5.5	6.0	6.4	15.8	18.4	4.8	16.1	4,354	2,148
M-650-0	7.1	8.0	10.7	11.2	11.9	7.0	8.8	3,506	3,008
M-520-130	6.1	6.9	12.1	22.4	26.9	8.4	26.0	4,247	2,154
M-600-0	6.4	7.9	10.0	16.5	22.7	7.1	17.3	3,943	3,087
M-480-120	4.5	6.3	9.4	14.1	20.3	6.4	11.6	3,632	2,132
M-420-180	4.7	5.5	6.1	13.8	19.6	5.4	13.9	3,391	1,768
M-600P-0	7.2	9.0	10.6	17.2	20.0	7.2	13.1	3,897	3,143
M-480P-120	5.5	6.1	6.6	14.8	19.7	5.2	12.3	3,746	2,575
M-420P-180	4.7	5.4	6.3	15.3	21.8	5.8	14.2	3,514	2,352
L-700-0	5.5	6.5	9.3	10.1	15.7	7.8	11.7	4,766	2,947
L-560-140	4.5	5.0	12.3	16.1	20.2	10.1	13.5	4,094	2,136
L-650-0	6.3	6.9	14.8	17.2	18.6	13.5	15.2	4,239	2,197
L-520-130	4.5	5.1	13.1	18.4	23.3	11.7	18.3	2,532	1,409
L-600-0	5.7	6.3	9.9	13.7	17.0	8.2	12.0	3,572	1,962
L-480-120	4.9	5.3	9.1	13.9	19.8	7.4	13.8	2,987	1,840
L-420-180	5.1	5.4	8.4	12.0	18.7	5.4	11.1	2,879	1,557

Surface Resistivity

Results of surface resistivity tests are presented in Table 4.4. These results can be qualitatively described with the permeability rating given in Table 4.5, which reflects guidance included AASHTO T 358 (AASHTO 2017). Figures 4.11 and 4.12 provide a graphical representation of results, with Figure 4.11 sorted by fly ash/non-fly ash content, and Figure 4.12 sorted by *w/cm*. In general, the surface resistivity increased from early ages to later ages, which indicates that additional hydration time reduces permeability, as expected. 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 4.5. All of the straight cement concrete mixtures 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 will be described in the next section.

Table 4.5: AASHTO T 358 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

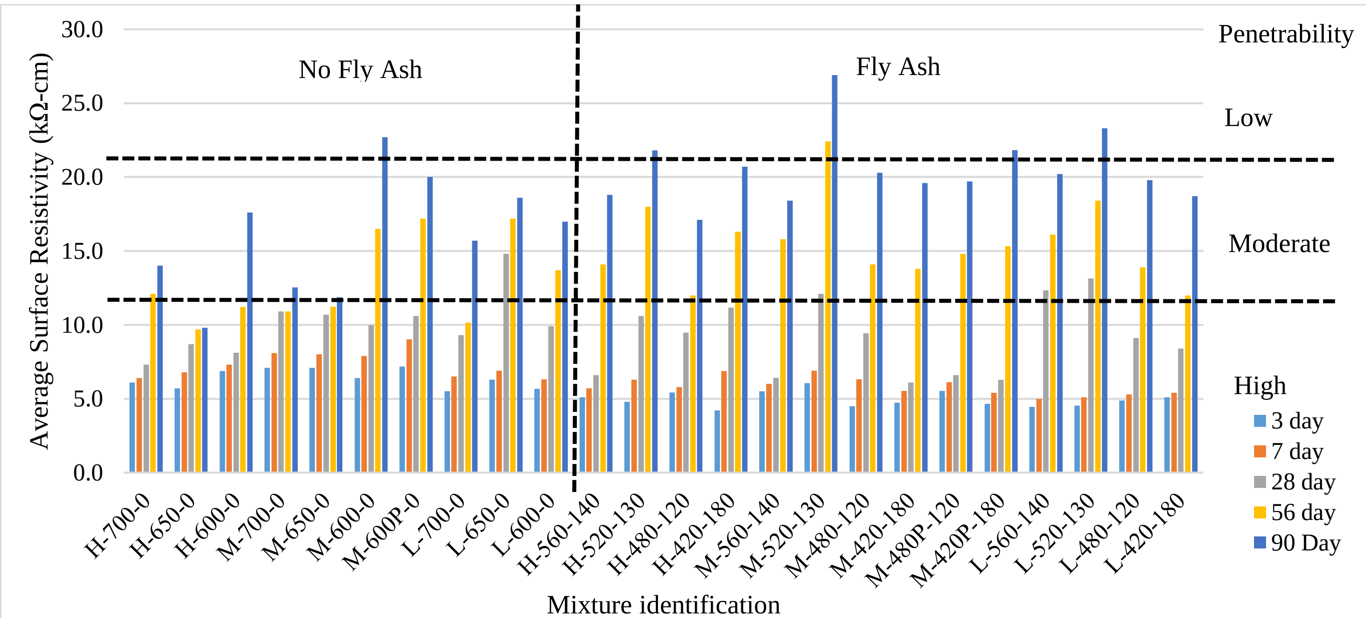

Figure 4.11: Surface resistivity test results with mixtures sorted by non-fly ash and fly ash mixtures

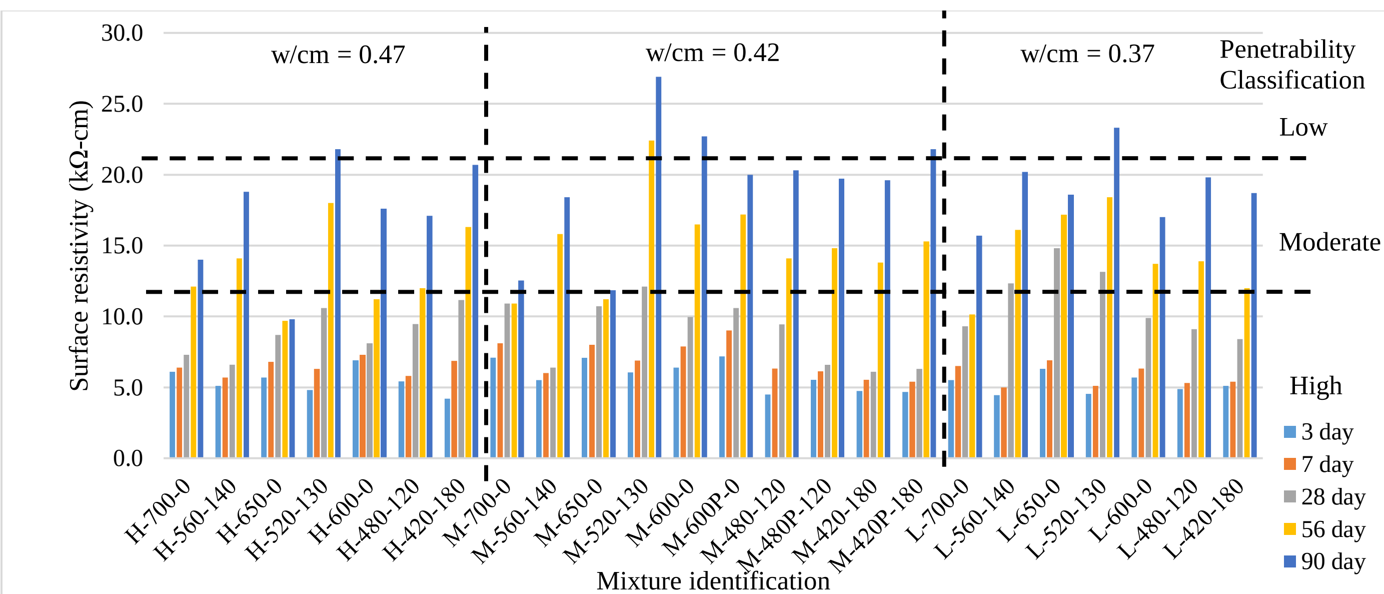

Figure 4.12: Surface resistivity test results with mixtures sorted by *w/cm*

The influence of the *w/cm* ratio at 28 days can be seen in the superior surface resistivity performance of the 0.37 *w/cm* mixtures. These mixtures outperformed their 0.42 and 0.47 companion mixtures in most instances. Twenty-eight day test results are represented graphically in Figure 4.13, which is color coded by total cementitious content in a manner similar to Figure 4.1. At 56 days the influence of the *w/cm* is less apparent, although the 0.37 *w/cm* averages are slightly higher than the averages of the 0.47 *w/cm*. At 90 days, the influence of the *w/cm* on surface resistivity can clearly be seen between the 0.37 *w/cm* and 0.47 *w/cm*. When comparing Figures 4.13 through 4.15 (*w/cm* vs. surface resistivity graphs) in sequence, the separation of the values can be clearly observed.

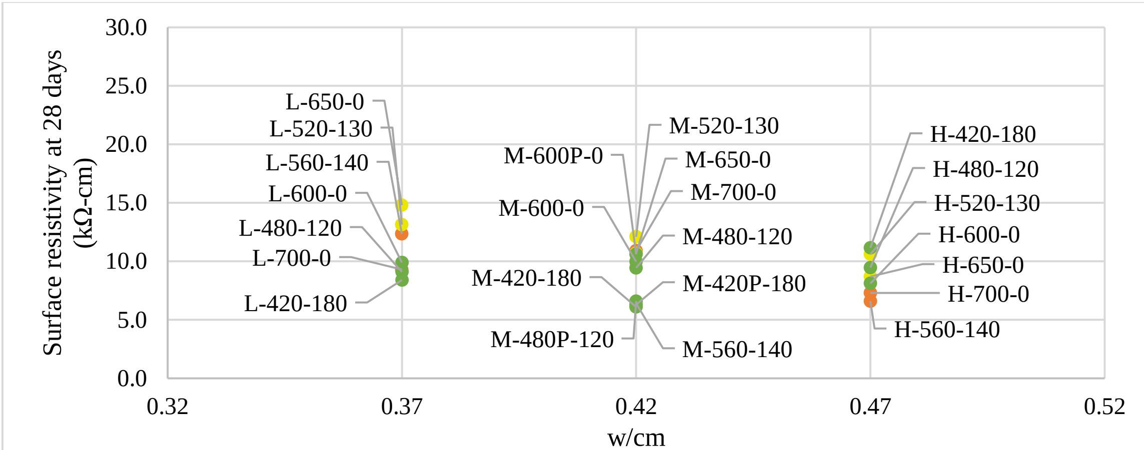

Figure 4.13: *w/cm* vs. surface resistivity averages at 28 days

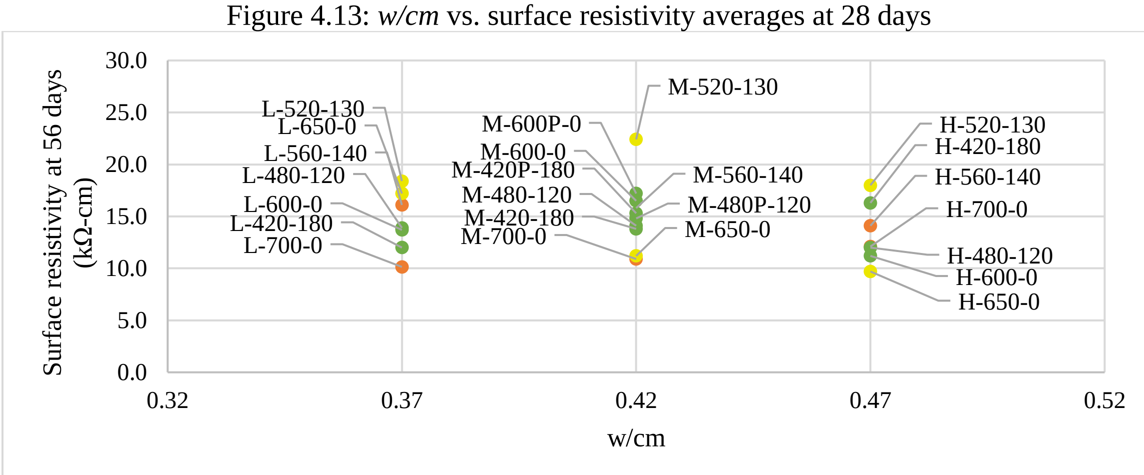

Figure 4.14: *w/cm* vs. surface resistivity averages at 56 days

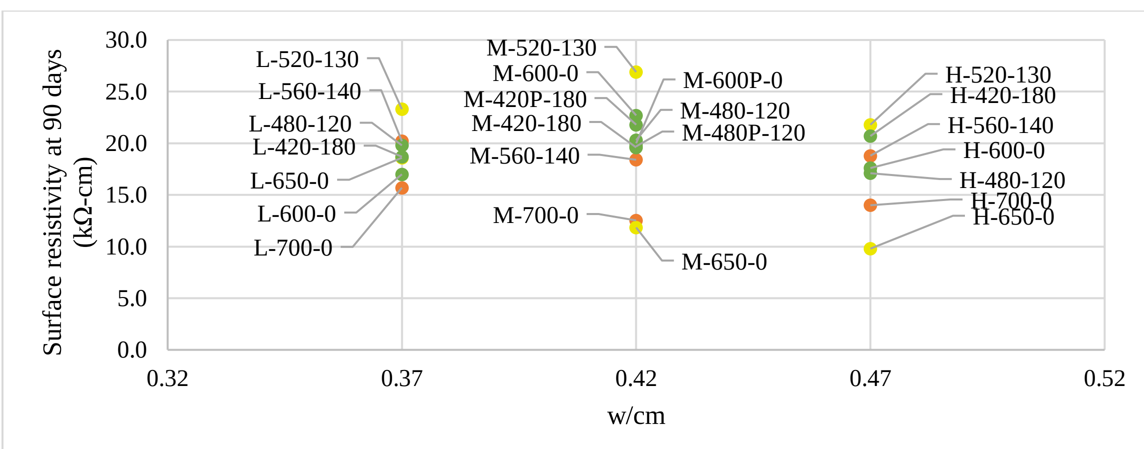

Figure 4.15: *w/cm* vs. surface resistivity averages at 90 days

The influence of total cementitous material content on surface resistivity at 28 days is most prevelant in the 650 pcy mixtures. As depicted in Figure 4.17, the mixtures containing a fly ash replacement outperform those without in most cases. Fifty-six day surface resistivity testing shows distinct seperation between fly ash and non-fly ash mixtures for the 650 pcy and 700 pcy mixtures. For all twelve of these mixtures, those with a fly ash replacement outperform their straight cement counterpart mixtures. Mixtures for the 600 pcy cementitious content also showed the improved performance of fly ash mixtures when compared to non-fly ash counterparts. These trends can be seen in Figure 4.18. At 90 days, the trends seen at 56 days become even more prevelant. The seperation between fly ash and non-fly ash mixtures for 650 pcy and 700 pcy mixtures is greater, as can be seen in Figure 4.19. The 600 pcy mixtures also show an improved performance for fly ash vs. non fly ash mixtures when compared to values at 56 days.

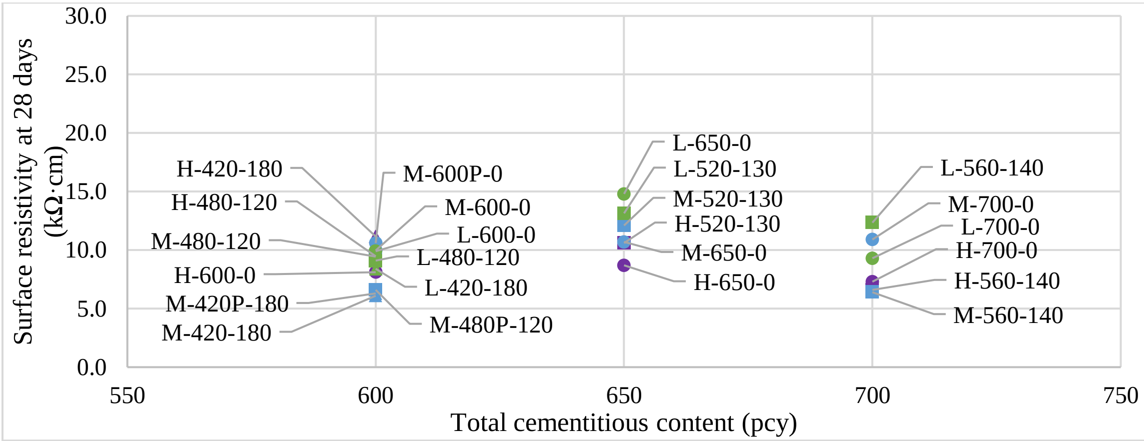

Figure 4.16: Total cementitious content (pcy) vs. surface resistivity averages at 28 days

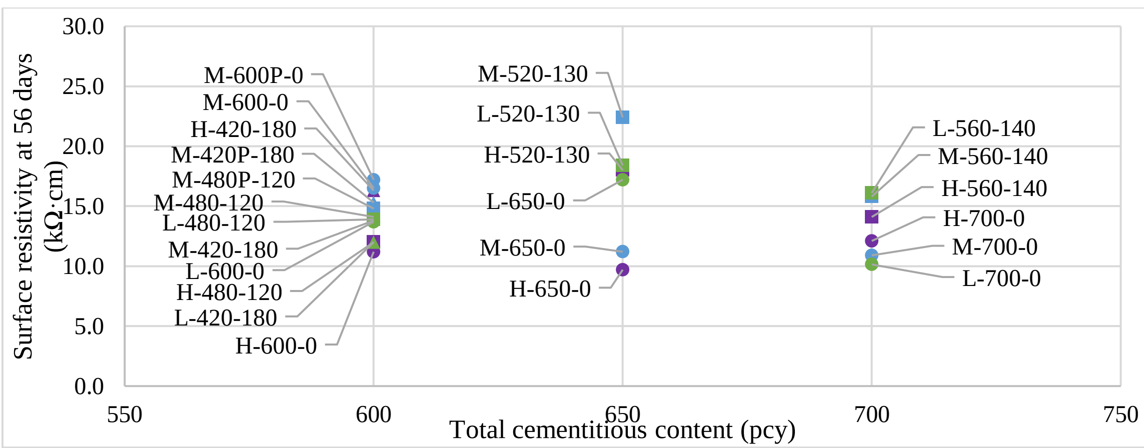

Figure 4.17: Total cementitious content (pcy) vs. surface resistivity averages at 56 days

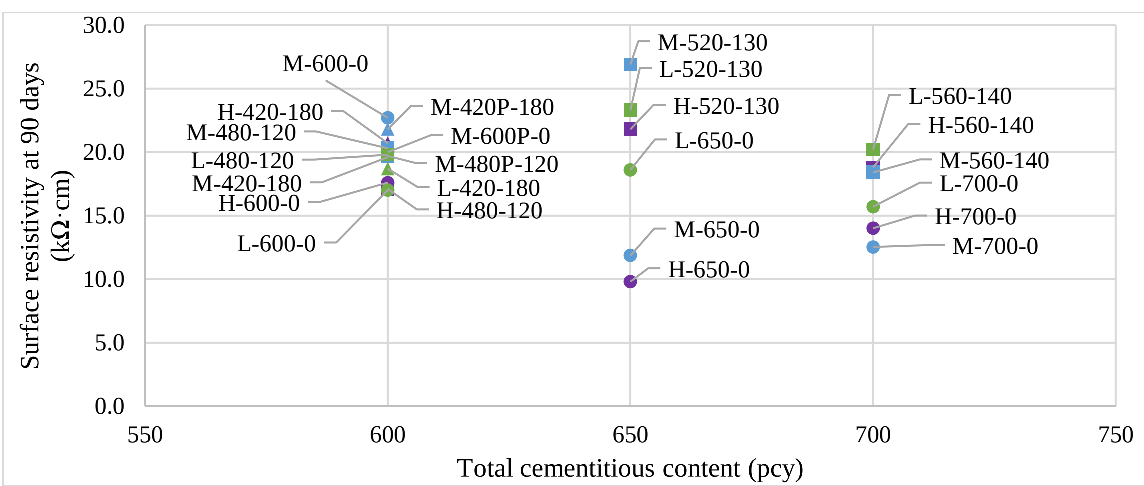

Figure 4.18: Total cementitious content (pcy) vs. surface resistivity averages at 90 days

Perhaps one of the most interesting findings was the evaluation of surface resistivities at 28, 56, and 90 days of OPC and PLC mixtures. The differences between surface resistivity readings at each test date for PLC mixtures and their OPC companions is relatively small (Figure 4.19). For mixtures containing a fly ash replacement, the trends are even more interesting. The 20 percent ash replacement showed a larger gap between readings at early ages with better results for the OPC mixture. At later ages however, the PLC companion mixture closed the gap by 56 days, with hardly any difference between the two at 90 days. For the 30% ash replacement OPC and PLC mixture, results at 28 days were nearly identical. By 56 days, the PLC mixture was slightly outperforming the OPC companion, and increased its outperformance by 90 days.

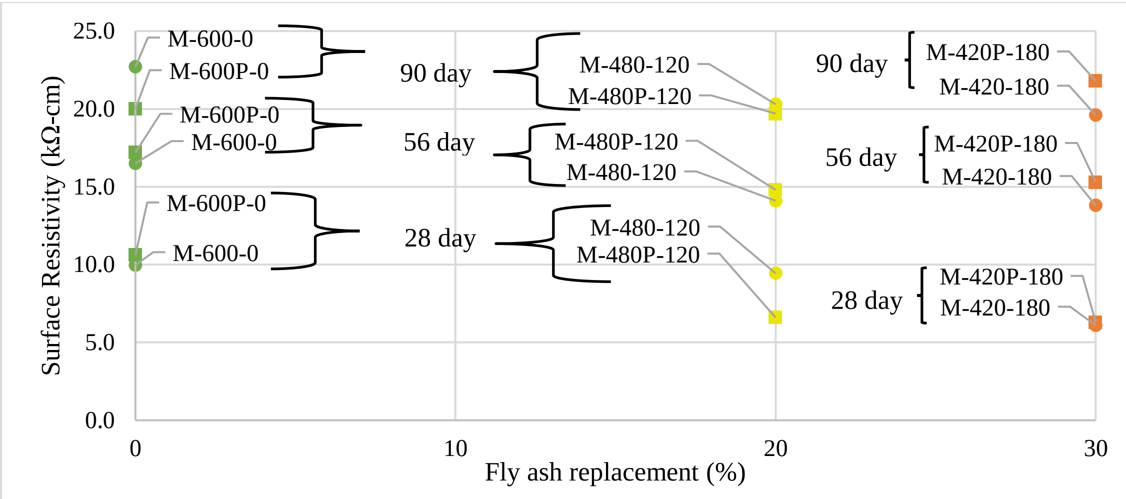

Figure 4.19: Fly ash replacement % vs. surface resistivity for OPC and PLC companion mixtures at 28, 56, and 90 days

Bulk Resistivity

For non-fly ash mixtures, those with lower *w/cm* performed better than the 0.47 *w/cm* companion mixtures. This trend can be seen in Figure 4.20. Mixtures with a fly ash replacement also typically performed better than their companion non-fly ash mixtures, particularly when comparing 90-day results.

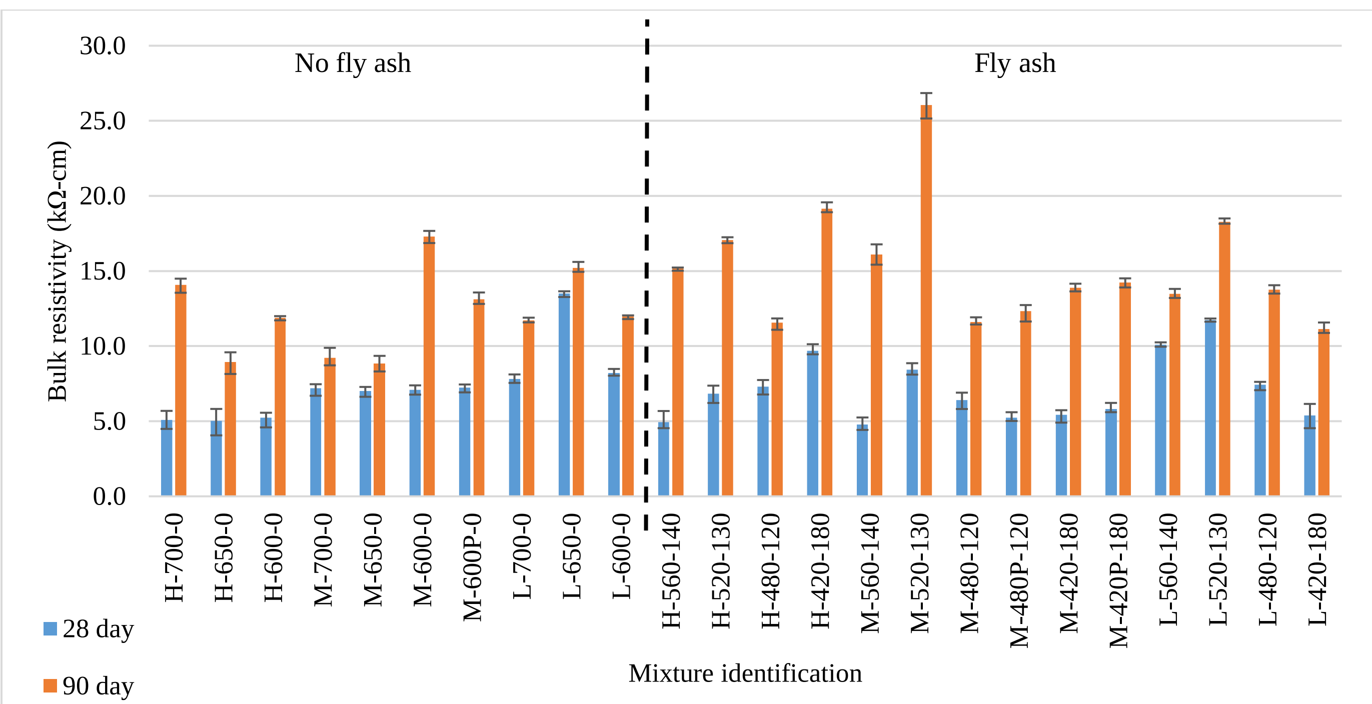

Figure 4.20: Bulk resistivity with mixtures sorted by non-fly ash and fly ash mixtures

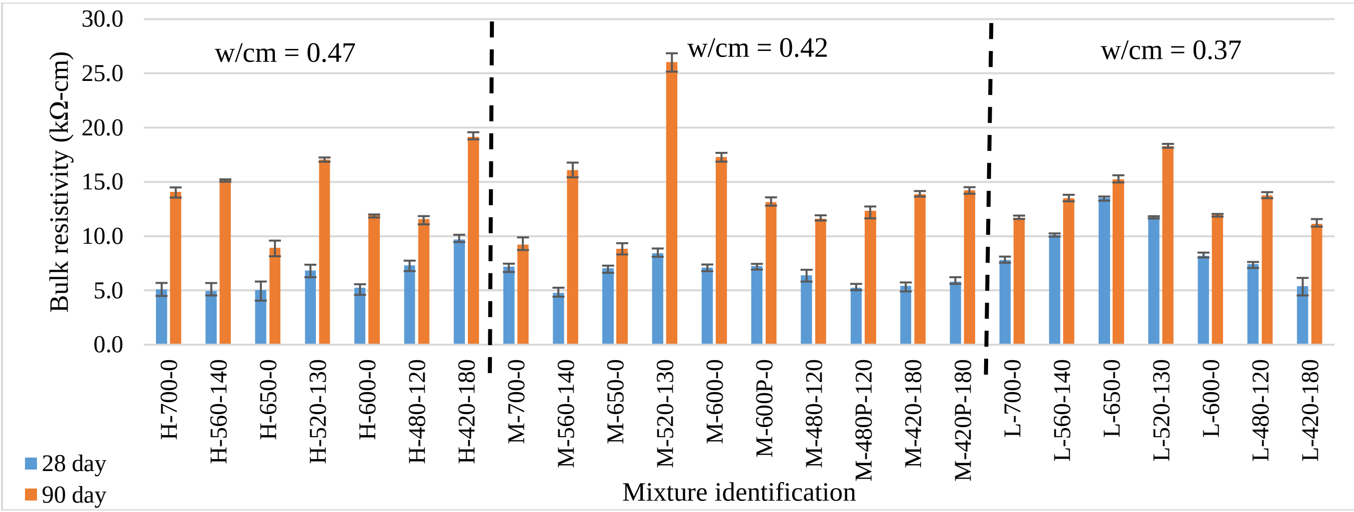

Figure 4.21: Bulk resistivity results with mixtures sorted by *w/cm*

Bulk resistivity results were also compared to surface resistivity results for each mixture, as bulk resistivity and surface resistivity should be directly correlated. There was a strong linear correlation between the two data sets, which is shown in Figure 4.22, with 28 day results in blue and 90 day results in orange. As these two properties are directly correlated, the relationship is expected to be linear, as shown in Figure 4.22 with an R^2 value of 0.86. However, a power model provided a slightly better fit to this data with an R^2 value of 0.89.

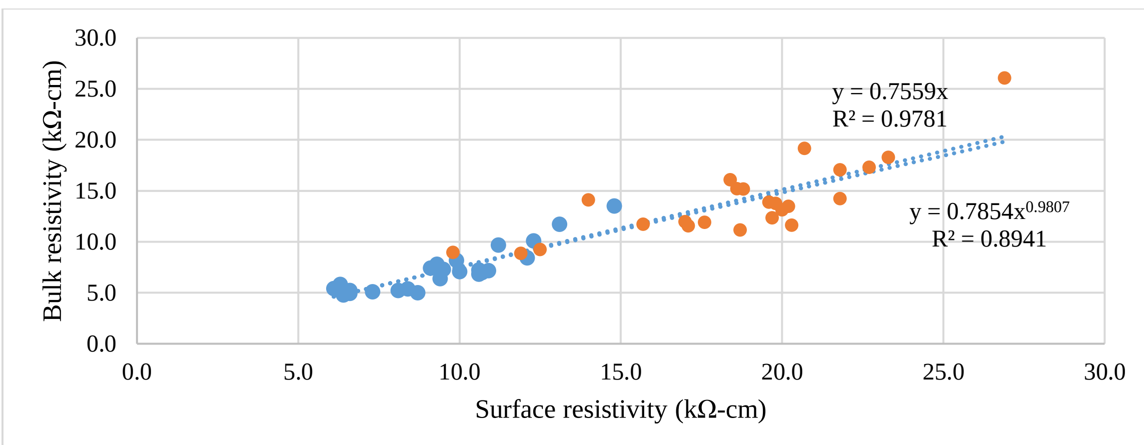

Figure 4.22: Bulk resistivity vs. surface resistivity with linear model and power model

RCPT

Results of RCPT tests may be interpreted with the qualitative descriptors given in Table 4.6, which is provided in ASTM C1202. Figure 4.23 shows test results with mixtures grouped by the presence or absence of fly ash, and Figure 4.24 shows test results with mixtures grouped by *w/cm*. 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. At 28 days, mixtures containing fly ash typically performed better than those without, with all but two (8 of 10) non-fly ash mixes accounting for the lower 50% of RCPT test results. The two mixtures without fly ash at 28 days that fell in the better 50% of performance were M-650-0 and L-600-0, which were in the middle and lower *w/cm* and cementitious material contents. These observations can be made in Figure 4.25. A graphical comparison of 28 and 90 day RCPT sorted by total cementitious content and *w/cm* is shown in Figure 4.26. At 90 days, similar trends were shown with 8 of the 10 non-fly ash mixes falling in the lower range of performance. The two non-fly ash mixtures that were in the better half at 90 days were L-600-0 and L-650-0, again showing better performances by the low *w/cm* and lower cementitious contents. At 90 days, mixes containing fly ash showed a much more significant reduction in chloride permeability. This can likely be attributed to the longer hydration requirements of the fly ash in comparison to the cement.

Table 4.6: 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

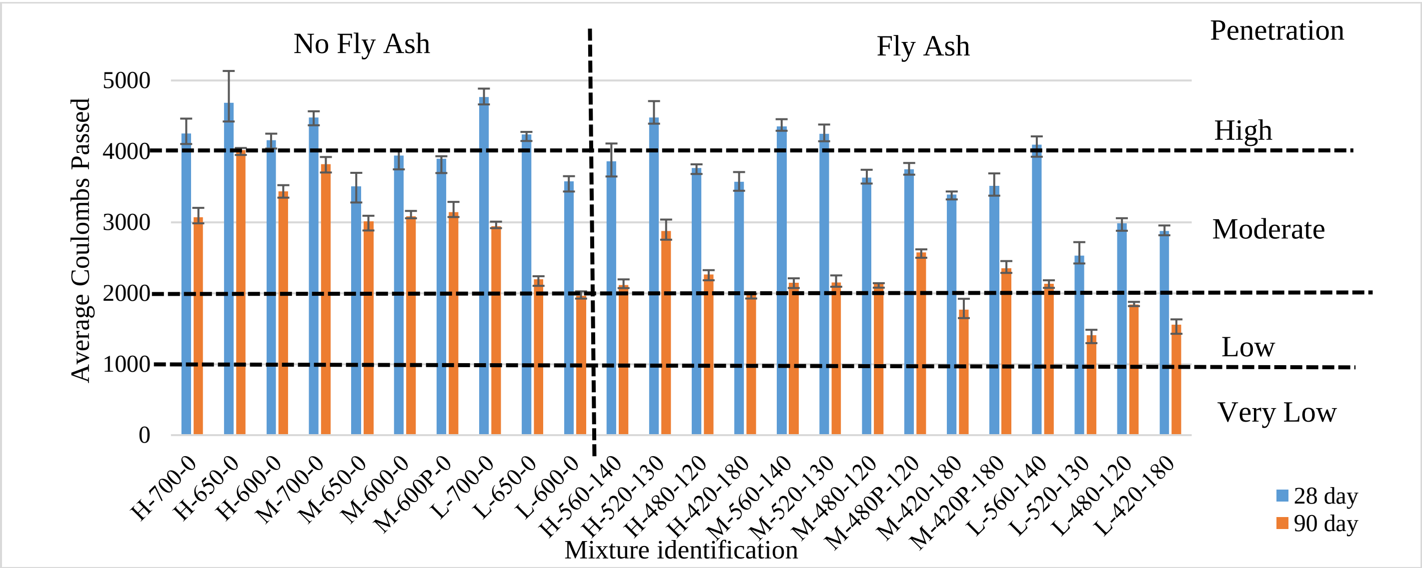

Figure 4.23: RCPT results with mixtures sorted by non-fly ash and fly ash mixtures

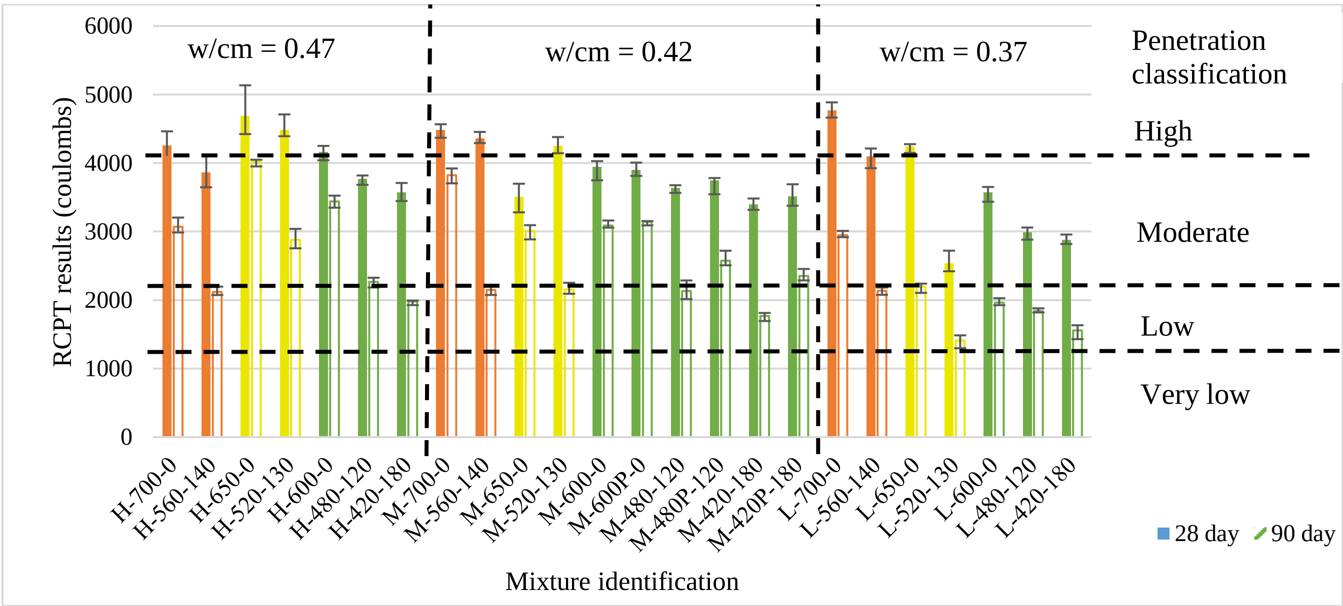

Figure 4.24: RCPT results with mixtures sorted by *w/cm*

Formation Factor

Testing to support evaluation of use of the formation factor (per AASHTO PP 84) instead of using an unmodified surface resistivity value, was performed. Ongoing research using the Bucket Test method helps to provide insight into the role of pore solution chemistry and pore structure on bulk resistivity and surface resistivity measurements. **At the time of this work, the Bucket Test procedure had recently been released by Weiss et al., and this work should be considered preliminary at this time. A number of ongoing studies are being performed to refine the test method and to better interpret the results.**

Two samples per mixture were tested using the procedure developed by Dr. W. Jason Weiss at intervals ranging from 2 hours to 91 days. Surface resistivity and bulk resistivity tests were performed on the cylinders after being removed from the buckets filled with a solution designed to mimic concrete pore solution. The average test result from the two specimens was calculated and used to compute the formation factor. Selected sample calculations and conversions for Bucket Test and formation factor values can be seen in Table 4.7.

Table 4.7: Sample formation factor calculations

Mixture ID	28-day surface resistivity (kΩ-cm)	28-day Bucket Test (kΩ-cm)	28-day formation factor	56-day surface resistivity (kΩ-cm)	56-day bucket test (kΩ-cm)	56-day formation factor
H-700-0	7.3	9.3	930	12.1	15.5	1550
H-420-180	11.2	12.5	1250	16.3	19.1	1910
M-700-0	10.9	12.2	1220	10.9	12.4	1240
M-420-180	6.1	7.8	780	13.8	14.5	1450
L-700-0	9.3	10.4	1040	10.1	10.5	1050
L-420-180	8.4	10.1	1010	12.0	13.2	1320

Table 4.8 shows the formation factors associated with various levels of chloride ion penetrability, as found in AASHTO PP 84. Due to the influence of conditioning on resistivity values as discussed previously, a pore solution resistivity of 0.10 Ωm was assumed. This value is described in AASHTO PP 84 (AASHTO 2017), and is used to adjust measured resistivity values using a standardized value for typical pore solution resistivity. The measured formation factor averages are provided in Table 4.4. Similar to surface resistivity and bulk resistivity testing results, all mixtures showed improved performance at 90 days (as compared to performance at 28 days). In a manner similar to the other electrical resistivity tests, mixtures with a fly ash replacement typically outperformed their companion mixtures. Mixtures with lower *w/cm* also had a tendency to perform better than the 0.47 *w/cm* mixtures. OPC and PLC companion mixtures (shown grouped in rectangles in Figure 4.28) had nearly identical formation factor values. These trends can be seen in Figures 4.25 and 4.26. Figures 4.27 and 4.28 also show the correlation between formation factor and surface resistivity testing at 28 and 56 days, which had R^2 values of 0.85 and 0.77 respectively. OPC and PLC companion mixtures are color coded purple.

Table 4.8: Chloride ion penetrability associated with various formation factor values

Chloride ion classification	Formation factor value
High	520
Moderate	520 – 1,040
Low	1,040 – 2,080
Very low	2,080 – 20,700
Negligible	20,700

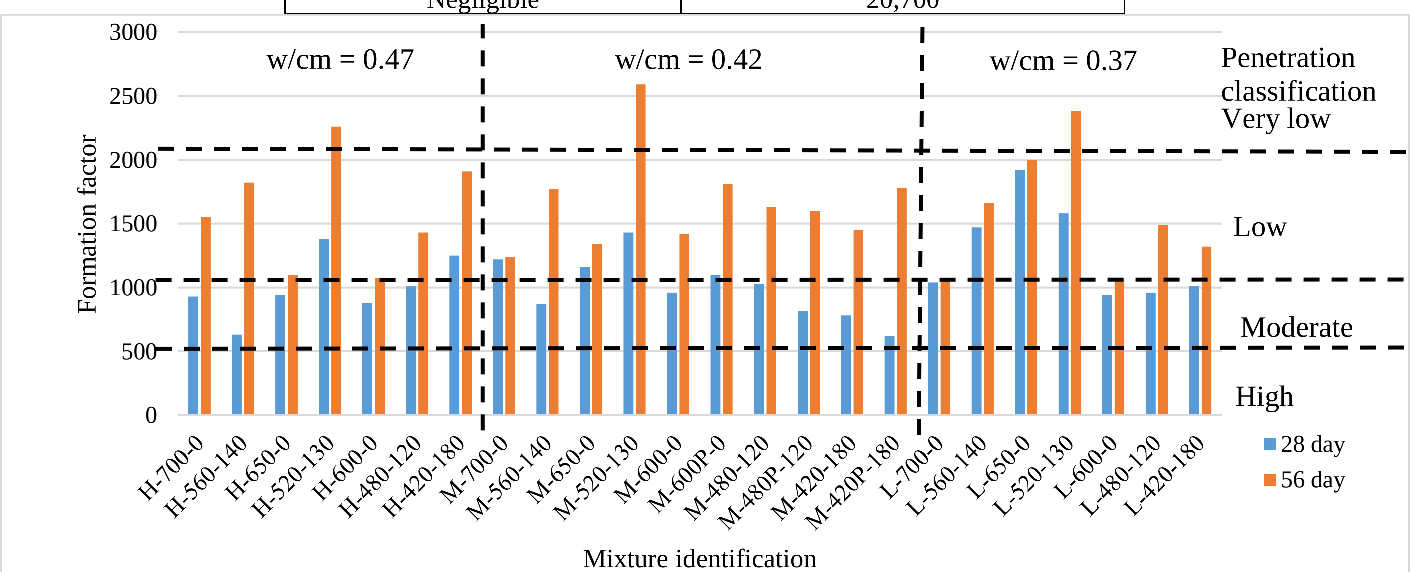

Figure 4.25: Formation factor values

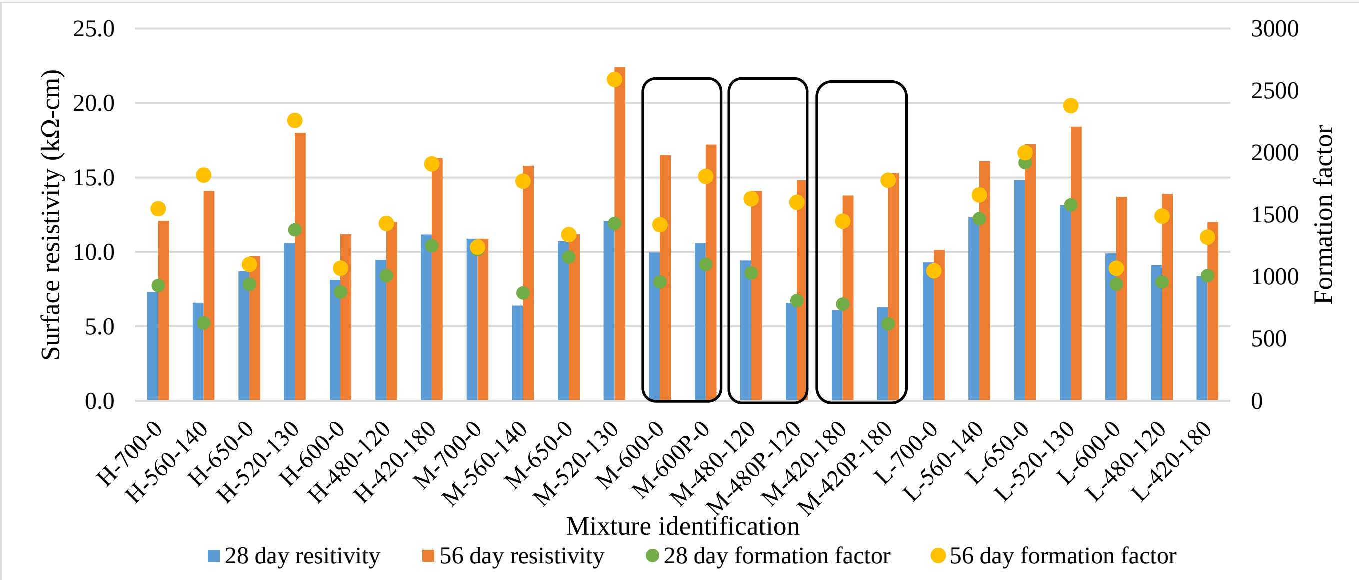

Figure 4.26: Formation factor and surface resistivity at 28 and 56 days

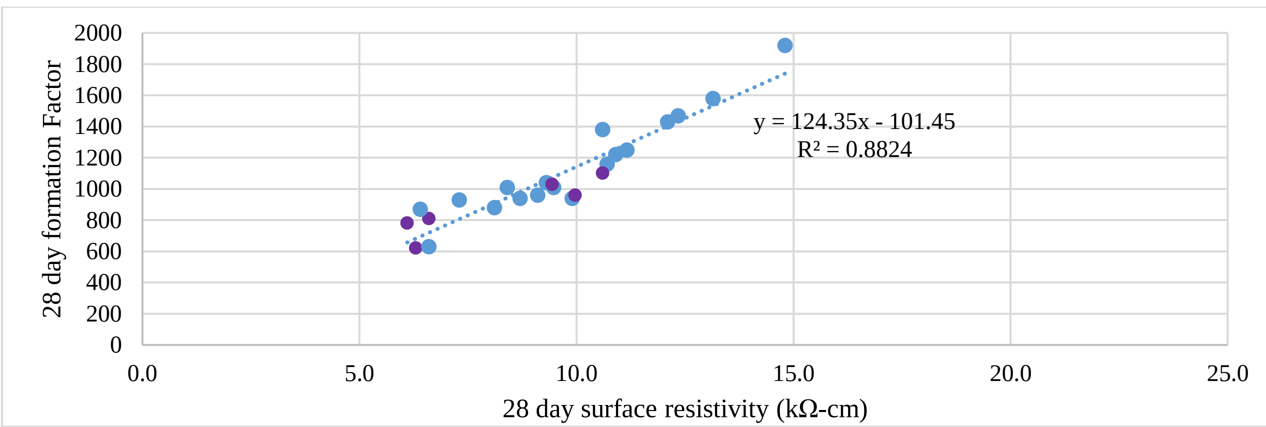

Figure 4.27: Formation factor vs. surface resistivity at 28 days

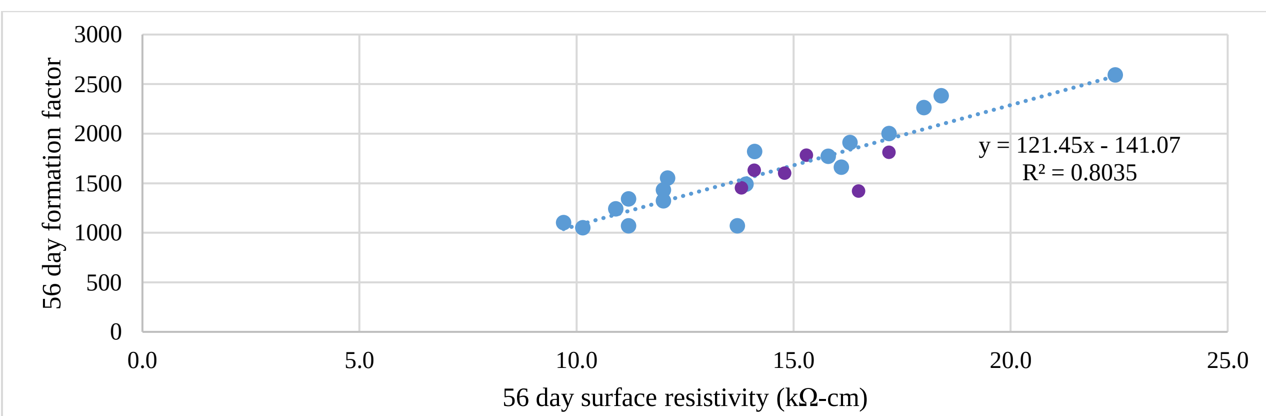

Figure 4.28: Formation factor vs. surface resistivity at 56 days

Based on the limited test data gathered as part of this study (which only utilizes two cements and one SCM), as well as ongoing current developments in the PEM initiative at the national level, use of the formation factor in NCDOT specifications is not recommended at this time. However, data found as part of this laboratory testing program shows a correlation between the chloride penetrability classifications given in AASHTO PP 84 for formation factor and RCPT, surface resistivity, and bulk resistivity. Ongoing developments associated with use of the formation factor, as well as related tests such as the Bucket Test, should be monitored and included in future PEM studies supported by NCDOT.

Volumetric shrinkage

Shrinkage tests were performed per ASTM C157, using concrete beam specimens consisting of 4 inch by 4 inch by 11¼ inch prisms. Measurements were made at 4, 7, 14, and 28 days, and at later ages, and results are provided in Appendix B in Table B.11. Since AASHTO PP 84 suggests a 28-day shrinkage target (focusing on *timely* performance criteria) be utilized, analysis of this data focused on 28-day test results. A summary of 28-day test results is shown in Figures 4.29, with variability indicated with range bars. Although the variation between specimen measurements is not judged to be particularly excessive, it is noted that some of the highest average shrinkage results were mixtures that had large variances between specimens. Figures 4.32 shows the 28-day shrinkage results sorted by non-fly ash and fly ash mixtures and Figure 4.33 shows the 28-day shrinkage results sorted by *w/cm* ratio. The 28-day maximum shrinkage target of 420 με suggested by AASHTO PP 84-19 is shown by the solid black line (AASHTO 2019).

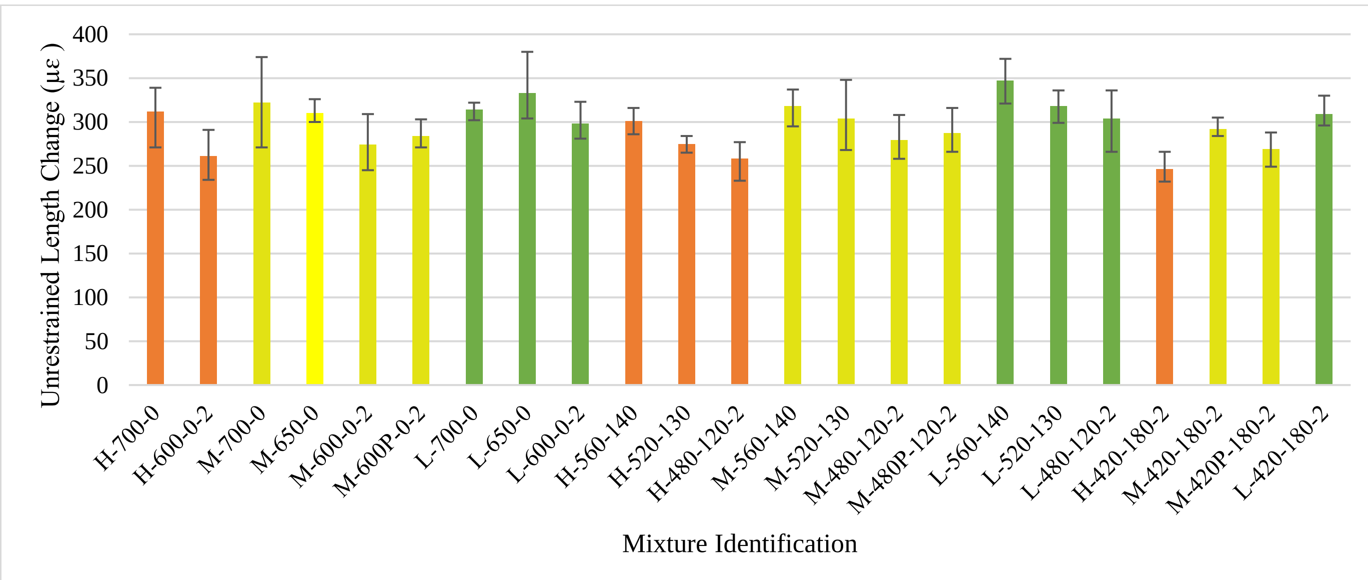

Figure 4.29: 28-day unrestrained shrinkage with variability

All of the mixtures readily met the 420με shrinkage limit suggested by AASHTO PP 84 (AASHTO 2019), and in fact all 28-day averages were lower than 350με. As can be observed from Figures 4.32 and 4.33, mixtures containing fly ash generally results in less shrinkage at 28 days for each *w/cm* ratio except the lowest *w/cm* (0.37). For mixtures with the lowest *w/cm* ratio of 0.37, mixtures without fly ash tended to have less shrinkage at 28 days than those with fly ash. For instance, the mixture with the highest length change at 28 days was mixture L-560-140. Along with having the lowest *w/cm* ratio, this mixture contained fly ash, but also contained 700 pcy total cementitious materials. This could likely be the cause of the high length change as the more cementitious material content, the more likely the structure is prone to cracking (Taylor et al. 2013), and this finding supports AASHTO PP 84 guidance regarding reducing paste content through optimized aggregate gradations and lower cementitious material contents. The two mixtures that had the smallest length change at 28 days were mixtures H-420-180 and H-480-120, which had the highest *w/cm* ratio (0.47). These mixtures had 30% and 20% fly ash replacement rates respectively, further supporting the fact that mixtures with SCMs tend to provide better durability performance than mixtures made with straight cement.

For paired OPC/PLC mixtures with the same *w/cm* ratio, performance results appeared to be dependent on fly ash content. Mixture M-420-180, which had a fly ash replacement rate of 30%, was outperformed by the PLC mixture M-420P-180 since it resulted in length change lower than the OPC mixture. On the other hand, mixture M-480-120 with a fly ash replacement rate of 20% had less change in length in comparison to the PLC mixture M-480P-120. Based on this data for mixtures of the same *w/cm* ratio, PLC mixtures with 30% fly ash replacement tend to perform better than PLC mixtures with 20% fly ash replacement rates.

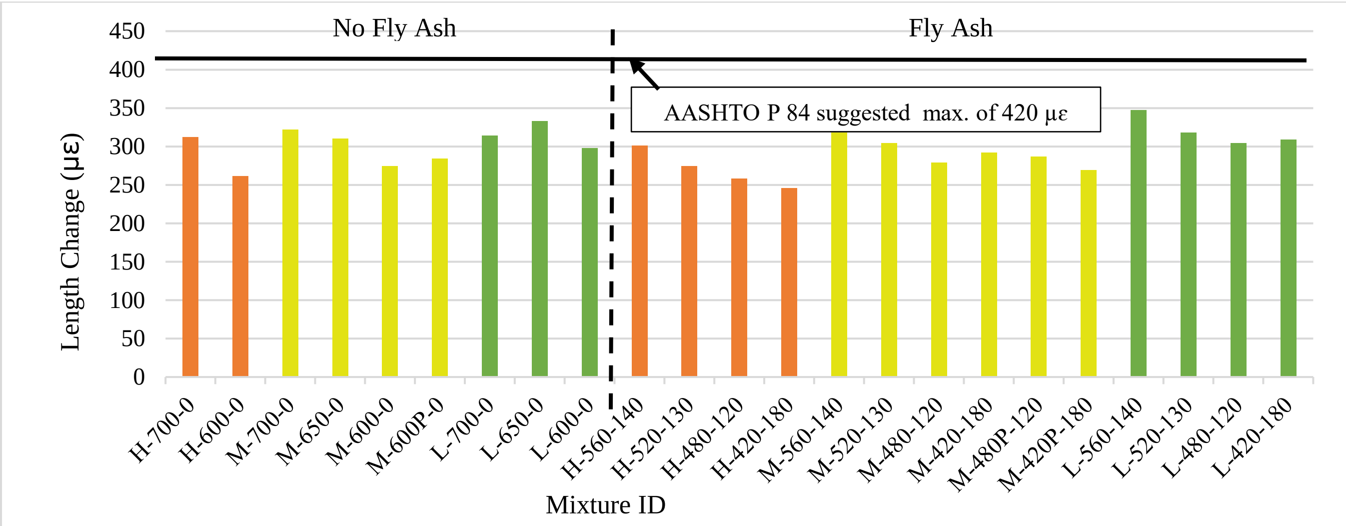

Figure 4.30: 28-day shrinkage results sorted by non-fly ash and fly ash mixtures in micro-strain (με)

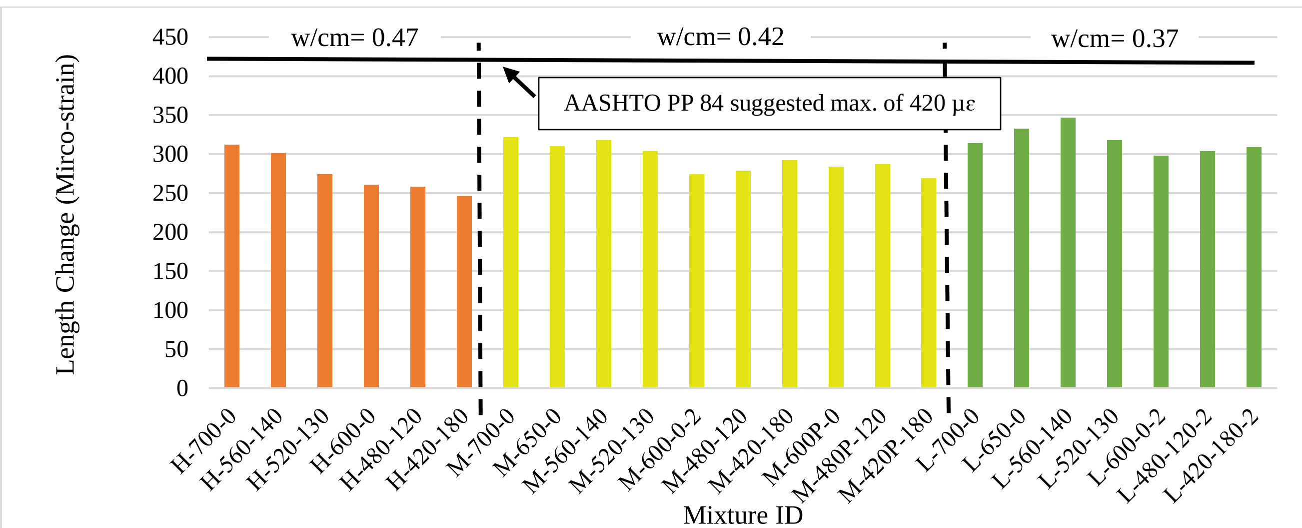

Figure 4.31: 28-day Shrinkage results sorted by *w/cm* ratios

4.4 Summary of Laboratory Findings

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

Fresh Properties

- Lower cementitious material contents and *w/cm* require higher dosages of WRA to achieve sufficient workability.
- Mixtures containing a fly ash replacement require higher dosages of WRA as well as AEA. Dosages for WRA have also shown to influence required AEA dosages.
- Higher *w/cm* and lower cementitious material contents typically showed higher unit weights.
- Fresh properties and admixture dosages for PLC mixtures did not differ significantly from those found for the companion OPC mixtures.

Mechanical Properties

- The generally accepted trends associated with *w/cm* were observed. Mixtures with lower *w/cm* typically outperformed the companion 0.42 and 0.47 *w/cm* mixtures in both compressive strength and MOE testing.
- NCDOT's decision to allow increased fly ash replacement rates (transitioning from 20% to 30% by weight of cement) has minimal impact on long-term strength. The difference in compressive strengths over all test dates was small, with 20% fly ash mixtures averaging an 8.0% higher compressive strength than companion straight cement mixtures. The results for MOE and MOR were even closer, with 20% fly ash mixtures having MOE results 1.0% higher on average, and MOR results 3.0% higher on average.
    - For 0.47 *w/cm* mixtures, the mixture with the 30% fly ash replacement had higher compressive strengths at 7, 28, 56, and 90 days by an average of 8.0%. The 20% ash replacement mixture had higher values of 3-day compressive strength, MOR, and MOE, but the average difference for each type of test was less than 14.0%.
    - Low *w/cm* (0.37) mixtures showed similar results as 0.42 and 0.47 *w/cm* mixtures when comparing 20% and 30% fly ash replacements. The 20% fly ash mixture had higher compressive strength results by an average of 7.0%, while 30% fly ash mixture had higher MOE and MOR results by an average of 8.0%.
- Measured MOE values were significantly lower (21.9%) than MOE values predicted using the ACI 318 equation. This finding matches findings of previous studies performed for NCDOT and may be of interest to designers.
- Significant differences in mechanical properties were not observed between PLC and OPC companion mixtures. PLC mixtures averaged higher compressive strength results at 7, 28, and 56 days by an average of 5.0%. OPC mixtures had slightly higher test results for 90-day compressive strength, MOR, and MOE, with average results 2.0%, 3.0%, and 3.0% higher, respectively.

Durability Performance

- Surface resistivity values are influenced by mixture characteristics and proportions.
    - Fly ash mixtures typically outperformed non-fly ash mixtures at later ages, with 56-day resistivity results higher than all non-fly ash companion mixtures.
    - Although it is known that mixtures with lower *w/cm* ratios typically provide improved (higher) surface resistivity test results, the difference for mixtures as part of this laboratory testing program did not provide trends as strong as in previous studies.
    - The influence of total cementitious material content on resistivity values is evident via greater improvements between 28- and 90-day tests for mixtures with lower total cementitious material contents, especially for those with fly ash.
    - Resistivity results for PLC mixtures improved with a fly ash replacement, specifically the higher (30%) replacement rate. Results for the non-fly ash mixtures showed the OPC mixture surpassing the values for the PLC mixture by 90 days. Test results for the 20% fly ash mixture showed minimal difference from the OPC and PLC mixtures, while the 30% replacement mixture had an interesting trend. At 28 days, average resistivity values were nearly identical, however at 56 days the PLC mixture showed an advantage over the OPC mixture, and further outperformed it at 90 days.
- Bulk resistivity test results typically improved with increasing fly ash replacements, particularly when comparing 28- and 90-day values. At 28 days, lower *w/cm* mixtures performed better than higher *w/cm* companion mixtures. Bulk resistivity values for PLC mixtures were comparable to OPC companion mixtures.
- Similar to other electrical tests to measure permeability, RCPT results benefitted from fly ash replacement. The 0.37 *w/cm* mixtures typically had RCPT values lower than higher *w/cm* companion mixtures. The 600 pcy

cementitious material mixtures typically outperformed 650 and 700 pcy mixtures, with the most noticeable difference evident for the 0.37 *w/cm* mixtures. At 28 days, this trend is seen for all mixtures except for 0.47 *w/cm* and 700 pcy mixtures and one 650 pcy mixture for the 0.42 and 0.37 *w/cm* mixtures. Differences between results for PLC and OPC mixtures were minimal, with the 600 pcy mixtures nearly identical, and the PLC mixture having better performances at both 28 and 90 days.

- Preliminary formation factor results show trends similar to other electrical resistivity tests. Mixtures with a fly ash replacement showed a performance advantage, particularly at later ages, when compared to non-fly ash mixtures. 28-day formation factor results showed improved performance for 0.37 *w/cm* mixtures, however results showed increased variability at later dates. In regards to total cementitious material content, the best performance was exhibited by 650 pcy mixtures, however when compared to the 700 pcy and 600 pcy mixtures, the difference was not judged to be significant. It should be noted that the testing and calculation method for formation factor testing is still being revised and improved, therefore these values are relevant only for preliminary observations.
- All mixtures tested exhibited volumetric shrinkage test results well below the 28-day AASHTO PP 84-19 suggested shrinkage limit of 420 $\mu\varepsilon$. Fly ash mixtures tended to be more resistant to shrinkage than the non-fly ash mixtures, supporting the fact that SCMs such as fly ash enhance the overall performance of concrete. Mixtures with 30% fly ash replacement rates showed reduced shrinkage by 28 days in comparison to several of the 20% fly ash replacement rate mixtures, as expected. An unrestrained shrinkage limit of 350 $\mu\varepsilon$ may be a more appropriate and readily achievable target for NC concrete mixtures.

5. EVALUATION OF RESULTS

The goal of this work was to identify preliminary target values for surface resistivity, early age opening to traffic, and volumetric shrinkage. Evaluation of data obtained from this project was supplemented with data from three previous research projects performed by the UNC Charlotte research team (two studies for NCDOT), supplementing the data obtained from the 24 mixtures included in this study with data obtained from an additional 23 mixtures from previous research studies (Cavalline et al., 2018, Ojo, 2018, and Cavalline et al., 2019). Mixture proportions for the 23 mixtures are shown in Table 5.1. The table has been color coded to shown pavement mixtures in green and structural mixtures in orange. Mixtures with the higher *w/cm* ratio of 0.48 are shown in purple and mixtures with the lower *w/cm* ratio of 0.35 are shown in green.

Each letter of the mixture identifications specifies a varied parameter. For paving mixtures from NCDOT RP 2015-03 (Cavalline et al. 2018), the first letter designates what region the coarse aggregate was the sourced from. P is for Piedmont Region, M is for Mountain Region, and C is for Coastal Region. The second letter designates type of cement used: A is OPC source A, B is OPC source B, and BL is PLC manufactured with the same clinker as OPC source B. The third letter represents rather or not the mixture includes fly ash: N indicates no fly ash, A is source A fly ash and B is source B fly ash (both as a 20% replacement rate for cement). The last letter of each mixture identification represents the fine aggregate type: N indicates natural sand and M indicates manufactured sand (Cavalline et al. 2018). Mixture CC is the control straight cement AA mixture and Mixture CF is the control fly ash mixtures from RP 2016-06 (Cavalline et al. 2019). Mixtures BC1, BC2, and BC3 are fly ash mixtures from a study funded by a private company (Ojo 2018).

Table 5.1: Characteristics and properties of the additional 23 mixtures included in the expanded dataset

Mixture ID	Mixture Characteristics			Mixture Proportions, pcy				
	Mixture type (Project Publication)	*w/cm*	Fly Ash Replacement (%)	Cement	Fly Ash	Coarse Aggregate	Fine Aggregate	Water
P.A.N.M.	Pavement (NCDOT RP 2015-03, Cavalline et al. 2018)	0.48	0	574	0	1798	1260	275
P.B.N.M.		0.48	0	574	0	1798	1260	304
P.BL.N.M.		0.48	0	574	0	1798	1260	275
C.A.N.M.		0.48	0	574	0	1661	1260	275
C.B.N.M.		0.48	0	574	0	1661	1260	275
C.BL.N.M.		0.48	0	574	0	1661	1260	275
M.A.N.M.		0.48	0	574	0	1798	1260	275
M.B.N.M.		0.48	0	574	0	1798	1260	275
M.BL.N.M.		0.48	0	574	0	1798	1260	275
P.A.A.M.		0.48	20*	460	137	1798	1260	304
P.B.A.M.		0.48	20*	460	137	1798	1260	275
P.BL.A.M.		0.48	20*	460	137	1798	1260	304
P.A.B.M.		0.48	20*	460	137	1798	1260	304
P.B.B.M.		0.48	20*	460	137	1798	1260	275
P.BL.B.M.		0.48	20*	460	137	1798	1260	304
P.A.N.N.		0.48	0	574	0	1798	1184	275
P.B.N.N.		0.48	0	574	0	1798	1184	304
P.BL.N.N.		0.48	0	574	0	1798	1184	275
BC1	Pavement (Ojo, 2018)	0.48	20*	460	137	1798	1094	291
BC2		0.48	20*	460	137	1798	1094	291
BC3		0.48	20*	460	137	1798	1094	291
CC	Bridge (Cavalline et al. 2019)	0.35	0	715	0	1720	1113	266
CF		0.35	20*	512	172	1720	1113	266

* The specification for these mixtures were 1.2 lb. of fly ash per 1.0 lb. of cement instead of the current 1:1 ratio (NCDOT 2012)

5.1 Development of a Surface Resistivity Specification

5.1.1 Analysis of Relevant Requirements

Standards implemented (or being proposed for implementation) by a number of state highway agencies were identified to provide insight into currently utilized specification targets for RCPT and surface resistivity. In total, 12 states with specification provisions were identified. The implementation level of these specifications ranged from project special provisions and to fully implemented specifications. Specifications for structural grade mixtures and pavement mixtures are summarized in Table 5.2. A summary table of RCPT and resistivity specifications for all types of mixtures is presented in Appendix C, Table C.1.

Table 5.2: Relevant state specifications for development of a surface resistivity specification

State/ Standard	**RCPT Specification**			**Resistivity Specification**		
	Concrete Type	**Require-ment (coulombs)**	**Age**	**Concrete Type**	**Require ment (kΩ-cm)**	**Age**
Virginia DOT design maximum lab permeability Note: [XXXX]* = design maximum lab permeability over tidal waters	A4 general	2500 [2000]*	28 days	-	-	-
	Low shrinkage A4 mod	2500 [2000]*	28 days	-	-	-
	A3a paving	3500 [3500]*	28 days	-	-	-

	A3b paving	3500 [3500]*	28 days	-	-	-
Florida DOT special circumstances (implemented AASHTO T 358 in January 2017)	-	-	-	Ternary blend - extremely aggressive environment	> 29	28 days
	-	-	-	Ternary blend - moderately aggressive environment	17 - 29	28 days
	-	-	-	Ternary blend - slightly aggressive environment	< 17	28 days
	-	-	-	Structural Concretes: Class IV, V, V (special), VI with use of silica fume, ultrafine fly ash, or metakaolin	≥ 29	28 days
New Hampshire DOT (SRT = surface resistivity test in kΩ-cm)	-	-	-	Class AA (Pay factor 1.05 - 0.06 (10 - SRT))	≥ 5 and ≤ 10	56 days
	-	-	-	Class AA (Pay factor 1.05)	> 10 and ≤ 35	56 days
	-	-	-	Class AA (Pay factor 1.05 + 0.0004347 (150 - SRT))	> 35 and ≤ 150	56 days
	-	-	-	Class AA (Pay factor 1.0)	> 150	56 days
Louisiana DOTD structural class concrete	-	-	-	Structural Concretes: Class A1, A2, A3; Prestressed Concretes: Class P1, P2, P3; CIP Structural: Class S	> 22	28 days
Kansas DOT special provisions	Concrete classified as high chloride risk	> 4000	28 days	Concrete classified as high chloride risk	< 7	28 days
	Concrete classified as moderate chloride risk	2000 - 4000	28 days	Concrete classified as moderate chloride risk	7 - 13	28 days
	Concrete classified as low chloride risk	1000 - 2000	28 days	Concrete classified as low chloride risk	13 - 24	28 days
	Concrete classified as very low chloride risk	100 - 1000	28 days	Concrete classified as very low chloride risk	24 - 190	28 days
	Concrete classified as negligible chloride risk	0 - 100	28 days	Concrete classified as negligible chloride risk	> 190	28 days
New Jersey DOT	-	-	-	HPC Design and Verification Requirements	≥ 36	56 days
	-	-	-	HPC Acceptance Requirements	≥ 19	56 days
	-	-	-	Concrete classified as high chloride risk	< 9	56 days
	-	-	-	Concrete classified as moderate chloride risk	9 - 20	56 days
	-	-	-	Concrete classified as low chloride risk	20 - 48	56 days
	-	-	-	Concrete classified as very low chloride risk	48 - 817	56 days
	-	-	-	Concrete classified as negligible chloride risk	> 817	56 days
New York DOT proposed thresholds for design mix performance criteria where specified	-	-	-	Superstructures and substructures	> 24	28 days
	-	-	-	Pavements, sidewalks, gutters, curbs, barriers, headwalls, drainage elements, pipe inverts, maintenance repair	> 16.5	28 days
New York DOT performance engineered concrete mixtures	Pay factor - 100%	≤ 1000	28 days	Pay factor - 100%	≥ 37	28 days
	Pay factor - 87.5%	> 1000 and ≤ 1500	28 days	Pay factor - 87.5%	< 37 and ≥ 27	28 days

for pavements based on application requirements	Pay factor - 75%	>1500 and ≤ 2500	28 days	Pay factor - 75%	< 27 and ≥ 19	28 days
	Reject concrete	>2500	28 days	Reject concrete	< 19	28 days
Rhode Island DOT concrete pre-qualification requirements	Structural and prestressed/ precast elements: Class HP	≤ 2000	28 days	Structural and prestressed/ precast elements: Class HP	≥ 15	28 days
	Structural and prestressed/ precast elements: Class HP	≤ 1000	28 day accelerated cure	Structural and prestressed/ precast elements: Class HP	≥ 21	56 days
Texas DOT	Pavement, structures, and other concrete construction	< 1500	56 days	-	-	-
	Pavement, structures, and other concrete construction	< 1500	28 day accelerated cure	-	-	-
UTAH DOT mix requirements	-	-	-	Class AA (LSF), AA (LS), AA (ES). (AA= bridge decks, LS= low shrinkage, LSF= low shrinkage with fibers, ES = Early strength. AA(LS) used for bridge decks & approach slabs, AA (AE) = other structural elements)	Must have "low to negligible risk" according to AASHTO T 358	
West Virginia DOT supplemental specs	Bridges	< 750	90 days	-	-	-
Montana DOT	-	-	-	Mix trial batches for Class "Deck" (superstructures, deck slabs, barriers) and "Overlay S-F" (silica fume overlays)	> 21	28 days

After review of existing state resistivity and RCPT specifications, the research team decided to focus upon specifications of Virginia's DOT (VDOT). Virginia was determined to be the most similar to NC due to its 1) proximal geographical location and similar climate, 2) similar mountain, piedmont, and coastal regions, and 3) similar population distribution (major urban corridors and rural lands) and highway network conditions. VDOT has also shown improved permeability characteristics in RCPT results through the use of SCMs (Sharp et al. 2014). It was also determined that provisions or targets of a number of state specifications in Table 5.1 were likely not appropriate for NCDOT mixtures due to various aspects of the specifications. These include specifications of Florida, Louisiana, Montana, New York, and Utah. Specification targets for these states were viewed as too aggressive for recommendation to NCDOT, as it was apparent that typical NCDOT mixtures will likely not meet these targets, particularly at early test ages. Many of these states commonly utilize ternary blends (portland cement with two or more SCMs to improve durability), which are not as commonly used in NC concrete mixtures. One example of a provision viewed as too aggressive for current NC concrete mixtures is the rejection of concrete by New York if surface resistivity results are less than 19 kΩ-cm. Other provisions viewed undesirable for use by NCDOT at this point in resistivity specification development included linking targets to pay factors.

5.1.2 Development of Performance Targets for a Surface Resistivity Specification

VDOT specifies their permeability requirements based upon RCPT and does not currently utilize a surface resistivity specification. However, as shown in this study and in previous research project for NCDOT (Cavalline et al. 2018, Cavalline et al. 2019), NC concrete mixtures show a strong correlation between RCPT and surface resistivity. For these RCPT vs. surface resistivity plots, a power model was chosen to show the relationship between the two sets of data, as previous research projects and literature have shown this is the best fit. For mixtures produced in the laboratory portion of this project, RCPT and surface resistivity data showed an R^2 of 0.54 (Figure 5.1). Previous research studies performed by the research team both had R^2 values of 0.94 (RP 2015-03 Cavalline et al. 2018, and RP 2016-06 Cavalline et al. 2019). The expanded dataset, including mixtures produced for this project and two previous projects for NCDOT is shown plotted in Figure 5.2 had an R^2 value of 0.77.

RCPT targets based on VDOT's current specifications were utilized to identify corresponding surface resistivity targets using NC data. The two numbers of interest for application for NCDOT were VDOT's 2,500 coulomb requirement for "Class A4 General" (structural) and 3,500 coulombs for "A3a Paving" mixtures. As shown in Figure 5.1 and 5.2, the surface resistivity measurements associated with these RCPT values for NCDOT mixtures are approximately 10.5 kΩ-cm for the 3,500 coulomb RCPT value for pavements, and 18.8 kΩ-cm for the 2,500 coulomb RCPT value for bridges.

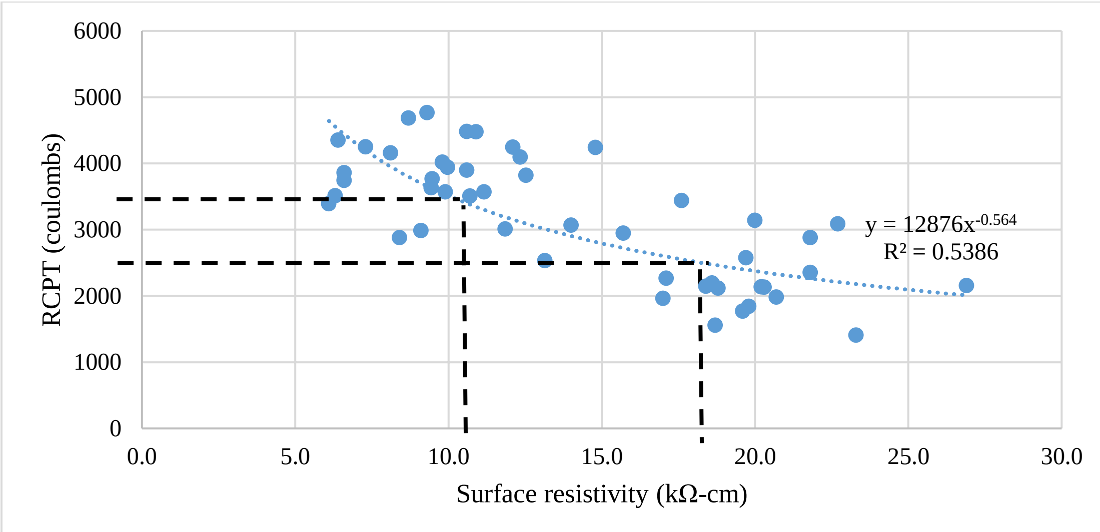

Figure 5.1: RCPT results vs. surface resistivity test results for RP 2018-14, with performance targets from VDOT for bridges (2500 coulombs) and pavements (3500 coulombs) denoted

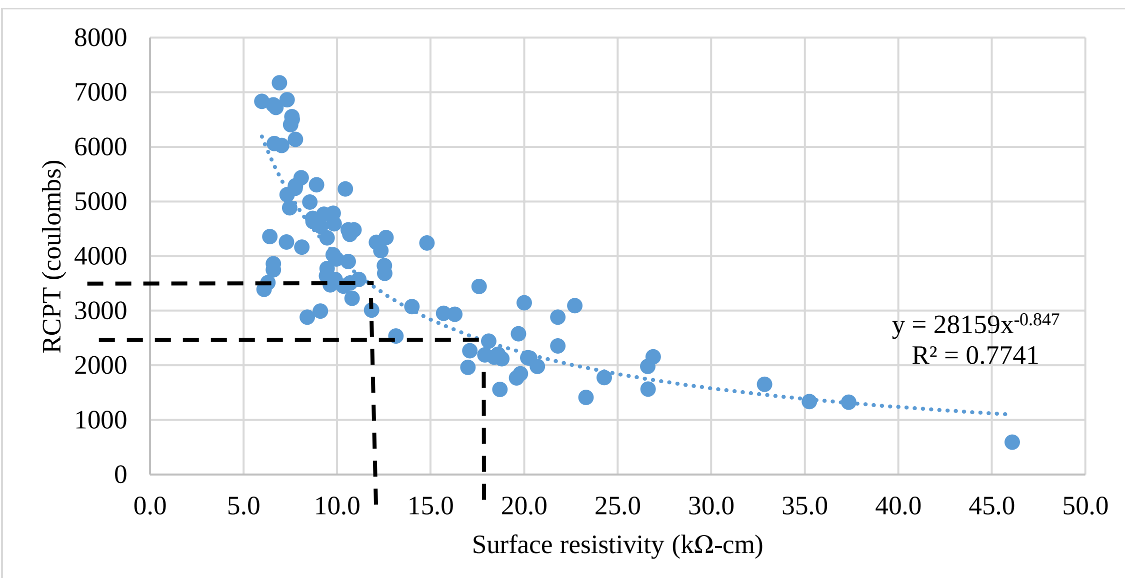

Figure 5.2: RCPT vs. surface resistivity for North Carolina mixtures, with performance targets from VDOT for bridges (2500 coulombs) and pavements (3500 coulombs) denoted

VDOT's values are associated with 28-day RCPT test results. Many NC mixtures would not have met the associated 10.5 kΩ-cm and 18.8 kΩ-cm at 28 days, although many mixtures (particularly those with moderate to low *w/cm* and those using fly ash) could readily meet these targets at 56 days. As a result, it is recommended that the target values be applied to 56-day surface resistivity testing to encourage use of fly ash mixtures, lower *w/cm* ratios, and other SCMs in NC infrastructure. New Jersey DOT also followed the same rationale in establishing 56-day targets, noting the significant increase in surface resistivity results and durability between 28 and 56 days for fly ash mixtures (Nassif et al. 2015). Figure 5.3 shows surface resistivity results from the expanded dataset, colored to represent paving mixtures in green and bridge mixtures in orange, with fly ash mixtures identified with a dot marker.

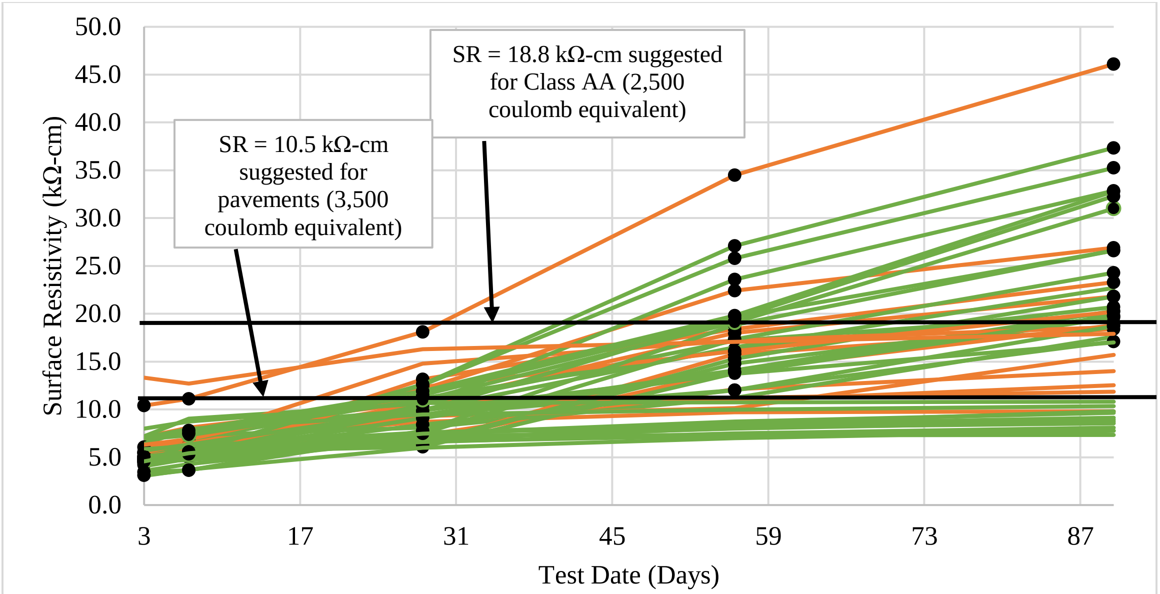

Figure 5.3: Surface resistivity data with target resistivity goals identified

Additional analysis was performed to assess the feasibility of these 10.5 kΩ-cm (pavement) and 18.8 kΩ-cm (structural) targets. First, it was desired that a whole number be identified as the target, since this is a simplified approach that should aid in initial implementation. A series of tables was created, tabulating the mixtures in the expanded dataset that passed and failed at various target values close to the 10.5 kΩ-cm (pavement) and 18.8 kΩ-cm (structural) targets. For each target, the percentage of mixtures in the expanded dataset passing at 28 and 56 days was calculated. For bridge (AA) mixtures, this can be seen in Tables 5.3 and 5.4. Note that mixtures added to the analysis included control mixtures (CC and CF) batched and tested as part of NCDOT RP 2016-06 (Cavalline et al. 2019). Upon finding the percent of mixtures passing at various target values, evaluation of the mixture characteristics of those passing *and failing* at each target was performed (Table 5.5). This was done to ensure the mixtures designed to have better durability properties fell within the passing mixtures at 56 days, while mixtures having higher *w/cm* ratios and/or no SCM content tended to not meet the target. For bridge mixtures, this full evaluation is provided in Appendix C, Tables C.2 through C.6.

Table 5.3: Analysis of bridge mixtures passing with higher performance targets

Target values	18.0 kΩ-cm		17.0 kΩ-cm		16.0 kΩ-cm		15.0 kΩ-cm	
Age	28 days	56 days	28 days	56 days	28 days	56 days	28 days	56 days
Mixtures passing target value	CF	H-520-130	CF	H-520-130	CF	H-520-130	CF	H-520-130
		M-520-130		M-520-130	CC	M-520-130	CC	M-560-140
		L-520-130		L-560-140		L-560-140		M-520-130
		CF		L-650-0		L-650-0		L-560-140
				L-520-130		L-520-130		L-650-0
				CF		CF		L-520-130
						CC		CF
								CC
Percent passing	7.1%	28.6%	7.1%	42.9%	14.3%	50.0%	14.3%	57.1%

Table 5.4: Analysis of bridge mixtures passing with lower performance targets

Target values	14.0 kΩ-cm		13.0 kΩ-cm		12.0 kΩ-cm		11.0 kΩ-cm	
Age	28 days	56 days	28 days	56 days	28 days	56 days	28 days	56 days
Mixtures passing target value	L-650-0	H-560-140	L-650-0	H-560-140	M-520-130	H-700-0	M-520-130	H-700-0
	CF	H-520-130	L-520-130	H-520-130	L-560-140	H-560-140	L-560-140	H-560-140
	CC	M-560-140	CF	M-560-140	L-650-0	H-520-130	L-650-0	H-520-130
		M-520-130	CC	M-520-130	L-520-130	M-560-140	L-520-130	M-560-140
		L-560-140		L-560-140	CF	M-650-0	CF	M-650-0
		L-650-0		L-650-0	CC	M-520-130	CC	M-520-130
		L-520-130		L-520-130		L-560-140		L-560-140
		CF		CF		L-650-0		L-650-0
		CC		CC		L-520-130		L-520-130
						CF		CF
						CC		CC
Percent passing	21.4%	64.3%	28.6%	64.3%	42.9%	78.6%	42.9%	78.6%

Table 5.5: Bridge mixtures passing/not passing at 28 and 56 days for performance targets 16.0 kΩ-cm and 15.0 kΩ-cm

Target value	Meeting 16.0 kΩ-cm		Not meeting 16.0 kΩ-cm		Meeting 15.0 kΩ-cm		Not meeting 15.0 kΩ-cm	
Age	28 day	56 day	28 day	56 day	28 day	56 day	28 day	56 day
Mixture identification	CF	H-520-130	H-700-0	H-700-0	CF	H-520-130	H-700-0	H-700-0
	CC	M-520-130	H-650-0	H-650-0	CC	M-520-130	H-650-0	H-650-0
		L-520-130	M-700-0	M-700-0		L-520-130	M-700-0	M-700-0
		CF	M-650-0	M-650-0		CF	M-650-0	M-650-0
		L-650-0	L-700-0	L-700-0		L-650-0	L-700-0	L-700-0
		CC	L-650-0	H-560-140		CC	L-650-0	H-560-140
		L-560-140	H-560-140	M-560-140		L-560-140	H-560-140	
			H-520-130			M-560-140	H-520-130	
			M-560-140				M-560-140	
			M-520-130				M-520-130	
			L-560-140				L-560-140	
			L-520-130				L-520-130	

Evaluation of the above tables along with Tables D.2 through D.6 showed that for bridge mixtures at 56 days, a target surface resistivity value of either 15.0 kΩ-cm or 16.0 kΩ-cm had a sufficient number of mixtures meeting the target, each exhibiting good durability performance characteristics in laboratory testing. Mixtures passing at these targets at 56 days were also judged to have characteristics representative of mixtures historically linked to suitable field performance (e.g., mixtures with low to moderate *w/cm* (0.37 to 0.42), and mixtures including fly ash). On the contrary, mixtures not passing at these targets were those mixtures which may not historically provide suitable durability performance (e.g. high *w/cm* mixtures, mixtures without fly ash). A surface resistivity target of 15.0 kΩ-cm would correspond to an RCPT value of approximately 2,800 coulombs, and a surface resistivity target of 16.0 kΩ-cm would correspond to an RCPT value of approximately 2,700 coulombs. Both targets would appear to reasonably discern between mixtures with higher and lower durability performance potential, with the target of 16.0 kΩ-cm providing an aggressive, but realistically feasible performance target for structural mixtures.

Although a surface resistivity value of 15.0 to 16.0 kΩ-cm should provide sufficient resistance to chloride ingress for structural concrete, the question regarding age at the time of meeting the surface resistivity target must be addressed. Figure 5.4 (an excerpt from Figure 5.3) shows the surface resistivity values for straight cement mixtures (those not including fly ash), with orange dots indicating higher (700 pcy or greater) cement contents, and yellow dots indicating mid-range (650 pcy) cement contents. It is evident that the many of the surface resistivity values typically do not obtain values as high at later ages when compared to fly ash mixtures, with only two (CC and L-650-0) meeting the suggested 56 day performance targets. These mixtures are identified in Figure 5.4.

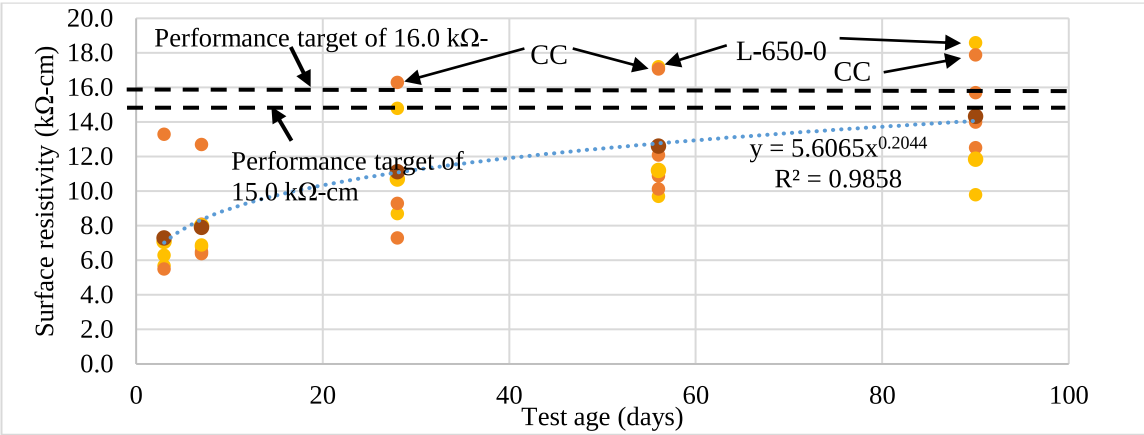

Figure 5.4: Surface resistivity averages for straight cement bridge mixtures with 15.0 and 16.0 kΩ-cm targets

Analysis indicated that a suitable target surface resistivity for structural concrete could be either 15.0 or 16.0 kΩ-cm at 56 days. Mixtures passing at these targets at 56 days were judged to have characteristics representative of mixtures historically linked to suitable field performance (e.g., mixtures with low to moderate *w/cm* (0.37 to 0.42), and mixtures including fly ash). On the contrary, mixtures not passing at these targets were those mixtures which may not historically provide suitable durability performance (e.g. high *w/cm* mixtures, mixtures without fly ash). A surface resistivity target of 15.0 kΩ-cm would correspond to an RCPT value of approximately 2,800 coulombs, and a surface resistivity target of 16.0 kΩ-cm would correspond to an RCPT value of approximately 2,700 coulombs. Both targets appear to reasonably discern between mixtures with higher and lower durability performance potential, with the target of 16.0 kΩ-cm providing an aggressive, but realistically feasible performance target for structural mixtures. For pavement mixtures, a similar approach was taken to identify performance resistivity targets. A resistivity target of 11.0 kΩ-cm appears to be a suitable target at 56 days for paving mixtures. This value roughly corresponds to an RCPT value of approximately 3,300 coulombs.

Table 5.6: Analysis of paving mixtures passing with various performance targets

Target values	11.0 kΩ-cm		10.0 kΩ-cm	
Age	28 days	56 days	28 days	56 days
Mixtures passing target value	H-420-180	H-600-0	H-420-180	H-600-0
	P.BL.A.M	H-480-120	M-600-0	H-480-120
	P.BL.B.M	H-420-180	M-600P-0	H-420-180
	BC1	M-600-0	P.BL.A.M	M-600-0
	BC2	M-600P-0	P.B.B.M	M-600P-0
	BC3	M-480-120	P.BL.B.M	M-480-120
		M-480P-120	P.B.N.N	M-480P-120
		M-420-180	BC1	M-420-180
		M-420P-180	BC2	M-420P-180
		L-600-0	BC3	L-600-0
		L-480-120		L-480-120
		L-420-180		L-420-180
		P.A.A.M		P.A.A.M
		P.B.A.M		P.B.A.M
		P.BL.A.M		P.BL.A.M
		P.A.B.M		P.A.B.M
		P.B.B.M		P.B.B.M
		P.BL.B.M		P.BL.B.M
		BC1		P.B.N.N
		BC2		P.BL.N.N
		BC3		BC1
				BC2
				BC3
%passing	18.18%	63.64%	30.30%	69.70%

Table 5.7: Paving mixtures passing and failing at 28 and 56 days for various performance targets

Target value	Meeting 11.0 kΩ-cm		Not meeting 11.0 kΩ-cm		Meeting 10.0 kΩ-cm		Not meeting 10.0 kΩ-cm	
Age	28 day	56 day	28 day	56 day	28 day	56 day	28 day	56 day
Mixture identification	P.BL.A.M	H-600-0	H-600-0	P.A.N.M	M-600-0	H-600-0	H-600-0	M-600-0
	P.BL.B.M	M-600-0	M-600-0	P.B.N.M	M-600P-0	M-600-0	L-600-0	M-600P-0
	BC1	M-600P-0	M-600P-0	P.BL.N.M	P.B.N.N	M-600P-0	P.A.N.M	P.B.N.N
	BC2	L-600-0	L-600-0	C.A.N.M	P.BL.A.M	L-600-0	P.B.N.M	P.BL.A.M
	BC3	H-480-120	P.A.N.M	C.B.N.M	P.B.B.M	P.B.N.N	P.BL.N.M	P.B.B.M
	H-420-180	M-480-120	P.B.N.M	C.BL.N.M	P.BL.B.M	P.BL.N.N	C.A.N.M	P.BL.B.M
		M-480P-120	P.BL.N.M	M.A.N.M	BC1	H-480-120	C.B.N.M	BC1
		L-480-120	C.A.N.M	M.B.N.M	BC2	M-480-120	C.BL.N.M	BC2
		P.A.A.M	C.B.N.M	M.BL.N.M	BC3	M-480P-120	M.A.N.M	BC3
		P.B.A.M	C.BL.N.M	P.A.N.N	H-420-180	L-480-120	M.B.N.M	H-420-180
		P.BL.A.M	M.A.N.M	P.B.N.N		P.A.A.M	M.BL.N.M	
		P.A.B.M	M.B.N.M	P.BL.N.N		P.B.A.M	P.A.N.N	
		P.B.B.M	M.BL.N.M			P.BL.A.M	P.BL.N.N	
		P.BL.B.M	P.A.N.N			P.A.B.M	H-480-120	
		BC1	P.B.N.N			P.B.B.M	M-480-120	
		BC2	P.BL.N.N			P.BL.B.M	M-480P-120	
		BC3	H-480-120			BC1	L-480-120	
		H-420-180	M-480-120			BC2	P.A.A.M	
		M-420-180	M-480P-120			BC3	P.B.A.M	
		M-420P-180	L-480-120			H-420-180	P.A.B.M	
		L-420-180	P.A.A.M			M-420-180	M-420-180	
			P.B.A.M			M-420P-180	M-420P-180	
			P.A.B.M			L-420-180	L-420-180	
			P.B.B.M					
			M-420-180					
			M-420P-180					
			L-420-180					

Although a surface resistivity value of 11.0 kΩ-cm could reasonably serve as a preliminary target to ensure sufficient resistance to chloride ingress for NC pavement mixtures, the question regarding age must be addressed. Figure 5.5 shows surface resistivity averages for straight cement paving mixtures, an excerpt from Figure 5.3. These mixtures are shown in green to indicate a low (600 pcy or less) cement content. Similar to bridge mixtures, it is evident that fly ash mixtures outperform their straight cement counterparts, particularly at later ages, with only four straight cement mixtures (H-600-0, M-600-0, M-600P-0, and L-600-0) meeting the suggested 56 day performance target.

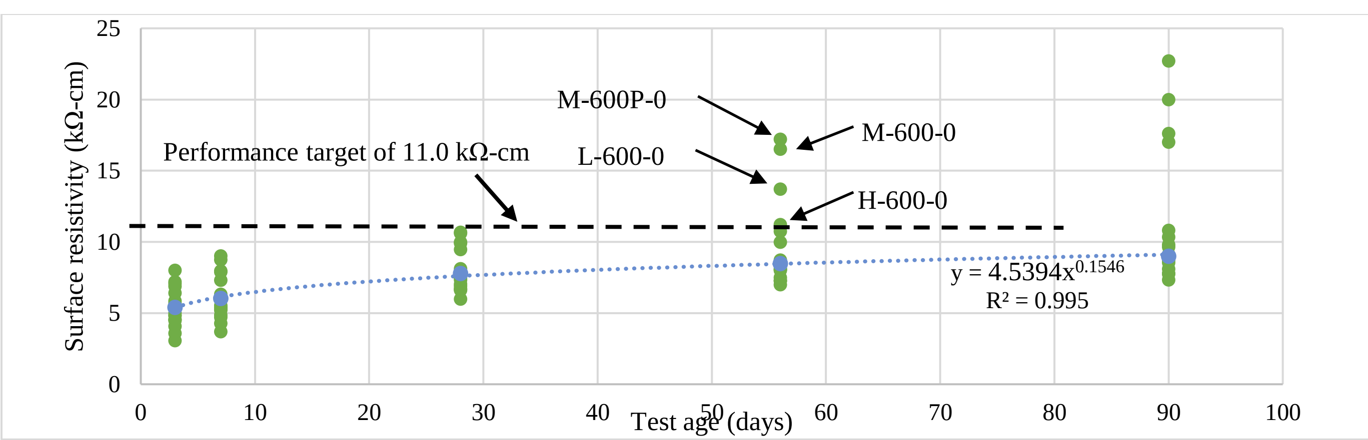

Figure 5.5: Surface resistivity averages for straight cement paving mixtures with 11.0 kΩ-cm target

Based upon the test results from the 24 mixtures included in this projects dataset, the targets of 11.0 kΩ-cm and 15.0 or 16.0 kΩ-cm appear reasonable for pavement and structural concrete, respectively. However, when the expanded dataset is included in the surface resistivity and RCPT curve, as shown in Figure 5.2, it is evident that more aggressive surface resistivity targets may be warranted in the future, as stakeholder experience provides comfort with the test and field performance is linked to a growing database of surface resistivity values. As shown in Figure 5.2, the expanded dataset provides evidence that a slightly more aggressive resistivity target for pavements and slightly less aggressive target for bridges (11.7 kΩ-cm and 17.5 kΩ-cm, instead of 10.5 and 18.8 kΩ-cm) correspond to RCPT values of 3,500 coulombs and 2,500 coulombs. Future work should include linking performance data with measured surface resistivity and/or historical RCPT data to evaluate whether recommended surface resistivity targets should be made more aggressive to promote more durable infrastructure. In the existing coastal corrosive zones, a higher resistivity could be warranted. Findings from RP 2019-22, a study of NCDOT's corrosive sites policy should help identify target values for these critical zones.

5.1.3 Suggested Shadow Specification for Surface Resistivity

The following is suggested as a revision to Section 1000-4C "Portland Cement Concrete for Structures and Incidental Construction" of the NCDOT 2018 Standard Specifications for Roads and Structures (NCDOT 2018). The method in which this specification is suggested for implementation is the same manner in which LADOTD initially implemented surface resistivity testing (LADOTD 2018). Pairing surface resistivity testing with compressive strength testing should ease the transition to adding the test, as it can be run on the same cylinders used for compressive tests. The revised Section 1000-4C is presented as follows, with recommended changes shown in *italics*:

> (C) Strength and Surface Resistivity of Concrete
> The compressive strength *and surface resistivity* of the concrete will be considered the average test results of two 6 inch x 12 inch cylinders, or two 4 inch x 8 inch cylinders if the aggregate size is not larger than size 57 or 57M. Make cylinders in accordance with AASHTO T 23 from the concrete delivered to the work. Make cylinders at such frequencies as the Engineer may determine and cure them in accordance with AASHTO T 23 as modified by the Department. Copies of these modified test procedures are available upon request from the Materials and Tests Unit. Testing for compressive strength should be performed in accordance with AASHTO T 22. *Testing for surface resistivity should be performed in accordance with AASHTO T 358.* When the average compressive strength or surface resistivity of the concrete test cylinders is less than the minimum targets specified in Table 1000-1 and the Engineer determines it is within reasonably close conformity with design requirements, these properties will be considered acceptable. *When the Engineer determines average cylinder strength or surface resistivity is below the specification, the in-place concrete will be tested.* Based on these test results, the concrete will either be accepted with no reduction in payment or accepted at a reduced unit price or rejected as set forth in Article 105-3.

The following table (Table 5.8) would be added or incorporated into Table 1000-1 with the associated footnote.

Table 5.8: Suggested addition to NCDOT specification for roads and structures

Class of Concrete	Minimum surface resistivity at 56 days (kΩ-cm)
AA	15.0*
Pavement	11.0

*A 56 day minimum of 16.0 kΩ-cm can be required at the engineer's discretion for applications where risk of chloride ion penetration is high.

5.2 Development of a Specification for Early Age Opening to Traffic

Section 700-13 "Use of New Pavement or Shoulder" of the 2018 NCDOT standard specifications states that "traffic or other heavy equipment will not be allowed on the concrete pavement or shoulder until the estimated compressive strength of the concrete using the maturity method has exceeded 3,000 psi." Compressive strength of concrete pavement is to be estimated using the maturity method in accordance with ASTM C1074 (ASTM 2019) unless otherwise specified. Digital datalogging maturity meters must be capable of storing at least 28 days of data, and loggers should be installed approximately 4 inches from the surface of the concrete after every two lots of concrete. The strength-maturity relationship should be developed using compressive strength tests at ages 1, 3, 7, 14, and 28 days. The temperature-time factor (TTF) from ASTM C1074 should be computed per Equation 1 in ASTM C1074, with a datum temperature of -10°C. The contractor should establish and submit a strength-maturity relationship in conjunction with each concrete pavement mixture design, and the TTF corresponding to 3,500 psi should be determined. A new strength-maturity relationship must be developed if any changes occur during production, unless otherwise directed. During construction the correlation between the strength-maturity relationship (and TTF) with compressive strength of cylinders and beams is validated during the first day's production. It is also to be validated at least every 30 calendar days, when the TTF varies by more than 10% from the last approved maturity curve, or if there is a material change from the approved mixture design. Also, the specifications require that "if the verification sample's compressive strength when tested at TTF is less than 3,000 psi, immediately suspend early opening of traffic on pavement that has not obtained TTF until a new strength-maturity relationship is established" (NCDOT 2018).

Section 420-20 "Placing Load on Structure Members" of the 2018 NCDOT standard specifications states that beams or girders cannot be placed on concrete substructures until the concrete in the substructure develops a minimum compressive strength of 2,400 psi. Vehicles or construction equipment cannot be placed on a bridge deck until the deck concrete develops the minimum specified 28-day compressive strength and attains an age of at least 7 curing days. A curing day is defined in Section 420-15(A) as "any consecutive 24-hour period, beginning when the manipulation of each separate mass is complete, during which the air temperature adjacent to the mass does not fall below 40°F (NCDOT 2018)." After initial set, the screed cannot be rolled across the bridge deck until the deck attains a compressive strength of at least 1,500 psi (NCDOT 2018). Bridge approach slabs can be traversed by construction equipment after the slab reaches a compressive strength of at least 3,000 psi with a minimum of 7 curing days. To remove formwork for bridge decks, beams, and girders a compressive strength of 3,000 psi is required (NCDOT 2018).

5.2.1 Analysis of Relevant Requirements

Standards implemented by a number of state highway agencies were identified to provide insight into currently utilized specification targets early age opening to traffic. In total, nine states with specification provisions were identified and summarized, as shown in Table 5.8. Several of the concrete strength requirements for opening pavements and structures are similar to AASHTO PP 84 recommendations but are not specific to PEMs. Concrete age at time of testing dates varies by state, but most SHAs required more conservative standards for bridge decks and structural concrete than those for concrete pavements. More specifically, most states require a higher strength for opening concrete bridge decks and structures to traffic in comparison to opening pavements. Out of the nine state specifications included in Table 5.9, four of them have specifications for high early strength (HES) concrete. Each of the four states shown with HES standard specifications vary in compressive and flexural strength requirements. For HES mixtures the opening strength requirements were expected to be obtained in 4 to 48 hours (LaDOTD, 2016, MnDOT, 2016, and IowaDOT, 2015).

Table 5.9: Selected agency specifications on early-age compressive and flexural strength

State/ Standard	**Concrete Type**	**Construction Equipment Requirement (psi)**		**Age (days)**	**Regular Traffic (psi)**		**Age* (days)**
		Compressive	**Flexural**		**Compressive**	**Flexural**	
Florida DOT	Class A paving	2,200	-	14	3,000	550	28
	Class II bridge deck	1,600 if verified by Engineer	-	14	4,500	-	28
Illinois DOT	PV paving	3,500	650	7 or 14	min of 3,500 or 650 by 14 days prior to loading		
	BS bridge deck	4,000	675	14	min of 4,000 or 675 by 14 days prior to loading		
Iowa DOT	Class A paving (unless otherwise noted in contract)	depends on project	500	14	specified by project, approved by engineer		
	HES Class M paving	depends on project	500	48 hrs.	specified by project, approved by engineer		
	Class A bridge deck	depends on project	550	7	specified by project, approved by engineer		

Louisiana DOTD	B and D paving	3,000	550	7	3,000	only if Engineer req.	14
	HES mod. A1 paving	3,000	-	4 hrs.	4,500	-	28
	Class A1, A2, A3 bridge deck (A/AA)	4,000	-	14	4,500	only if Engineer req.	28
Minnesota DOT	Class A paving	3,000	500-350 (depends on slab thickness)	7	4,500	-	28
	HES Grade F paving or structural	3,000	-	48 hrs.	4,500	-	28
	Y bridge deck	100% req. strength	500	7	4,000	-	28
New York DOT	Class A, C paving	2,500	-	3,7	4,000	600	28
	HES Class F paving or structural	2,500	-	-	4,000	-	28
	Class A or project specified	depends on project	-	-	depends on project	-	28
Virginia DOT	A3 paving	maturity method	600	14	3,000	600	28
	HES Class A4	3,500	-	7	3,500 must be achieved in 7 days prior to loading		
	A4 bridge deck	maturity method	-	14	4,000	-	28
West Virginia DOH	Class A paving	maturity method or prove 28-day strength met		4,6,8	3,000	500	28
	Class H bridge decks	3000 or maturity method	-	7	4,000		28

* 28-days is not the requirement wait time for opening to regular traffic, it represents the age when the concrete should reach the strength required.

5.2.2 Development of Performance Targets for Early Age Strength Specification Provisions

Currently NCDOT specifies an early-age strength requirement to open to traffic of 3,000 psi for pavements and bridge deck approach slabs, with a requirement of 4,500 psi compressive and 650 psi flexural strength for acceptance. Bridge decks must reach the design strength of 4,500 psi to open to vehicles or construction traffic (NCDOT 2018). Three-day compressive strength results for pavement (lower cement content) mixtures batched as part of this work are shown in Figure 5.6 (sorted by *w/cm* ratio), and in Figure 5.7 (sorted by fly ash content). All pavement mixtures except H-480-120 and H-420-180 surpassed 3,000 psi at the age of 3 days. This mixture had the highest *w/cm* (0.47) at the highest fly ash content (30%), so this result is as expected.

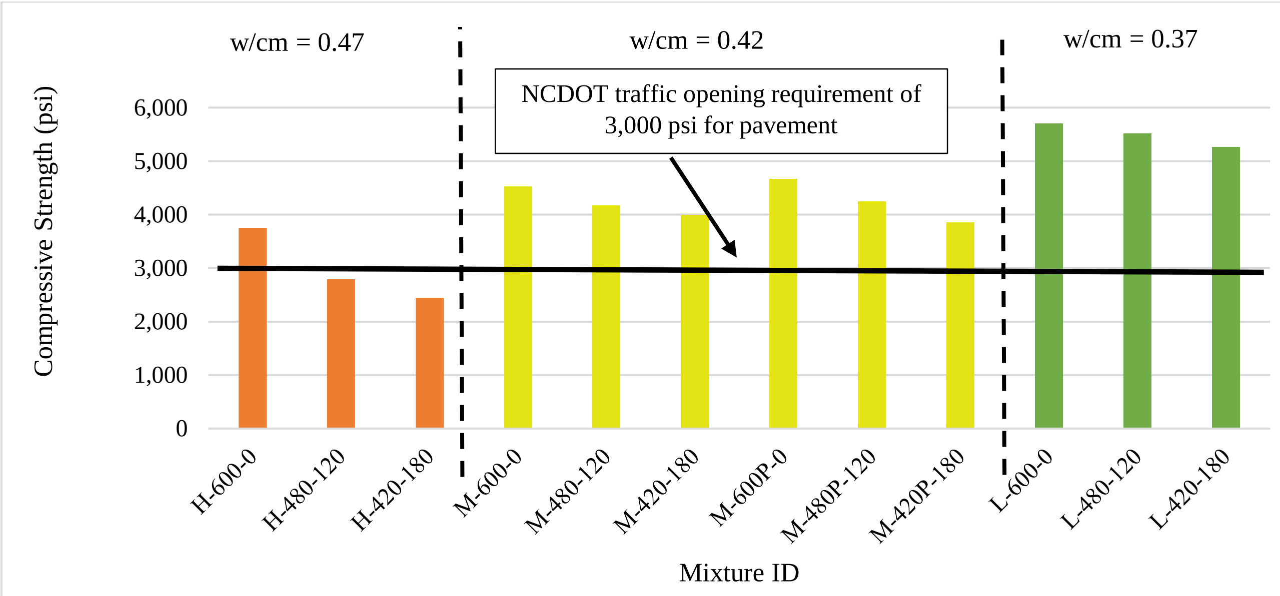

Figure 5.6: 3-day Compressive strength of pavement mixtures

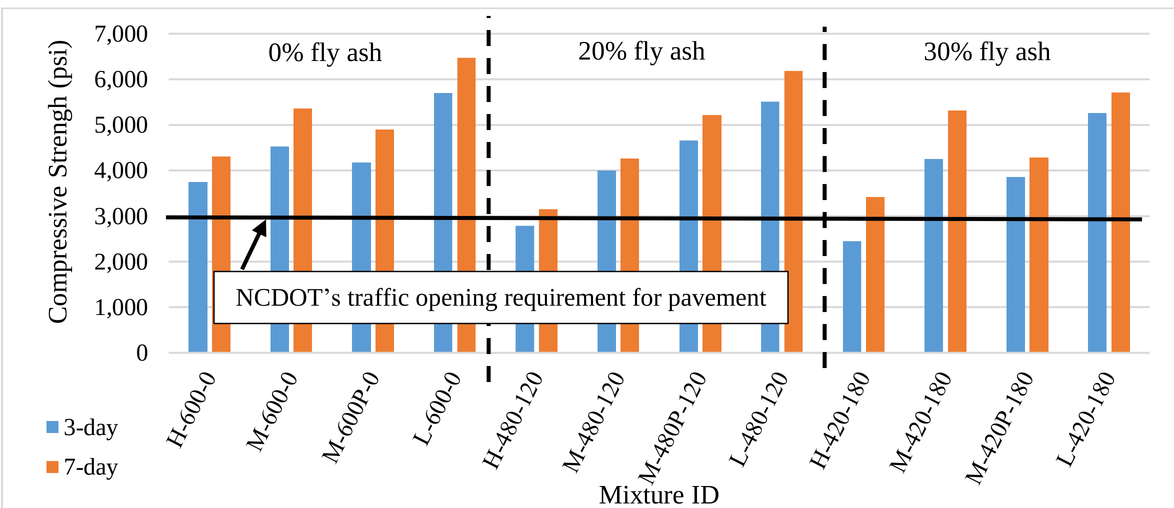

Figure 5.7: 3 and 7-day compressive strength of pavement mixtures sorted by fly ash content

Based upon test results from pavement mixtures batched as part of this project, as well as from RP 2015-03 (Cavalline et al. 2018) and a project for private industry (Ojo 2018), shown in Figure 5.10, the current opening to traffic requirement of 3,000 psi is readily met by most pavement mixtures with reasonable *w/cm* and higher SCM contents. It is noted that these results are under laboratory conditions, and field conditions (particularly temperature) may significantly impact early age strength gain. As described in Section 4.2.2, all mixtures batched and tested for this project readily met the 28-day compressive strength requirement of 4,500 psi and flexural strength requirement of 650 psi for acceptance.

For structural concrete mixtures, beams or girders cannot be placed on concrete substructures until the concrete in the substructure develops a minimum compressive strength of 2,400 psi. Vehicles or construction equipment cannot be placed on a bridge deck until the deck concrete develops the minimum specified 28-day compressive strength. Results from this study (shown in Figure 5.8, sorted by *w/cm*), indicated that for structural mixtures, 2,400 and 4,500 psi are generally achievable by 28 days for all mixtures. As shown in Figure 5.9 and 5.10, low (0.37) *w/cm* structural mixtures readily met the targets by 3 days at each cementitious content and at both 0 and 20% fly ash replacement rates. Mixtures with a moderate (0.42) *w/cm* met the targets at 3 days for straight cement mixtures, but at 7 days for the 20% fly ash mixtures at both cementitous materials contents. At the high (0.47) *w/cm* ratio, all mixtures but one required 28 days to meet the 4,500 psi acceptance target. At this high *w/cm* ratio, the 2,400 psi target for substructure loading was readily met by 3 days for all mixtures. The impact of use of fly ash on strength development, for mixtures of varying cement contents and *w/cm* ratios is clearly evident in Figure 5.11, which includes the control mixtures for RP 2016-06.

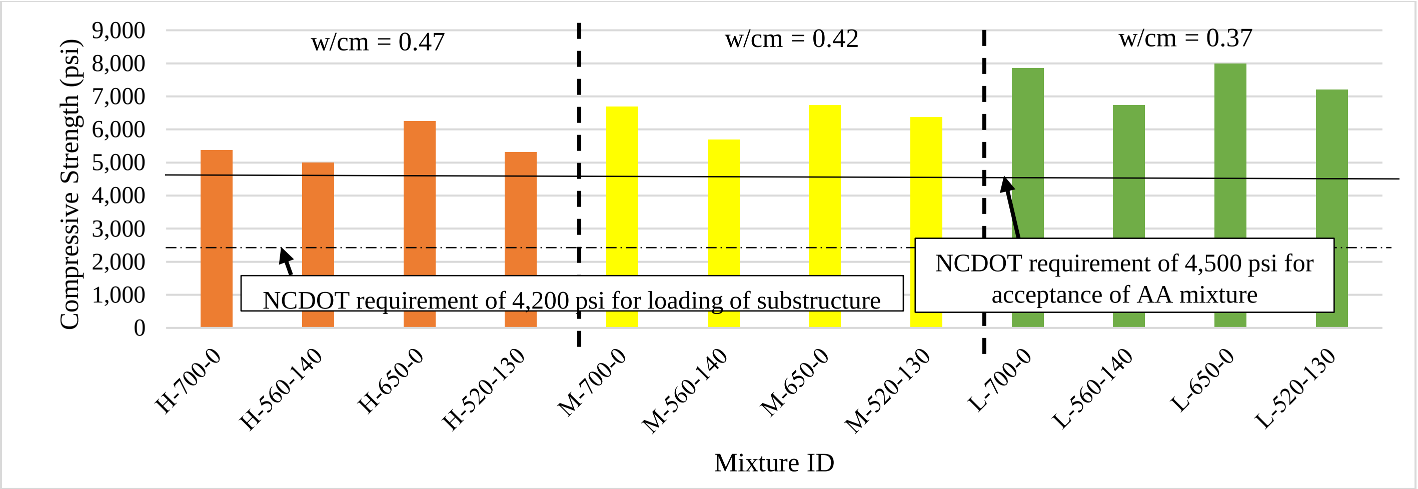

Figure 5.8: 28-day compressive strength test results showing current structural loading and acceptance targets, sorted by *w/cm*

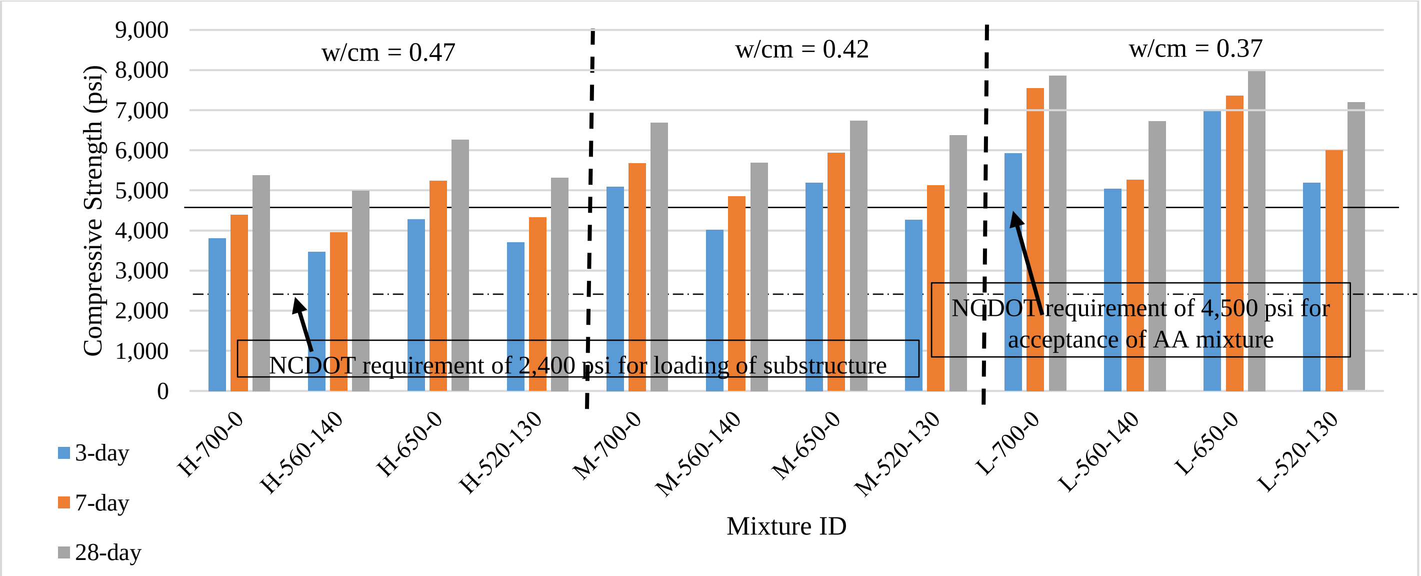

Figure 5.9: 3, 7, and 28 day compressive strength test results showing current structural loading and acceptance targets, sorted by *w/cm*

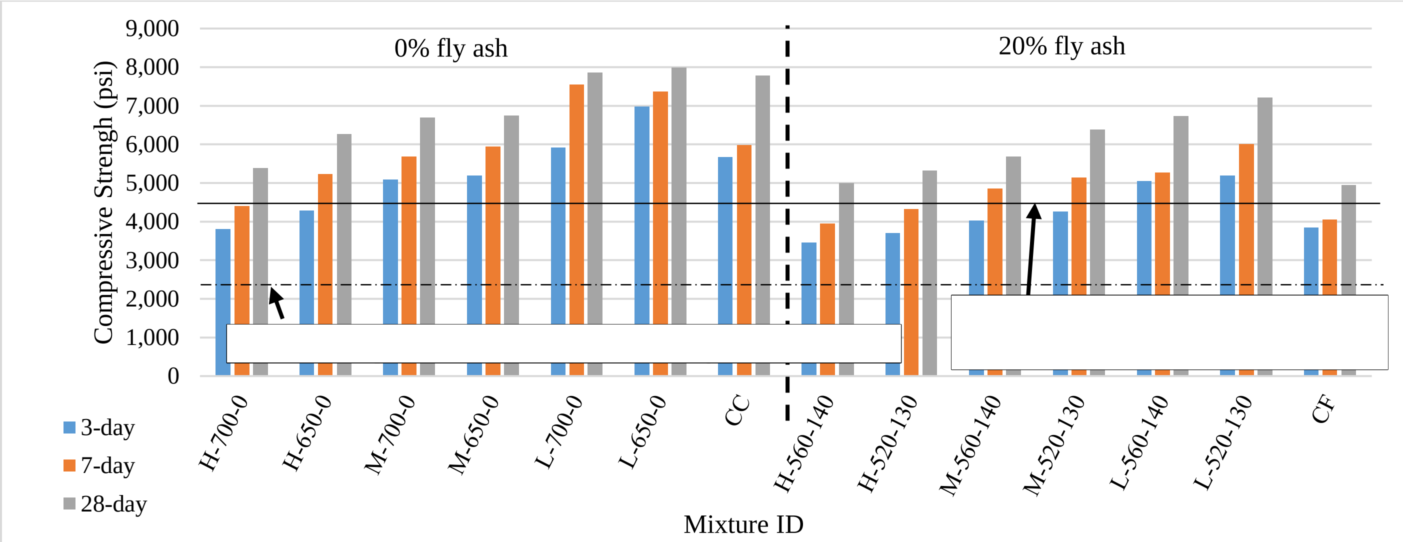

Figure 5.10: 3, 7, and 28 day compressive strength test results showing current structural loading and acceptance targets, sorted by fly ash content

5.2.3 Suggested Shadow Specification for Early Age Opening to Traffic

Analysis of both pavement and structural mixtures indicates that the current targets appear appropriate for most mixtures provided that a reasonable *w/cm* ratio is utilized. Use of fly ash will provide durability benefits, but the delay in strength gain may impact the time required to meet the current targets for opening to construction traffic (for pavements) and loading of substructures and opening of bridge decks to traffic (for structural mixtures).

Review of other state specifications indicates that several states (including Louisiana, Minnesota, New York, Virginia, and West Virginia) open bridge decks to traffic at 4,000 psi. NCDOT could investigate the potential for use of this target at 28 days if desired. Lowering this target could promote additional use of SCMs, providing enhanced durability benefits, provided that this opening strength would not cause excessive risk to damage by traffic. Alternatively by developing two opening-to-traffic strength targets (a specification target for mixtures used for less critical structural components and another target for more critical structural components), contractors may have an increased ability to both optimize concrete mixtures and meet their desired schedule. This approach aligns with NCDOT's use of Class A and Class AA concrete for structural uses. It is noted that verification of early-age compressive strength does not ensure concrete structures will not be damaged from external loads.

5.3 Development of a Specification for Volumetric Shrinkage

Several SHAs have developed and implementing shrinkage specifications. Despite its known shortcomings, volumetric (unrestrained) shrinkage testing using ASTM C157 (AASHTO T 160) remains a test relied upon by a number of SHAs. Other shrinkage tests under evaluation for inclusion in AASHTO PP 84 include several forms of a restrained ring test (AASHTO 2017). However, these tests remain in development at this time and have been removed from recent editions of AASHTO PP 84 (2019). In AASHTO PP 84 (2019), limiting on the volume of paste to no more than 25% is a currently suggested prescriptive specification provision for reducing shrinkage potential for pavement mixtures. Upon review of a historical database of mixture designs submitted to NCDOT for approval, a 25% paste content would be challenging for most local stakeholders to achieve with current mixtures which typically do not include optimized aggregate gradations. Structural mixtures would also not likely be able to meet the 25% paste target due to the required workability and smaller coarse aggregate size. In lieu of specifying a target paste content, AASHTO PP 84 (2019) recommends use of an AASHTO T 160 volumetric shrinkage limit of 420 με.

5.3.1 Analysis of Relevant Requirements

Based upon a review of literature and current agency specifications, AASHTO T 160 appears to provide a reliable preliminary step towards a shrinkage specification to help reduce or prevent cracking. Published guidance regarding volumetric shrinkage targets include 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 potential for cracking (Mokarem et al. 2003). High Performance Concrete (HPC) for structural concrete has also been rated for performance, and shrinkage is included in the criteria. Based on the grading system, the lowest grade is given for concrete exhibiting unrestrained length change of 600-800 με, the middle grade for length change of 400-600 με, and the highest grade for length change less than 400 με (Russell et al. 2006).

To support the development of standard shrinkage specifications that are practical, current SHAs specification provisions for shrinkage targets were reviewed and compared to the AASHTO PP 84 (2019) target of 420 με. Shrinkage requirements for concrete structures and pavements in several state agency specifications are summarized in Table 5.10. It is noted that relatively fewer state specifications on shrinkage were found, and many existing specifications applied to materials used for specialized applications such as repair materials used for dowel bar retrofits. Since these applications are not the focus of this work, this information is not included in Table 5.10.

Table 5.10: Relevant SHA standards on volumetric shrinkage

Agency	Concrete type	Volumetric (unrestrained) shrinkage limit (% length change unless noted)	Curing and/or construction requirements	Age (days)
Louisiana DOTD	Rapid hardening concrete for dowel bar retrofit	<0.013 (130 με)	-	4
	Structural concrete patching material	<0.070 (700 με)	-	28
Minnesota DOT	Type R3 - Rapid hardening concrete for dowel bar retrofit	<0.050 (500 με)	-	28
New York State DOT	High Performance Concrete for precast and pre-stressed bridge beams	<600με	cured for the same time in lab as field	56
	PCC mix for precast repairs	<0.050 (500 με)	cured for the same time in lab as field	56
Florida DOT	Concrete using Petroleum Coke Class F fly ash	-	compare results with ASTM C618 Class F fly ash concrete	28
	Type Q - epoxy compound and repair materials for bridge/pavements	<0.012 (120 με)	water cured and compared to one day length	28
Virginia DOT	Class A4 modified - low shrinkage (bridge deck, overlay)	<0.035 (350 με)	moist cured for 7 days prior to testing	28
West Virginia DOH	Class S-P - self-consolidating and precast concrete	≤0.020 (200 με)	28-day cure per ASTM C157 then Air Storage for 28-days	56

For most of the SHAs, AASHTO T 160 shrinkage testing requirements are established at 28 days with the exception of Louisiana, New York, and West Virginia, which utilize an age of 56 days. New York State DOT provides shrinkage limits for HPC for precast and pre-stressed concrete bridge beams, with the unrestrained shrinkage is limited to 600 με tested at 56 days (NYSDOT 2019). Virginia specifies a volumetric shrinkage requirement of 350 με at 28 days for Class A4 modified low shrinkage concrete to be used for bridge decks and overlays, which is stricter than the AASHTO PP 84 recommended target limit (VDOT 2016). For West Virginia, the class of concrete is a self-consolidating mixture which are normally used for mass concrete structures. This class of concrete is limited to 200 με tested at 56 days, which is the strictest requirement of all shrinkage specifications relevant to concrete pavements and bridges listed in this report (WVDOH 2017).

5.3.2 Development of Performance Targets for Volumetric Shrinkage

The AASHTO PP 84 specification is targeted towards pavement mixtures, although targets could readily be established for bridge (AA) mixtures as part of this work. Most NC mixtures from this project and previous projects performed by this research team (including all mixtures from this study) readily met the AASHTO PP 84 suggested 28-day target of 420 με. In previous research performed for NCDOT for pavement mixtures (RP 2015-03) drying shrinkage test results (performed in accordance with ASTM C 157) indicated that concrete mixtures made with both OPC and PLC readily met the threshold of 400 με (0.04%) at 28 days suggested by Mokarem et al. (2003), with minimal differences in shrinkage observed between the two types of mixtures (Cavalline et al. 2018). Control mixtures with and without fly ash in RP 2016-06 had 28-day volumetric shrinkages values of 318 με and 348 με, respectively, both well under the published guidance.

Figures 4.31 through 4.33 shows the shrinkage for each bridge and pavement mixture in this project's laboratory program. To determine the practicality of selected targets for allowable shrinkage in pavements and structural mixtures such as 400 με and 350 με, further analyses were performed, identifying the percentage of mixtures (and the characteristics of those mixtures) passing and not passing these proposed 28-day shrinkage targets. This is shown in Appendix D, Tables

D.7 to D.9. Findings indicated that an unrestrained shrinkage a target limit of 420 με, as suggested by AASHTO PP 84, could be specified for concrete used in both pavements and bridges. For concrete mixtures susceptible to shrinkage or in applications where shrinkage is highly undesired, a more aggressive target of 350 με could be used. These targets are readily attainable by NC concrete mixtures for both bridges and pavement mixtures exhibiting satisfactory performance in other tests. Both targets should not eliminate concrete mixtures made with SCMs at up to 30% replacement, as evidenced in Figure 5.11, which shows the expanded dataset of 28-day volumetric shrinkage of concrete pavement mixtures from this project and RP 2015-03, sorted by fly ash content. As can be seen in Figure 5.11, only one mixture P.BL.N.M from RP 2015-03, exceeded the recommended limit at 423 με. This mixture had a *w/cm* ratio of 0.48. Other mixtures from this project with high *w/cm* (0.47) readily met the 420 με target.

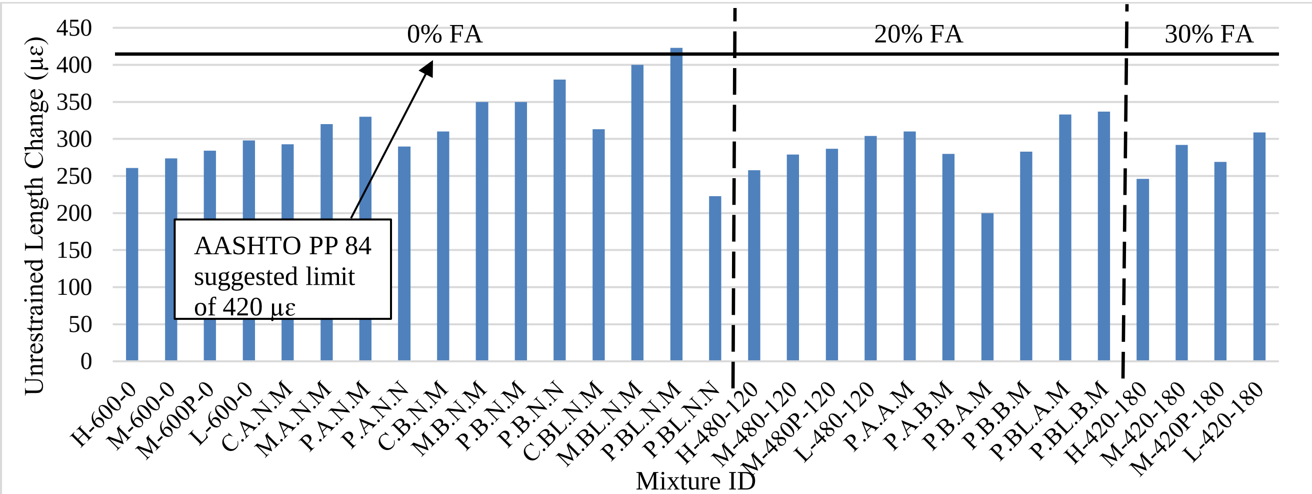

Figure 5.11: 28-day Average volumetric shrinkage for pavement mixtures from RP 2015-03 and this project, sorted by fly ash content

Although unrestrained shrinkage testing was conducted at 4 weeks, 8 weeks, 16 weeks, 32 and 64 weeks, the test age of 28 days is recommended by AASHTO and several SHAs as a target age for testing due to convenience. To encourage use of higher SCM mixtures, a 56-day target could also be utilized. As this specification is implemented, target(s) could be adjusted depending on contractor and producer ability to test and limit shrinkage. More aggressive targets could ensure NCDOT concrete pavement and bridge mixtures are durable, by keeping unrestrained shrinkage to a minimum.

5.3.3 Suggested Shadow Specification for Volumetric Shrinkage

To add a new specification provision for volumetric shrinkage, AASHTO T 160 should be added to the table of prescribed laboratory tests to be submitted along with Materials and Tests Unit Form 312U as directed in Section 1000-4 “Portland Cement Concrete for Structures and Incidental Construction” in the NCDOT Standard Specifications (NCDOT 2018). In Section 1000-4, the table under (A) Composition and Design, the table of laboratory tests to be performed on mixtures for approval could be amended to that shown in Table 5.11.

Table 5.11: Table of laboratory tests to be submitted with Form 312U for mixture approval

Property	Test Method
Aggregate Gradation	AASHTO T 27
Air Content	AASHTO T 152
Slump	AASHTO T 119
Compressive Strength	AASHTO T 22 and T 23
Shrinkage	AASHTO T 160

Additional information could be provided in a new Section (E) Shrinkage requirements, as shown below, or added to Project Special Provisions for use at acceptance.

(E) Shrinkage Requirements

Concrete should be tested for unrestrained length change at 28 days using AASHTO T 160. For typical concrete pavement and bridge applications, the length change is limited to 420 με. For concrete applications where enhanced provisions against cracking are desired, length change can be limited to 350 με at the engineer's discretion. (Table 5.12 should be added or incorporated into Table 1000-1 with the following note)

Table 5.12: Suggested addition to NCDOT Specifications for Roads and Structures Table 1000-1, or use as additional requirement

Class of Concrete	Shrinkage Limit (με) at 28 days
AA	420*
Pavement	420*
*For concrete where a reduction in cracking due to shrinkage is desirable, 350με could be used.	

6. SUMMARY AND CONCLUSIONS

A data analysis of NC concrete mixtures was performed, with a goal of linking mixture characteristics and early age test results to field performance. Key findings included:

Correlations between Mix Design and Early Age Characteristics for Deck Mixtures

- Review of early age test data confirmed that most of the accepted and placed mixtures comply with the standards and specifications. Notable exceptions to this finding include air content in Class AAA mixtures and slump in paving mixtures. However, for both of these classes of concrete, the team was unable to link many records to results, and so the number of failed examples is small.
- The design slump value has a positive relationship with the difference between the design and actual air content values. The increase in fine aggregate amount has a positive relationship with the difference between design and actual air content. Since well-rounded particles in the fine aggregate can lead to higher air entrainment, increasing the amount of fine aggregate should positively increase the amount of air in the concrete.
- The cement content of mixtures has a negative relationship with the difference between design and actual slump, meaning that adding more cement to the mixture tends to decreases the slump. The *w/cm* ratio has positive relationship with the difference between design and actual slump amount, meaning that as the ratio is increased (either by water content increasing or cement value decreasing), the mixture becomes more workable and the measured slump increases.
- The amount of fine aggregate in the mixture design has a strong positive correlation to a difference between the specified and actual compressive strength. This indicates that increasing the amount of fines in a mixture tends to increase the compressive strength, in many cases above the design compressive strength. Increasing the total volume of fines tends to lead to higher increases in compressive strength.
- Paste content has a negative relationship with the difference in specified and actual compressive strength, so increasing the paste content increases the chance that the compressive strength will not be as high. Paste content is a combination of cement content (positive), pozzolan amount (negative), water amount (negative), and latex (positive). The two positives, cement content and latex, are either very lowly correlated or (in the case of latex) not present in many of the mixtures, which is why the negatives control this variable. The coarse aggregate amount has a negative relationship with the difference between design and actual compressive strength, which is logical because too many large particles can increase the likelihood of void spaces, as well as decrease the space available for the binding paste.

Correlations between Mix Design and Early Age Characteristics for Pavement Mixtures

- The *w/cm* ratio is the most strongly correlated variable, with a positive relationship to the difference in specified and measured flexural strength. This finding is supported by the fact that the variable, "Water amount," is also positively correlated, as an increase in the *w/cm* ratio indicates either an increase in water amount or a decrease in cement content.

- For North Carolina concretes, only Class C Fly Ash and Class F Fly Ash are used. The correlation results, as well as the negative regression sign, indicate that for mixtures in the state, increasing the pozzolan amount leads to lowered (but not necessarily lower than required) flexural strength. Given that pozzolans hydrate more slowly than cement, and the test date is not currently adjusted for fly ash mixtures, this finding could be expected.
- Fine aggregate amount has a positive correlation with flexural strength, indicating that increasing the fine aggregate content typically increases the strength above the design value. This, combined with the fine aggregate SG (positively correlated at #9 in the list, and also included in the regression equation) indicate that for NC mixtures, increasing the fine aggregate content as well as ensuring good gradation increases the concrete's flexural strength.
- Fine aggregate SG has a negative relationship with the difference between actual and required compressive strength. A negative correlation here means that, while the compressive strength can still be larger than the design value (as it is in most cases), use of higher SG fine aggregate leads to an overall lowered compressive strength. The SG of manufactured sand is higher than that of natural sand (Megashree et al. 2016), so the results indicate that the use of manufactured sand leads to a lower difference in compressive strength.
- Pozzolan amount has a positive correlation to the difference between design and actual compressive strength. Fly ash is the only pozzolan present in the pavement mixture designs covered by this dataset. As with much available research on fly ash (Harison et al. 2014) in concrete, NCDOT pavements that included it showed increases in the later-age strength of concrete.

<u>Correlation between Early Age Characteristics and Long Term Bridge Deck Performance</u>

- The amount of water present in the mixture was found to be the most important variable between bridges that were under-performing versus over-performing. The difference in water amount between the under and over performing groups is strongly linked to several other important factors, such as paste content, the *w/cm* ratio and fly ash content (all of which are significant at the 90% confidence level, and cause an increase in water content).
- The mean amount of fly ash in mixture designs is greater for the over performing bridges. This indicates that increasing the fly ash amount does help improve durability of concrete. The mean difference between the amount of Class F fly ash used increased from under to over performing bridge decks (from 162.6 lbs to 168.8 lbs), and this difference is significant at the 90% confidence level. 25% of the mixtures in the under-performing category are below 163 lbs. As mentioned previously, at least 75% of the mixtures use fly ash, and since it has been shown that use of Class F fly ash above the recommended minimum, as well as an increase between the means from under to over performing bridges, continuing to add at least 35% replacement of cement with SCM's is recommended.
- The SG of the coarse aggregate is a significant variable at the 90% confidence level. The difference in the mean of the SG is not large (only a decrease of 0.04 from underperforming to overperforming).
- The mean value of the paste content for the over performing bridges was higher than for the underperforming bridges, and the mode value is also much higher. Although this somewhat contradicts guidance to reduce paste content, it is likely that the workability benefit of increased paste were more influential.

<u>Laboratory test results</u> for the matrix of 24 mixtures batched and tested as part of this project were analyzed. Key findings from the laboratory testing included:

<u>Fresh Properties</u>

- Fresh properties and admixture dosages for PLC mixtures did not differ significantly from those found for the companion OPC mixtures. This is consistent with previous studies for NCDOT (Cavalline et al. 2018, 2019), and supports the use of PLC to improve the durability and sustainability of NCDOT's concrete infrastructure.

<u>Mechanical Properties</u>

- The generally accepted trends associated with *w/cm* were observed (e.g., mixtures with lower *w/cm* typically outperformed the companion 0.42 and 0.47 w/cm mixtures in both compressive strength and MOE testing).
- NCDOT's decision to allow increased fly ash replacement rates (transitioning from 20% to 30% by weight of cement) should have minimal impact on long-term strength. The difference in compressive strengths over all test dates was relatively small, with 20% fly ash mixtures averaging an 8.0% higher compressive strength than companion straight cement mixtures at later ages.
- The test results MOE and MOR were even closer, with 20% fly ash mixtures having MOE results 1.0% higher than non-fly ash mixtures on average, and MOR results 3.0% higher than non-fly ash mixtures on average.

- Measured MOE values were significantly lower (21.9%) than MOE values predicted using the ACI 318 equation. This finding is similar to that of previous studies for NCDOT performed by this research team, which may be of interest to designers.
- Significant differences in mechanical properties were not observed between PLC and OPC companion mixtures. PLC mixtures averaged higher compressive strength results at 7, 28, and 56 days by an average of 5.0%. OPC mixtures had slightly higher test results for 90-day compressive strength, MOR, and MOE, with average results 2.0%, 3.0%, and 3.0% higher, respectively. These findings again support use of PLC in future NC infrastructure.

Durability Performance

- Surface resistivity values are influenced by mixture characteristics and proportions.
  - Fly ash mixtures typically outperformed non-fly ash mixtures at later ages, with 56-day resistivity results higher than all non-fly ash companion mixtures.
  - Although it is known that mixtures with lower *w/cm* ratios typically provide improved (higher) surface resistivity test results, the difference for mixtures as part of this laboratory testing program did not provide trends as strong as observed in previous studies.
  - The influence of total cementitious material content on resistivity can be seen with greater improvements between 28- and 90-day tests for lower total cementitious material contents, especially for those with fly ash.
  - Resistivity results for PLC mixtures improved with use of fly ash, specifically the higher (30%) replacement rate. Results for the non-fly ash mixtures showed the OPC mixture surpassing the values for the PLC mixture by 90 days, and results for the 20% fly ash mixture showed minimal difference from the OPC and PLC mixtures. At a 30% fly ash replacement rate, a promising trend was observed. At 28 days, average resistivity values were nearly identical. However, at 56 days the PLC mixture showed an advantage over the OPC mixture, and further outperformed it at 90 days, which supports use of ternary (PLC + fly ash) blends for durable concrete.
- Bulk resistivity test results typically improved with increasing fly ash replacements, particularly when comparing 28- and 90-day values. At 28 days, lower *w/cm* mixtures performed better than higher *w/cm* companion mixtures. Bulk resistivity values for PLC mixtures were comparable to OPC companion mixtures.
- Similar to other electrical tests to measure permeability, RCPT results benefitted from fly ash replacement. The 0.37 *w/cm* mixtures typically had RCPT values lower than higher *w/cm* companion mixtures. The 600 pcy cementitious material mixtures typically outperformed 650 and 700 pcy mixtures, with the most noticeable difference evident for the 0.37 *w/cm* mixtures. At 28 days, this trend is seen for all mixtures except for 0.47 *w/cm* and 700 pcy mixtures and one 650 pcy mixture for the 0.42 and 0.37 *w/cm* mixtures. Differences between results for PLC and OPC mixtures were minimal, with the 600 pcy mixtures nearly identical, and the PLC mixture having better performances at both 28 and 90 days.
- Preliminary formation factor test results show trends similar to other electrical resistivity tests. Mixtures with a fly ash replacement showed a performance advantage, particularly at later ages, when compared to non-fly ash mixtures. It should be noted that the testing and calculation method for formation factor is still being revised and improved at this time. Therefore, values published in this report are relevant only for preliminary observations.
- All mixtures tested exhibited volumetric shrinkage test results well below the 28-day AASHTO PP 84-19 suggested shrinkage limit of 420 $\mu\varepsilon$. Fly ash mixtures tended to be more resistant to shrinkage than the non-fly ash mixtures, with mixtures with 30% fly ash replacement rates showing reduced shrinkage by 28 days in comparison to several of the 20% fly ash replacement rate mixtures, as expected.

Analysis of relevant requirements and potential performance targets was performed to support development of a <u>shadow specification for surface resistivity</u>. Key findings included:

- Laboratory test results indicated that NCDOT concrete mixtures may not meet some of the more aggressive RCPT and surface resistivity targets utilized by other states at early ages. At later ages, such as 56 days, some other state specification targets are achievable by typical NC mixtures, particularly by fly ash mixtures.
- To produce a preliminary specification for possible implementation by the NCDOT, field performance verified RCPT targets from a state with similarities to NC (Virginia) were evaluated against the resistivity test results from NC Concrete mixtures to identify potential resistivity targets.
- For structural mixtures, a surface resistivity target of 15.0 kΩ-cm or 16.0 kΩ-cm (corresponding to RCPT values of 2,800 and 2,700 coulombs respectively) are recommended. Both targets appear to reasonably discern between

mixtures with higher and lower durability performance potential, with the target of 16.0 kΩ-cm providing an aggressive, but realistically feasible performance target for structural mixtures.

- In the existing coastal corrosive zones, a higher resistivity target could be warranted. Findings from RP 2019-22, a study of NCDOT's corrosive sites policy should help identify target values for these critical zones.
- For pavement mixtures, a target surface resistivity value of 11.0 kΩ-cm at 56 days was identified, roughly corresponding to an RCPT of approximately 3,300 coulombs.
- To ease implementation, the suggested method of specification for surface resistivity was integrated into NCDOT's current compressive strength testing specifications.

Analysis of relevant requirements and potential performance targets was performed to support development of a shadow specification for early age opening to traffic. Key findings included:

- Analysis of compressive and flexural strength data both pavement structural mixtures indicates that the current targets appear appropriate for most mixtures provided that a reasonable *w/cm* ratio is utilized. Use of fly ash will provide durability benefits, but the delay in strength gain may impact the time required to meet the current targets for opening to construction traffic (for pavements) and loading of substructures and opening of bridge decks to traffic (for structural mixtures). Specifications could be modified slightly to ensure SCM mixtures are readily considered for use by contractors.
- Review of other state specifications indicates that several states open bridge decks to traffic at 4,000 psi. NCDOT could investigate the potential for use of this target at 28 days if desired. Lowering this target could promote additional use of SCMs, providing enhanced durability benefits.

Analysis of relevant requirements and potential performance targets was performed to support development of a shadow specification for volumetric shrinkage. Key findings included:

- Test results from this project and previous projects for NCDOT indicate that the shrinkage a target limit of 420 $\mu\varepsilon$ suggested by AASHTO PP 84 could be used for concrete pavements and bridges. However, an unrestrained shrinkage limit of 350 $\mu\varepsilon$ may be a more appropriate and readily achievable target for NC concrete mixtures, particularly where shrinkage is highly undesired.
- Both targets are attainable for structural and pavement mixtures and should not eliminate concrete mixtures made with SCMs. An age of 28 days is recommended by AASHTO PP 84 for this test, and several SHAs utilize this age likely due to convenience. To encourage use of higher SCM mixtures, a 56-day target could also be utilized.
- As this specification is implemented, target(s) could be adjusted depending on contractor and producer ability to test and limit shrinkage. More aggressive targets could ensure NCDOT concrete pavement and bridge mixtures are durable, by reducing further unrestrained shrinkage.

7. ROADMAP TOWARDS A PERFORMANCE ENGINEERED CONCRETE SPECIFICATION

In addition to the initial steps to move towards performance engineered concrete mixtures made as part of this project (NCDOT RP 2018-14), NCDOT has sponsored Research Project 2020-13, "Continuing Towards Durable and Sustainable Concrete Through Performance-Engineered Concrete mixtures. Together, these research studies aim to 1) identify trends in currently used mixtures and link to field performance, 2) perform targeted laboratory testing to establish performance-related criteria for several emerging PEM test technologies, 3) provide insights into performance of concrete mixtures utilizing sustainable materials (fly ash and portland limestone cement) and 4) provide and update a "roadmap" of recommendations and guide specifications for additional work and pilot projects for performance engineered concrete.

Concurrently, NCDOT applied for additional funds to support PEM implementation as part of FHWA's Demonstration Project for Implementation of Performance Engineered Mixtures/AASHTO PP 84." NCDOT applied for in funds in three of four categories:

Category A: incorporating two or more AASHTO PP 84-17 tests in the mix design/approval process. Shadow testing is acceptable.

Category B: for incorporating one or more AASHTO PP 84-17 test in the acceptance process. Shadow testing is acceptable.

Category D: for requiring the use of control charts, as called for in AASHTO PP 84-17.

Implementation funds were awarded to NCDOT in Spring 2018, and some funds were used internally to support equipment purchases and other NCDOT PEM implementation efforts. Lane Construction partnered with NCDOT and UNC Charlotte in supporting a PEM demonstration project as part of the FHWA Implementation Funds program. The PEM demonstration project was a design-build urban interstate project that included the widening of 5.3 miles of I-85 in Rowan County, NC (TIP Project I-3802B). Testing technologies utilized at the implementation site included the Box Test, SAM, and surface resistivity. The FHWA's Mobile Concrete Technology Center (MCTC) visited the site in late April / early May 2019 during Phase 2 paving. During the two-week visit, MCTC personnel sampled and performed shadow tests on concrete from the PEM Demonstration Project and worked with Lane Construction personnel to understand their practices and processes. The visit culminated in an Open House event, coordinated by the Carolinas Concrete Paving Association (CCPA) and the National Concrete Pavement Technology Center. During this visit, various PEM technologies were demonstrated to the NCDOT and industry personnel, as well as QA/QC practices promoted with the PEM initiative. A summary presentation of the MCTC's findings was presented at the Open House (Gudimettla 2019), and a full report regarding FHWA MCTC visit and test results will be published by the MCTC staff in the near future. Overall, the PEM demonstration project was a success, and the PEM implementation funds were utilized in a manner consistent with the requirements of the application. The contractor and NCDOT personnel gained valuable experience with three PEM testing devices and tests during the course of the demonstration project. A detailed report of the PEM Demonstration Project is presented in the report for NCDOT RP 2019-41 (Cavalline et al. 2020).

As part of ongoing NCDOT RP 2020-13, UNC Charlotte is studying of the effects of optimized aggregate gradations on fresh and hardened concrete test results. The mixture matrix for RP 2020-13 "mirrors" that of this project (RP 2018-14), with the same cementitious materials content and *w/cm* levels, but with optimized graded aggregate systems. Findings of RP 2020-13 should support development of recommendations to help NCDOT encourage stakeholders to reduce paste content (and as such, cement content) in mixtures. Findings will also provide guidance on reducing the prescriptive *w/cm* specification provisions and increase the understanding of emerging testing technologies and preliminary specification targets to support durable, sustainable infrastructure. Upon completion of testing for RP 2020-13, the research team plans to perform analyses to quantify the cost savings and greenhouse gas reductions that could be achieved by reducing the cement content in optimized gradation concrete mixtures.

NCDOT's PEM efforts to date are providing confidence in emerging test technologies and performance targets for use in future QA/QC. In addition to the recommended specifications and performance targets for surface resistivity, shrinkage, and early age strength presented herein, findings from RP 2020-13, supported with data from previous projects, will allow development of a proposed specification and performance target for the Super Air Meter (SAM). As part of RP 2020-13, UNC Charlotte is investigating the potential utility of the surface resistivity meter as a QA/QC tool for bridge deck overlays. Use of each of these approaches in a more rigorous quality system should improve construction practices and allow NCDOT to accept higher quality infrastructure, ultimately resulting in reduced maintenance costs over the lifetime of service (Gross et al. 2017). Ultimately, target values for resistivity, shrinkage, and other tests should be compared with field performance of NC structures and pavements to facilitate assessment of the targets and further refinement of the specification.

In the future, it will be necessary to adapt materials specifications to embrace new technologies and alternative materials. Pressure to adopt new materials is already felt due to shortages of quality fly ash as reduced coal combustion results in a diminished supply, and as new products are brought to market and made available to contractors. For instance, the mixture matrix for RP 2020-13 also includes batching and testing of several mixtures using carbon nanotubes, an emerging technology showing potential durability benefits. Laboratory and field data from PEM "tests that matter" provide an excellent basis for NCDOT to benchmark the performance of innovative and emerging materials (such as carbon nanotubes) against traditionally produced mixtures and mixtures optimized using locally available conventional materials. This approach should provide NCDOT a means to justify the potential initial cost burden of novel materials, or (in lieu of use of novel materials) support the decision to utilize more conventional approaches to achieve similar performance at a potentially reduced cost.

Although there was a large sample size of concrete mixtures used for the development of target values for each specification, the volume of concrete produced daily for construction supporting NCDOT infrastructure is magnitudes larger than the collective amount of mixtures proportions and materials used for this research. If additional external stakeholders such as contractors and ready-mixed concrete suppliers become engaged in the PEM effort, test results from field-produced concrete could be used in conjunction with laboratory test results and field performance data for further refinement of specification targets. Additional engagement of stakeholders would also open the door to exploration of ways to improve durability performance for their mixture designs through mixture proportioning, use of other SCMs, and enhanced testing and specification methods per AASHTO PP 84.

Movement towards implementation of performance-related specifications will require stakeholder education and buy-in. Significant progress was made on this front as part of RP 2019-41, with the willing engagement of a contractor partnering to utilize PEM tools as part of the first NC PEM pilot study and a diverse audience of stakeholders in attendance at the MCTC Open House. This contractor has verbally indicated they plan to utilize AASHTO PP 84 guidance as part of an upcoming paving project in Fall 2020. Additionally, RP 2020-13 includes two pilot projects that will expand PEM field implementation to bridge applications. The pilot projects will include: 1) construction of a new bridge (substructure, superstructure, and deck if possible) and 2) construction of a concrete overlay. Specification provisions and performance targets for resistivity, early age strength, and shrinkage developed as part of RP 2014-18 (this project) will support provisional specifications for the proposed pilot projects, along with previous work using the SAM. Engagement of additional stakeholders, including both contractors and ready-mixed concrete suppliers, will be essential to the success of the PEM initiative in NC and other states. Training materials prepared by UNC Charlotte for NCDOT divisional/regional personnel as part of RP 2019-41 should support this need, particularly if made available to industry stakeholders.

In summary, moving forward, NCDOT should consider:

- Use of recommended specification provisions for surface resistivity, shrinkage, and/or early age opening to traffic as shadow specifications or project special provisions in upcoming projects.
- Means to increase stakeholder awareness of the PEM initiative, potentially at the upcoming North Carolina Concrete Pavements Conference or other events. Stakeholders should be encouraged to use PEM guidance and tests.
- Designing and providing incentives to encourage more contractors to utilize PEM methods and testing provisions through shadow specifications. This will increase contractors' comfort with the PEM tools and technologies, as well as increase the field data available to further develop performance targets.
- Development of specification provisions and performance targets for additional PEM testing methods, including:
    - Formation factor testing to assist with optimizing electrical tests to accommodate more sophisticated cementitious systems such as ternary blends and a range of SCMs including metakaolin, slag, and other materials.
    - Workability tests such as the VKelly and Box Tests. Ensuring better workability should allow concrete structures and pavements to be more constructible and offer improved performance over longer lifespans. The contractor at the initial PEM pilot study utilizes the Box Test and has found it highly useful, running it each time a change is made to the mixture to ensure adequate workability for placement.
- Reduction of current prescriptive *w/cm* ratio limits and cementitious materials contents. Some guidance is presented as part of this study, but additional data anticipated from findings of RP 2020-13 should refine UNC Charlotte's recommendations.
- Means to introduce pay incentives for meeting performance targets for performance-related specification provisions.

8. VALUE OF RESEARCH FINDINGS and RECOMMENDATIONS

8.1 Value of Research Findings

Research products produced from this work included:

- Analysis of the characteristics of currently utilized and historically utilized concrete mixtures, along with trends linked to good, acceptable, and poor performance.
- Additional data to support NCDOT in decisions regarding use of PLC and higher (30%) fly ash contents in NC concrete, as well as test data on typical conventional highway concrete mixtures to support movement towards performance engineered specifications.
- Suggested performance targets and proposed specification provisions for three targeted PEM provisions (surface resistivity, shrinkage, and early age opening to traffic) and some guidance to assist with prescriptive specification measures such as *w/cm* ratio and paste content.
- Specification provisions use by NCDOT on upcoming PEM pilot projects or in other trial situations supporting use of performance engineered concrete mixtures.
- A "roadmap" outlining suggested future work for NCDOT to move towards a specification for performance-engineered concrete mixtures.

At this time, quantification of the value of this research is challenging. Ultimately, findings of this work should support more durable, economical, and sustainable concrete highway infrastructure. Increased use of SCMs and PLC should allow for lower cementitious materials contents to achieve the same durability performance. This would result in initial cost savings as well as cost savings over the life cycle of infrastructure components, which should achieve longer service lives and will require reduced maintenance and rehabilitation actions. Increased use of SCMs and PLC, along with lower cement contents will also result in a reduction in greenhouse gas emissions, allowing NCDOT to demonstrate progress towards MAP-21 goals. As mentioned in Section 7.0 of this report, upon completion of testing for RP 2020-13, the research team plans to perform analyses to quantify the cost savings and greenhouse gas reductions that could be achieved by reducing the cement content in optimized gradation concrete mixtures.

The true measure of the economic benefits of movement towards use of the proposed specification provisions, along with increased use of sustainable materials such as PLC and higher SCM contents will become evident only after infrastructure components are constructed in this manner, and then the life cycle costs compared to similar components constructed without use of PLC/higher SCM contents and use of PEM technologies. In the future, several methods for quantification of the value of this research could include:

- A life cycle cost analysis (LCCA) on a roadway or structure constructed using the provisions recommended in this report, comparing it to a previously constructed roadway or structure. An LCCA would provide a measure of the economic (cost) savings associated with findings and recommendations presented herein. The cost savings could be computed on a materials-alone basis (cost per cubic yard of as-constructed concrete), on an annual basis, or on a percent cost savings per lane-mile basis. A history of maintenance actions (or a reliable estimate of anticipated maintenance actions) and expected service life would be needed to complete this analysis. Ongoing work to identify or predict these inputs to the LCCA is being performed by many researchers and could be supplemented by data obtained from an increasing number of pilot project studies planned and ongoing.

- A life cycle assessment (LCA) on a roadway or structure constructed using the provisions recommended in this report, comparing it to a previously constructed roadway or structure. FHWA's recently published "Pavement Life-Cycle Assessment Framework" should be used (Harvey et al. 2016). An LCA would provide a measure of the sustainability benefits, including economic (cost) savings, reduced environmental impact, and reduced societal impacts associated with findings and recommendations presented herein. Many inputs required for the LCA could likely be reasonably assumed using previous research. However, limitations would still include information on required maintenance actions and expected service life as described above.

- 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 kilograms (kg) or pounds (lb) of emissions) could be computed for a roadway or structure. 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.

8.2 Recommendations

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

- Increased use of SCMs such as fly ash, at higher replacement rates, should provide improved durability performance and hence, long-term economic and environmental benefits. It is recommended that NCDOT encourage increased use of SCMs such as fly ash on their concrete infrastructure projects.

- Use of fly ash and some other SCMs, particularly at higher replacement levels, can sometimes impact the ability of concrete mixtures to meet existing early age strength targets for opening to construction vehicles and other traffic. It is recommended that NCDOT make efforts to avoid discouraging use of SCM mixtures on the basis of early strength requirements. Specification guidance on early age strength gain presented in this report could assist NCDOT in accommodating these types of mixtures.

- Use of PLC should provide equivalent performance to OPC in NC concrete mixtures, while providing sustainability benefits associated with reduced emissions. These benefits can be enhanced further if fly ash is used with PLC. NCDOT should encourage use of PLC in NC infrastructure. NCDOT should specify inclusion of SCMs if PLCs are utilized in high sulfate environments.

- Additional work to identify ways to reduce the paste content and total cementitious content of NC concrete mixtures is warranted. Research to investigate the benefits associated with use of optimized aggregate gradations is ongoing as part of NCDOT RP 2020-13.

- Surface resistivity provides a promising technology for rapid evaluation of durability performance. The test is easy to perform, utilizes specimen already cast for compressive strength tests, and has been reasonably linked to long-term performance of existing infrastructure via the RCPT and field experience of other states currently utilizing resistivity in their specifications. Surface resistivity testing should be promoted by NCDOT, and it is recommended that NCDOT move towards adoption a resistivity specification to support more durable, sustainable concrete infrastructure.

- Specifying volumetric shrinkage testing would provide a means for NCDOT to promote construction of less permeable infrastructure that is less prone to cracking. The shrinkage specification target presented in this work could be utilized in pilot studies to explore the benefits of such specification provisions in promoting enhanced performance.

- Gaining stakeholder buy-in is critical to ensure success of the PEM initiative. Initial steps to engage contractors in PEM work has been initiated for concrete pavement projects as part of the FWHA PEM Implementation Funds program (RP 2019-41, Cavalline et al. 2020), and is continuing with two additional pilot projects for bridge construction included in RP 2020-13. Technology transfer activities, including seminars for division and regional personnel are also ongoing as part of RP 2019-41. These types of activities that support stakeholder education, training, and use of the PEM technologies should be continued. Presentations at the North Carolina Concrete Pavements Conference (as well as similar meetings) could help engage local/regional industry in the PEM effort.

- Efforts to grow and improve FHWA's PEM initiative are ongoing in many areas, including research, pilot projects, and development of technology transfer tools. NCDOT should continue to stay engaged in this initiative through the Pooled Fund Studies, the National Concrete Consortium, and other avenues that may emerge.

9. IMPLEMENTATION AND TECHNOLOGY TRANSFER PLAN

RP 2019-41 provided several opportunities for technology transfer supporting the PEM initiative, including training and use of PEM technologies by a contractor, an open house and demonstration by the by the FHWA's MCTC which was attended by a range of contractors, engineers, and NCDOT personnel. Technology transfer of this work is also ongoing as part of a series of training and seminars being prepared by UNC Charlotte personnel as part of RP 2019-41. Additional, specific technology transfer actions for the products of this research project are listed below.

Research Product 1	Digital database of test results from laboratory testing
Suggested User	Materials & Tests Unit, Pavement Design & Collection Unit, Structures Management 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 2	Laboratory test data indicating that PLC should perform similarly to OPC in NC concrete mixtures
Suggested User	Materials & Tests Unit, Pavement Design & Collection Unit, Structures Management 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 NC market.
Recommended Training	None recommended at this time.

Research Product 3	Laboratory test data confirming the durability benefits of use of fly ash in concrete mixtures.
Suggested User	Materials & Tests Unit, Pavement Design & Collection Unit, Structures Management Unit
Recommended Use	This information could be utilized to support decisions to specify that fly ash be incorporated in concrete for certain projects, or incorporated at higher replacement rates.
Recommended Training	None recommended at this time.

Research Product 4	Recommended specification for surface resistivity
Suggested User	Pavement Design & Collection Unit, Materials & Tests Unit, Structures Management Unit
Recommended Use	The recommended specification could be used as a shadow specification for upcoming PEM pilot projects for RP 2020-13, or on other projects as desired by NCDOT.
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 6	Recommended specification for early age strength for opening to traffic
Suggested User	Materials & Tests Unit, Pavement Design & Collection Unit, Structures Management Unit
Recommended Use	The recommended specification (particularly a lower opening to traffic strength or later age for the currently utilized strength) could be used as a shadow specification for upcoming PEM pilot projects for RP 2020-13, or on other projects as desired by NCDOT.
Recommended Training	None recommended at this time.

Research Product 7	Recommended specification for volumetric shrinkage
Suggested User	Materials & Tests Unit, Pavement Design & Collection Unit, Structures Management Unit
Recommended Use	The recommended specification could be used as a shadow specification for upcoming PEM pilot projects for RP 2020-13, or on other projects as desired by NCDOT.
Recommended Training	This test is outlined in ASTM C157. UNC Charlotte personnel could meet with Materials & Tests Unit personnel to assist in training, if requested.

10. 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
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Durable and Sustainable Concrete Through Performance Engineered Concrete Mixtures

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APPENDIX A – LITERATURE REVIEW AND REFERENCES

A.1 Motivation for Study and Research Needs

As demand for new infrastructure increases, along with an increased burden of maintenance and rehabilitation of existing infrastructure, resources required to support this work are becoming more limited. To address this need, researchers and industry leaders are increasingly investigating and developing sustainable solutions in a variety of areas, including construction materials and practices. Sustainability demands and resource scarcity are the root cause of the development of the Performance Engineered Mixtures (PEM) initiative, in which mixtures are designed to control and optimize the resources used in the process of making concrete (Ahlstrom and Richter, 2018). By engaging engineering approaches during initial mixture development and qualification to determine the most efficient proportions of each material in concrete, PEMs are designed to last longer, have lower life cycle costs, and lower environmental impact amongst several other benefits. In addition, designers are able to enhance the quality of concrete by matching the properties of concrete to performance (Ahlstrom and Richter, 2018).

Rather than specifying prescriptive requirements for concrete mixtures (such as materials and methods), specification provisions based on performance requirements can allow concrete to be tailored to the environment and use in which it will serve (AASHTO 2017, 2018, 2019). Including performance requirements such as resistance to cracking and ingress of deleterious substances in specifications can result in concrete produced and constructed that is far more durable than concrete produced and constructed under prescriptive specification provisions (AASHTO 2017, 2018, 2019). Performance requirements allow concrete manufacturers to innovate and leverage their experience, adjusting mixture inputs with sustainability, economy, and constructability in mind, and producing concrete more finely tuned to perform under specific conditions.

Development and implementation of performance-related specifications is an extensive undertaking, and the shift will impact all stakeholders in the construction process. Therefore, the scope of work in this proposed project supports an <u>initial</u> effort by the North Carolina Department of Transportation (NCDOT) to move towards a specification for performance-engineered concrete mixtures. This literature review provides background information supporting the objectives and work performed, as outlined in the main body of this report.

A.2 Concrete Durability

In addition to mechanical sufficiency, the durability performance of concrete is an important consideration in the long-term success of the structure or pavement. In its 2013 circular on concrete durability, Transportation Research Board (TRB) Committee AFN30 Durability of Concrete stated “durability is not an intrinsic, measurable property of concrete. Instead it is a set of material properties that are required for the concrete to resist the particular environment in which it serves (Taylor et al. 2013).” Many mechanical property tests provide insight into structural capacity and early age performance, but do not often provide a good indication of concrete’s performance over the life cycle. The emphasis of ongoing and current research associated with concrete durability performance focuses on material properties and characteristics linked to successful long-term performance, along with tests that provide insight into properties related to long-term durability.

To be durable, concrete must withstand distress from a variety of aggressive agents and environmental conditions, as well as service loads. Concrete durability has been defined as “the ability of concrete to resist weathering action, chemical attack, and abrasion while maintaining its desired engineering properties (Kosmatka et al. 2016).” The American Concrete Institute (ACI) Guide to Durable Concrete addresses fresh properties, resistance to freezing and thawing deterioration, resistance to alkali-aggregate reaction (AAR), resistance to chemical attack and corrosion, and resistance to abrasion (ACI 2008). The environment in which concrete is produced, placed, and maintained also plays a major part in the performance of the concrete. This is why identical mixtures placed in different climates can produce vastly different short and long-term performances (Kockal and Turcker 2007).

Weathering can be thought of as the effects of exposure to weather and climatic conditions on the concrete structure, along with other factors such as exposure to chemicals, storm water, or other elements. Wind, precipitation, temperature change, humidity, and other environmental factors can cause or contribute to deterioration of the concrete. Concrete is susceptible to attack from chemical substances introduced in the form of sulfates, chlorides, or other compounds. When these chemicals are introduced, reactions can occur producing new substances growing in the concrete structure. Secondary reactions, which often involve materials aside from those initially present during cement hydration, are generally not desirable once concrete hydration is essentially complete (ACI 2008).

Weathering and other mechanical distress can also be exacerbated by mechanical loads. Abrasion of concrete surfaces becomes a more prevalent issue as traffic loads on roadway systems increase. The demand for shipping of goods has resulted in heavier weights and increased passes of freight trucks, and concurrently greater wear on concrete pavements. As traffic loads and design expectations continue to rise, ability to mitigate deteriorating factors becomes increasingly important (ACI 2008).

A.2.1 Performance Requirements for Durable Concrete

Performance-related specifications provide the ability for agencies to obtain the desired construction quality while allowing contractor greater control and flexibility (Ahlstrom 2016). For instance, current prescriptive specifications for minimum cement content and rate of strength gain may preclude the acceptance of mixtures that have superior economy, durability, and satisfactory mechanical performance, but contain high proportions of SCMs.

In September 2016, a proposed AASHTO provisional specification, AASHTO MP XX-17, "Standard Specification With Commentary for Performance Engineered Concrete Pavement Mixtures" was submitted to AASHTO member states for balloting (AASHTO 2016). This specification, which has been updated and reissued yearly between 2020, has become AASHTO PP 84, "Standard Practice for Developing Performance Engineered Concrete Pavement Mixtures." At the time of the start of this work, AASHTO PP 84-17 was the current standard, with PP 84-18 and PP 84-19 released during the course of the project. AASHTO PP 84 provides a framework and guidance for state highway agencies to develop a specification for performance engineered concrete mixtures that focuses on measurement and acceptance of concrete based on characteristics that have been linked to satisfactory long-term durability performance of the concrete. As stated in AASHTO PP 84-19, "A significant barrier to adoption of pure performance-based specifications, or even performance-related specifications, is the lack of effective test methods that assess the ability of a concrete mixture to resist the environment to which it is exposed. To address this barrier, new testing methods that measure performance-related parameters have been developed and are being evaluated in the field, while other advancements are emerging." Per AASHTO PP 84, Concrete performance parameters that should be addressed in a materials specification include:

- Sufficient strength
- Low risk of cracking and warping due to drying shrinkage
- Resistant to damage from freeze-thaw stresses
- Resistant to ingress by chemical deicers and other deleterious agents
- Low absorption, diffusion, and other transport related properties
- Aggregate stability to resist D-cracking and alkali-aggregate reactivity (AAR)
- Workable

Workability is included because many failures and performance issues are the result of (or linked to) inappropriate workability of the concrete mixtures for the placement technique utilized. Without the proper workability, issues such as segregation, poor consolidation, edge slump, and/or inadequate air content can result (AASHTO 2016).

Performance-related specifications require measurement of key properties and performance characteristics. In order for performance specifications to be successfully utilized, QA/QC tests should be rapid, effective, reliable, and inexpensive (Taylor et al. 2014). A number of state agencies, including NCDOT, are using and evaluating new, rapid, early-age testing technologies such as resistivity, sorptivity, and air void system analysis that support development and use of performance-engineered concrete mixtures. The capabilities of these tests to evaluate the durability performance of concrete mixtures is improving as state highway agencies build sufficient data to correlate the test results with durable field performance. Ongoing concrete materials research is beginning to provide NCDOT data to support use of performance engineered concrete mixtures. However, additional work is needed to identify appropriate performance measures, performance goals, and QA/QC protocol. To support FHWA's performance-engineered concrete initiative and implementation of AASHTO PP 84, ongoing research is being performed at other universities to enhance the knowledge of the basic science and emerging tests that form the foundation of the draft specification. NCDOT personnel have expressed a desire to concurrently perform research on North Carolina concrete mixtures to support implementation of certain targeted testing technologies suggested by AASHTO PP 84, and begin movement towards specification and use of performance engineered concrete mixtures.

A.2.1.1 Strength

Strength of concrete is its ability to resist rupture under an applied load. Much of this strength comes from the bond between the paste to the aggregates (Neville 2011). The required strength of concrete is different depending on the concrete element being designed. For vertical construction projects, the design strength is often specified as concrete's compressive strength, while flexural strength tends to be a more specified characteristic in pavement concrete mixtures. Specifications for compressive strength often utilize an age of 28 days for acceptance testing, although other ages such as 7 days or 56 days are commonly utilized depending on the concrete mixture type and construction objectives.

Careful control of the strength capacity of a concrete mixture to ensure it does not excessively exceed the required strength is important. Compressive strength in excess of that required is not always desirable, as this typically comes from an increased cement content, which increases cost and contributes to issues such as increased temperature during hydration and cracking). For monolithic bridge decks, crack densities have been shown to rise 0.16-0.49 m/m^2 as compressive strength increased from 31-45 MPA (4500-6500 psi) (Darwin et al. 2004). This increased likelihood of cracking stems from higher shrinkage (drying, autogenous, and plastic) in high paste content concrete. Higher compressive strengths can also lead to higher tensile strengths, increasing the changes that the reinforcement yields, as well as a higher modulus of elasticity, causing additional internal restraint (Frosch et al. 2003). Increases in the cement content also results in higher heat of hydration and increased risk of thermal cracking, as well as lower creep (Wright et al. 2014). An increase in strength does have a positive effect on abrasion resistance, as the two are positively related (Papenfus 2003).

AASHTO PP 84 identifies the different characteristics that affect the previously mentioned six aspects that have influence on concrete durability. Although not a conventional measure of durability, strength is often somewhat related to durability and will always be required to ensure adequate structural performance of a pavement or structure (AASHTO 2017). AASHTO PP 84 Section 6.3 indicates that concrete strength should consider either flexural or compressive strength, or both. The specification identifies a target value of 600 pounds per square inch (psi) at 28 days for flexural strength using AASHTO T 97, "Standard Method of Test for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading). Section 6.3.2 of AASHTO PP 84 specifies AASHTO T 22, "Standard Method of Test for Compressive Strength of Cylindrical Concrete Specimens" for the testing method, with a target of 3,500 psi at 28 days (AASHTO 2017, AASHTO 2018).

A.2.1.2 Resistance to Cracking

Cracking in concrete can be caused by a multitude of factors. The two ways in which concrete cracks are formed are differential volume change (shrinkage), and when movement is restrained (Maggenti et al. 2013, Mehta and Montiero, 2014). Causes of cracking can also be sorted into three categories: mechanical loading, volumetric stability, and environmental loading and durability (Taylor et al. 2013). Cracks occurring due to volumetric stability issues include those caused by settlement and shrinkage, as well as from temperature differentials during curing. Shrinkage occurs both in fresh concrete as well as in hardened concrete. In fresh concrete, plastic shrinkage cracking can occur within a few hours of placement. High capillary stress development near the surface is caused when the bleeding rate is exceeded by the surface evaporation rate (Cohen et al. 1989) as a result of factors such as high temperatures, high winds, low ambient humidity, and mixture design (ACI 1999). As the amount of shrinkage is directly related to the amount of water loss, the higher the evaporation the more cracking occurs.

Settlement cracking occurs as freshly mixed concrete settles and encounters restraint. Plastic settlement cracking occurs most commonly occurs at changes in cross section, and has been observed to occur in the construction of reinforced slab and bridge decks. Uniform settlement does not cause plastic cracking (as no tensile force is built up in the concrete), but differential settlement can lead to cracking. Clear cover depth, as well as rebar size and spacing, have been identified as major contributors to differential cracking (Weyers et al. 1982). Larger bars (and as a result, greater spacing) and smaller cover amount lead to larger cracking.

Autogenous shrinkage typically occurs when the water-to-cementitious materials (*w/cm*) ratio is below 0.42, and there is no loss of moisture (otherwise it could be considered as drying shrinkage). This type of shrinkage has been linked to several factors, and is known to occur at higher rates with higher temperatures, cement fineness, and cement content (ElSafty and Abel-Mohti 2013). Cracking in hardened concrete can result from the internal stress built up over time. If there is restraint on the free movement of concrete (the most common condition for concrete, particularly reinforced concrete, to exist in), cracking resulting from dying shrinkage is a concern. The amount of drying shrinkage that occurs is also related to aggregate type, as well as the cement paste content in the concrete (ElSafty and Abel-Mohti 2013). Using methods such as contraction joints and shrinkage-compensating admixtures can help prevent drying shrinkage for being a durability concern.

Temperature differences in concrete lead to tensile stresses, which in turn can lead to cracking. This thermal cracking can be reduced by controlling the rate at which the concrete cools, increasing the early age tensile strength, and reducing the maximum internal core temperature (ElSafty and Abel-Mohti 2013). This is primarily an issue in the first few days of the concrete's lifespan, as that is when the most changes in temperature occur.

Under mechanical loading, micro-cracking occurs immediately after sufficient loading is applied. If the bond between the aggregates and the cement paste is not strong enough, or the load is too high, these cracks will get larger and localize until they form larger, visible cracking. Cyclic loading, or fatigue, is the second contributed to mechanical loading failure. Progressive cracking occurring each time the concrete is loaded and unloaded builds on each other until the cracks become visible.

Over the last 50 years, Portland cements produced have been containing higher levels of tricalcium silicate (C3S) and alkalis (Bentz 2007). These increased C3S and alkali levels along with the cements becoming finer create concrete that hydrates at a faster rate than in the past. Concrete mixtures that hydrate rapidly gain strength at a much earlier age and have a higher heat of hydration, leading to a greater tendency to have issues with autogenous strains and stresses (ACI 2013). Proper curing techniques are needed to combat the possibility of cracking as increased hydration rates tend to use more of the water for hydration earlier in the concrete's strength development period.

At later ages, concrete in certain exposure conditions is susceptible to cracking due to reinforcing steel corrosion. As corrosion of the steel occurs, the oxidation byproducts seek to fill voids in the concrete, creating internal stresses. These stresses eventually cause cracking which extend from the steel to the surface of the concrete (Alonso et al. 1998). Increased permeability caused by cracking allows more deleterious substances such as chlorides to enter the concrete which can increase the rate of corrosion, and subsequently the loss of load bearing capacity.

A.2.1.3 Resistance to Deleterious Substances

Transport of fluids and gases into concrete can result in a variety of durability issues. In fact, it is believed by many that permeability may be the most important factor related to concrete durability (Baykal 2000, Taylor et al. 2013). Fluids and gasses can carry deleterious substances such as sulfates, chlorides, and deicers. Permeability is defined as a measure of flow through a material due to a pressure differential. However, it is often expanded to include other transport mechanisms including absorption and diffusion (Mehta and Montiero 2014). In concrete, function of the quantity of the voids or pores within concrete as well as their interconnectedness, and permeability tests aim to provide a measure of the pore structure of concrete (Milla et al. 2020, in review). Figure A.1 provides a schematic of transport phenomena in concrete. In general, five primary transport mechanisms include 1) adsorption, 2) vapor diffusion, 3) liquid assisted vapor transfer (film transfer), 4) saturated liquid flow, and 5) ionic diffusion under saturated conditions (Nokken et al. 2006).

Corrosion of the reinforcing steel is a common cause for cracking of concrete. Corrosion can happen in various ways, but is often exacerbated by chloride ingress. Chlorides are most commonly introduced into roadway systems in colder climates via deicing salts. Concrete permeability is the concrete's tendency to allow water and air through the system. Aside from water being a conveyor of harmful substances into concrete, permeability of concrete can allow oxygen to become an issue. When oxygen is allowed to react with reinforcing steel, galvanic reaction can cause corrosion (Samples and Ramirez 1999).

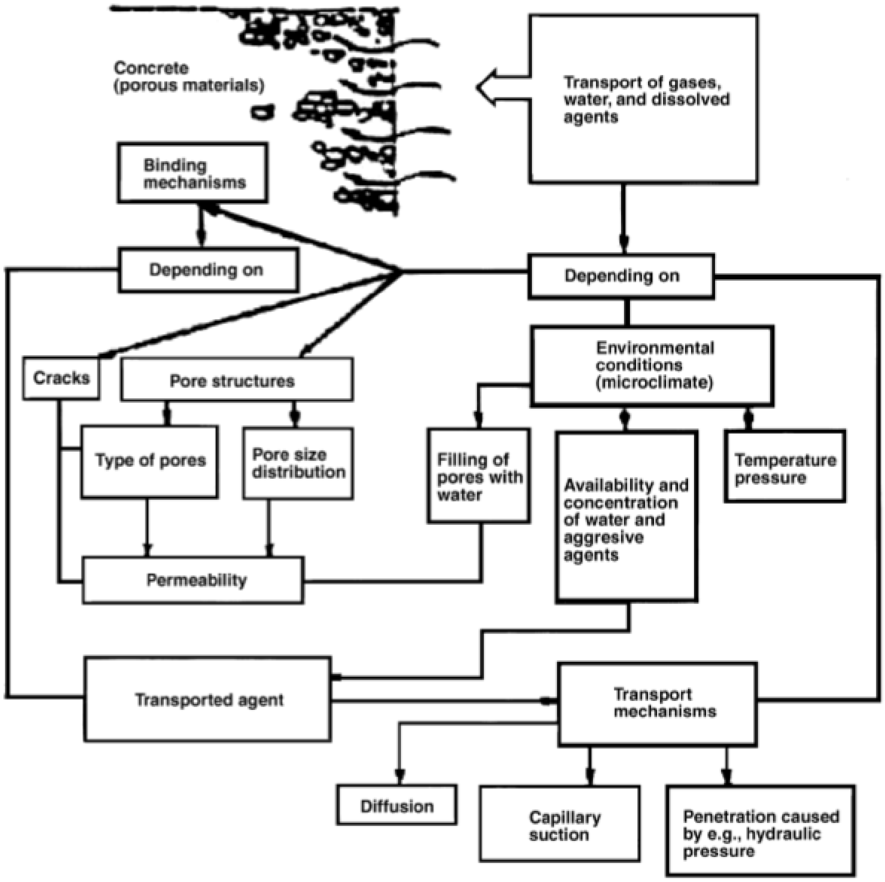

Figure A.1: Transport phenomena in concrete (from ACI 201.2R-4, originally Schiessl 1992)

Factors affecting concrete permeability include the type and composition of aggregates (Zaharieva et al. 2003, Debieb and Kenai 2008, Tangchirapat et al. 2012, Melugiri-Shankaramurthy et al. 2018), the cementitious materials content, the use of supplementary cementitious materials (SCMs) (Hassan et al. 2000, Yang and Wang 2004, Yazici 2008, Dhanya et al. 2018), the use of chemical admixtures such as air-entraining admixtures (Zhang et al. 2018, Wong et al. 2011), shrinkage reducing admixtures (Weiss 1999, Ribeiro et al. 2006, Qiao et al. 2015), and water reducing admixtures. The use of water reducing admixtures has provided stakeholders perhaps one of the most commonly utilized means of reducing concrete permeability, achieving the desired workability at a lower slump.

In addition to the types of materials utilized in a concrete mixture, the mixture proportions will also affect the permeability of concrete. Concretes with lower water/cementitious material (*w/cm*) ratios tend to last longer due to lower permeability (Goto and Roy 1981, Dhir et al. 1989, Chen and Wu 2013, ACI 2016), and many transportation departments have utilized water/cementitious ratio limits as a specification provision to support durable concrete (Taylor et al. 2013, Sutter et al. 2018).

The age of concrete and curing regime affect the permeability of concrete. Appropriate curing measures, particularly at early ages, are essential to ensuring that adequate moisture is available to support the hydration of cement and pozzolanic reactions of SCMS (Nassif and Suksawang 2002). As hydration occurs, the hydration products fill voids within the cement past, reducing total volume of the capillary pores, as well as the size of the interconnectivity of these pores (Mehta and Montiero 2014). Poor curing has been shown to be more detrimental to concrete's permeability than to strength (Lamond and Pielert 2006).

Concrete cracks also affect the permeability of concrete, accelerating the ingress rate of deleterious substances as well as the rate of deterioration. Much research, however, has focused on transport mechanisms of uncracked concrete, which does not represent typical field conditions for many concrete elements which have cracked due to service loads, thermal changes, shrinkage, and deleterious reactions. It has been shown that once a critical crack width is achieved, crack widths impact the diffusion coefficient (Jang et al. 2011).

A.2.1.4 Abrasion Resistance

For concrete mixtures to be used as a roadway, abrasion resistance must be considered in order to ensure that the material resist wears from vehicle tires. The abrasion resistance of concrete is defined as "the ability of a surface to resist being worn away by rubbing and friction" (ACI 2000). Abrasion resistance is linked to several factors: compressive strength, aggregate properties, surface finishing, curing, and use of surface hardness or toppings (Hadchiti and Carrasquillo 1988, Bakke 2006).

The quality of the aggregates, particularly the aggregate hardness, have been shown to heavily influence the abrasion resistance of concrete (Liu 1981, Laplante et al. 1991), likely due to their use as the majority of the volume of the material. The strength of the concrete has also been shown to be positively correlated to abrasion resistance (Witte and Bakstrom 1951). Use of SCMs have also been shown to increase concrete's abrasion resisance (Holland et al. 1986, Laplante et al. 1991).

Concrete with a higher *w/cm* will often have lower abrasion resistance, so a suitably low *w/cm* ratio (particularly near the top of the concrete) is required to ensure this durability performance aspect is achieved (Liu 1981, Bakke 2006). This can be achieved by using water reducing admixtures, taking steps to prevent bleeding, or avoiding the addition of water while finishing. Proper finishing and curing procedures are also necessary. Finishing of concrete should be done after bleedwater has evaporated, as applying it before would decrease the strength of the upper layer (Bakke 2006).

A.2.2 Characteristics of Durable Concrete

A.2.3.1 Materials

Selecting appropriate and efficient materials for concrete is essential in having durable, sustainable concrete. The primary materials that impact concrete durability include aggregates, admixtures, and (the most expensive and highest environmental impactor) portland cement. Other cementitious materials, particularly SCMs, are often essential to ensuring concrete that is durable is batched and placed. Material selection should provide the prescribed mechanical performance while also considering durability goals. A concrete mixture design that combines the correct materials in proper proportions will succeed far more often than if either design aspect is neglected (Kosmatka and Wilson 2016).

Cementitious Materials

Cementitious materials mixed with water acts as the binder paste for concrete, and is responsible for most of the concrete's overall strength (Kosmatka and Wilson, 2016). Concrete durability can be directly tied to the chemical and physical properties of cement along with the microstructure (Taylor et al. 2013). For example, finer cementitious material will hydrate faster and reacts with water rapidly since the surface area is larger. Cement type selected for a concrete element should be based upon the concrete's proposed application, and cements should be chosen in a manner which supports long-term durability in addition to mechanical strength. The paste content (quantity of paste) and the quality of the paste in concrete are two main factors that can affect permeability characteristics (Hearn et al. 2006).

SCMs such as fly ash, silica fume, and slag have proven to be beneficial as a cement replacement. SCMs can improve mechanical properties, and generally provide improved durability characteristics as well (Papadakis 2000). As concrete cures, its pore structure becomes finer and less permeable (Cui and Cahyadi 2001). Cementitious materials, including SCMs can produce denser paste structures, and therefore less permeable concretes (Kosmatka and Wilson 2016). Using SCMs improves hydration of the paste by bonding ions to hydration which would normally be free to become deleterious and detrimental to concrete performance (Taylor et al. 2013).

SCMs are also beneficial for concrete mixtures placed in cold environments and/or places that use chemical deicers. Calcium chloride from deicers can cause reactions that will expand and crack hardened concrete due to calcium oxychloride formations (Farnam et al. 2015). SCMs will reduce the calcium chloride content due to dilution and the tendency of some SCMs to absorb the calcium chloride (Cackler et al. 2017). The appropriate proportion of fly ash, slag, and/or silica fume mixed with cement can increase resistance to sulfate attack. Conversely, too much fly ash will make the concrete susceptible to sulfate attack, so proportioning is a key factor in controlling how SCMs affect concrete (Kosmatka and Wilson 2016).

In addition to the concrete durability performance benefits resulting from substitution of SCMs for ordinary portland cement, many of the beneficial SCMs approved for use in concrete are a byproduct of another industry, providing sustainability benefits (Juenger and Siddique 2015). SCMs also lower heat of hydration in concrete, which is beneficial when placing concrete in hot weather or in mass concrete structures (Cackler et al. 2017). In addition, SCMs reduce bleeding and segregation of fresh concrete (Taylor et al., 2013). Another advantage to using SCMs, is the fact that less water is needed to increase workability (Cackler et al. 2017a).

The primary SCM utilized in this study is fly ash. Fly ash is the byproduct of coal combustion in coal-fired electrical power plants. During the processes of burning coal, clays and other inorganic material present within the coal are melted, then solidify as they are cooled by the exhaust fumes of the coal plant. This creates fine, hollow, fine, spherical particles which are trapped in electrostatic precipitators or bag filters before they can disperse into the atmosphere. Fly ash is made primarily of silicate glass containing silica, calcium, iron, and alumina. Crystalline compounds can also be present, as well as minor constituents such as sulfur, sodium, magnesium, potassium, and carbon. Currently, fly ash is the most widely used SCM in concrete due to cost savings, and behavior in concrete (Kosmatka and Wilson, 2016).

Class F and Class C fly ash in accordance with ASTM C618, "Standard Specification for Coal Fly Ash and Raw or Calcinated Natural Pozzolan for Use in Concrete," are defined by calcium content as Class F generally is low-calcium (less than 10% CaO) while Class C generally is high-calcium (10-30% CaO) (ASTM 2015). In North Carolina, Class F ash is commonly used, so for the rest of this literature review, Class F fly ash is implied, as it is the fly ash used in the testing program.

The characteristics of fly ash change depending on coal type, boiler type, operating conditions, and processing, which affects the overall efficiency of fly ash in concrete (Xu and Shi 2017). Fly ash in concrete tends to impact fresh and hardened concrete properties in several ways. The spherical shape of fly ash generally increases the workability of fresh concrete, reducing a mixture's water demand, reducing bleeding and segregation, and reducing the heat of hydration released while batching and placing fresh concrete (Kosmatka and Wilson 2016).

Along with improving fresh concrete properties, fly ash used in concrete can enhance hardened properties as well. The spherical shape of fly ash particles improves the particles' ability to fill voids in order to increase long-term strength and decrease permeability (Kosmatka and Wilson 2016). As a pozzolanic material, fly ash tends to impact the overall strength of concrete. Although most specifications require a minimum 28- day compressive strength, 7-day strength of concrete is generally 75% of the strength at 28-days and can often be used to estimate strength at 28-days (Neville 2008). Fly ash tends to gain strength slowly initially, but the ultimate strength is higher than concrete made with just portland cement (Mehta and Montiero 2014). For example, concrete made with Class C fly ashes tends to gain strength earlier than mixtures produced with Class F fly ash, but concrete made with either Class C or Class F ash tends to surpass specified 28-day strength in 28-90 days (Kosmatka and Wilson 2016). However, research has shown that fly ash replacement levels over 35% tend to decrease overall compressive strength which is why most specifications limit substitution to 30 % (Kurad et al. 2017).

There are a few challenges to using fly ash in concrete. One of the main issues that directly impacts constructability and contractors is the slow early strength gain. As mentioned previously, concrete with fly ash tends to gain strength more slowly than concrete with portland cement, which could cause delays in finishing and passing inspections ultimately delaying the construction schedule (Taylor et al. 2013). Since NCDOT standard specifications state 28-day strength requirements, the slower rate of strength gain of concrete made with SCMs is not specifically considered. Also, the addition of fly ash tends to require more air-entraining admixture due to fine fly ash particles, and the tendency of unburnt carbon remaining in the fly ash to interfere with the admixture's ability to retain bubbles in the mixture, which could increase the overall cost of concrete per CY (Kosmatka and Wilson 2016). Another challenge is the current scarcity of fly ash in some regions, as well as potentially running out of fly ash in the next coming years. As the demand for fly ash increases locally and globally, there is a potential danger of low supply or high cost due to power companies turning to alternate power methods, such as natural gas (NPCA 2017).

Proper placement and curing and of concrete with fly ash (and any other SCM) is essential in avoiding plastic shrinkage since bleeding is reduced (Taylor et al., 2013). Generally, fly ash in low doses does not impact drying shrinkage directly. However the slow setting time of concrete made with fly ash can increase drying shrinkage and prolong finishing operations (Kosmatka and Wilson 2016). Shrinkage can be mitigated with immediate and constant (over the specified duration), curing of the concrete to ensure it continues to hydrate, since the reactions are delayed due to the fly ash (Taylor et al. 2013). Since SCMs like fly ash can replace a portion of cement, the *w/cm* ratios can be lowered as well (while achieving the same workability) resulting in a more durable mixture (Cackler et al. 2017a).

Aggregates

Typically, the coarse and fine aggregates in a concrete mixture compose the largest volume of inputs to concrete, and for this reason, selecting aggregates that will allow the concrete to perform as designed is critical. Aggregates should be stable, meaning non-reactive by nature. Non-stable aggregates can react with other materials resulting in problems such as ASR which impact concrete performance. Properties such as gradation, specific gravity, absorption, particle shape/angularity, abrasion/impact resistance, chemical stability, and chemical composition will vary by source, and should be considered during mixture design (ACI 2007). The nature of the particles and the grading of the aggregates in concrete influence the workability of fresh concrete, as well as their impact on the strength and durability of the hardened concrete. The characteristics of the particles, which includes shape, surface type, and porosity, affect how the bond between the concrete paste and the aggregates themselves, as well as the mixture proportions (ACI 2007).

Typical aggregates used in portland cement concrete are dense and inert, although aggregate size and shape impact concrete longevity in multiple ways. For example, rounded aggregates require less water than more angular coarse aggregate for equal slump (Mamlouk and Zaniewski 2011). Concrete mixtures that have well-graded aggregates last longer due to water reduction and improved dimensional stability that is a result of having aggregates of multiple sizes (Kosmatka and Wilson 2016). With an appropriately graded aggregate system, the volume of paste can be reduced, decreasing permeability (Taylor et al. 2013).

The area in which aggregates and paste meet is called the interfacial transition zone (ITZ). The characteristics of the ITZ are influenced by aggregate composition and size. Often considered the "weak link" in concrete's microstructure, the ITZ is considered to be an at-risk area: an increases in the size of the ITZ or the density of the components comprising the ITZ results in an increase in concrete's permeability (Mehta and Monteiro 2014).

Admixtures

As the performance expectations and requirements for concrete have become more diverse and challenging over the years, admixtures have become a more important input component in concrete mixture design. Admixtures can provide benefits to several performance characteristics of concrete, from workability in fresh concrete to color to air content. It is important to consider interaction between admixtures when considering use in a design, as adverse effects could occur (Kosmatka et al. 2016).

An air entraining admixture (AEA) is specified in a majority of concrete mixture designs when the element is exposed to moisture and freezing and thawing conditions. An adequate total air content, which must be a well dispersed network of small voids, is required for freeze/thaw durability (Jeknavorian 2006). This network of air voids is able to accommodate ice or unfrozen water during a freezing cycle, reducing pressures within the concrete and preventing damage from freezing and thawing stresses (Hover 2006).

Water-reducing admixtures and superplasticizers lower the water content, and as a result to *w/cm* ratio, which results in lower permeability to water and deleterious substances. Lower *w/cm* ratios result in a denser paste microstructure (and subsequent lower paste permeability) due to the presence of less water remaining in the paste after hydration (Neville 2008, Mehta and Monteiro 2014). Corrosion inhibitors improve corrosion resistance from chloride by reducing the corrosion rate, but caution must be used since these have only been tested over short-term, which can be misleading (Berke et al. 1997). Shrinkage-reducing admixtures reduce drying shrinkage in restrained concrete (Nmai and Kraus 1994).

A.2.3.2 Mixture Proportions

Mixture proportioning is important for a number of reasons, such as its influence on fresh properties, the ability of a mixture to meet the required mechanical properties and durability performance, and adhering to specification provisions and/or guidance limiting specific materials based on application (Kosmatka and Wilson 2016, ACI 2019). To optimize concrete performance, concrete mixtures must be designed with proportions that 1) are economical and 2) tend to exhibit the appropriate workability and ease of placement and finishing in the field (Kosmatka and Wilson 2016). Mixture characteristics should be selected based upon the environment of concrete placement and service, the size and shape of concrete members, the desired physical and chemical concrete properties, and anticipated exposure conditions (Taylor et al. 2013, ACI 2019). Also, concrete performance characteristics such as resistance to sulfate attack and resistance to chloride penetration should be considered (Cackler et al. 2017).

Early mixture designs were simple, using a 1:2:3 ratio of cement, sand, and coarse aggregate (Abrams 1918). Present mixture proportioning methods are more technical, utilizing measured material properties along with rules-of-thumb or computational algorithms to calculate proportions of mixture materials. The most commonly used mixture proportioning method is ACI 211.1, "Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete" (ACI 2002). The mixture proportioning phase also allows the designer to consider proportion characteristics related to economic considerations and sustainability.

When the concrete has workability to support proper consolidation and includes sound, durable aggregates, the *w/cm* ratio is the primary factor influencing concrete strength and durability (Taylor et al. 2013). Because of the *w/cm* ratio's major impact on hardened concrete properties, over 100 years since its initial study, *w/cm* ratio continues to be one of the primary proportioning decisions (Cackler et al. 2017a). For instance, some guidance suggests concrete needing corrosion protection for steel reinforcement should not have a *w/cm* ratio over 0.40 with a minimum strength of 35 MPa (approximately 5,100 psi), while concrete in an area where frost resistance is desired should have a *w/cm* ratio of 0.45 with a minimum strength of 31 MPa (approximately 4,500 psi) based on consensus of the authors of the Durable Concrete Circular (Taylor et al. 2013). Controlling the *w/cm* ratio of concrete is the best way to mitigate drying shrinkage, as more water leads to increased likelihood of cracking or shrinking (Cackler et al. 2017a).

The project requirements, as well as the available materials for achieving it will be the baseline for determining the workability needed for the mixture. The water demand to achieve the desired workability is influenced by a range of conditions: cementitious materials content, the size, gradation, and shape of the aggregates, the air content, slump, use of admixtures, etc. Factors such as increasing air content and using water-reducing admixtures can decrease the overall water need, while increasing the cement content or slump increase the water required for the mixture (Taylor et al. 2013).

Cement and SCM contents are essential in proportioning for several reasons. Since cement is the most expensive component of concrete, it is most generally most economical to limit cementitious content without impacting quality; however, the proportion of cementitious content should be based on performance requirements instead of solely economic benefits (Taylor et al. 2013). The overall cementitious materials content required for the mixture is driven by the desired strength, the *w/cm* ratio, and the water content. Research has shown that using the stiffest mixture that will work practically, the largest practical nominal maximum size aggregate, an optimum ration of fine-to-coarse aggregate, and a uniform aggregate distribution will all aid in minimizing water and cementitious material content (ACI 2002). SCMs can be proportioned into the cementitious materials content depending on the requirements for the final concrete product.

Specification provisions used by many state highway agencies result in relatively high cement contents. Although these high cement contents ensure compressive strengths are met at an early age, these high cement contents are also responsible for many durability problems in concrete (Taylor et al. 2013). Optimized aggregate gradations have emerged as a useful mixture proportioning approach that improves particle packing (reducing paste content and associated durability issues) and workability. The use of aggregates for particle packing can reduce the amount of cement needed in mixture designs to reach comparable strengths (Ley et al. 2012, Cook et al. 2013). Cement contents are also easily reduced using SCMs. Replacing a portion of the cement with a SCM can reduce the negative effects of high cement contents, as well as offer the benefits to concrete durability and internal structure described previously.

In regard to *w/cm* ratio, only 30% water by weight of cement is needed to hydrate plain cement particles (Mehta and Monteiro 2014). Therefore, *w/cm* are typically specified with workability as a governing factor. Concrete mixtures used for a variety of applications use much higher water contents (*w/cm* ratio of 0.40 or greater), meaning roughly half the water in the mixture design is included solely to improve workability. As previously discussed, means are available to reduce *w/cm* while maintaining workability, such as WRAs and plasticizers. These admixtures work by influencing the electrostatic properties of the cement particles, allowing less water for proper hydration, and therefore more is available for workability (Kosmatka and Wilson 2016).

The two major components of aggregates that are important in terms of proportioning are the gradation (particle size distribution) and the nature of the particles (shape, porosity, surface texture), as both play a major role in the workability of the fresh concrete and the strength of the hardened concrete (Mehta and Montiero 2014). As mentioned previously, optimized gradation leads to economical mixtures, greater workability, and less water and cementitious material are required to fill inter-aggregate spaces. Typically, a lack of midsized aggregates will likely result in concrete with poor workability, high water content, and high shrinkage properties (Kosmatka and Wilson, 2016). In order to compensate for lack of midsized aggregates, blending is suggested to ensure the aggregates are well-graded (Taylor et al. 2013). In addition, desired grading for fine aggregates depends on the type of work, leaner mixtures a finer grade is desired to ensure the concrete is workable, while richer mixtures coarse grading is more economical (Taylor et al. 2013).

Using larger aggregates reduces the required amount of paste and can help decrease shrinkage, but there are also limits on how large the aggregates can be based upon construction considerations such as minimal spacing between reinforcing bars (Kosmatka and Wilson 2016, ACI 2018 building code). The maximum aggregate size should not exceed three-fourths of the clear space between reinforcing rods/wire, prestressing tendons, or sets of bars. It should also not exceed one-fifth of the minimum distance between the sides of the forms (Taylor et al. 2013). This allows to concrete to flow through the forms without concern of clogging around the reinforcing material. For fine aggregate, the grading depends on the work, the richness of the mixture, and the size of the coarse aggregate.

For concrete exposed to deicing chemicals as well as freeze thaw cycles, entrained air must be used to allow for durability, as well as improves the workability of the concrete. Air entrainment can be controlled by using air-entrained Portland cement or by adding air-entraining admixtures. The recommended amount of air content is dependent on the maximum aggregate size, as well as the exposure level of the concrete (ACI 2002).

A.2.3.3 Construction

Selecting quality materials and proper proportions are important to ensure the desired durability performance, but if the concrete is not batched, placed, and cured properly during construction, poor performance can result. Incorrect placement and/or curing methods can also compromise the concrete's mechanical properties. Many of the factors relating to concrete durability such as pore structure and air void systems are directly influenced by how the concrete is placed and finished (Hearn et al. 2006, Hover 2006). Material storage, batching, transport, placement consolidation, and curing methods each impact the quality of fresh and hardened concrete. Avoiding segregation between the coarse aggregate and the mortar is necessary, and any mixtures that will be pumped into place must have the air content change taken into account during the design phase. If workability of the concrete is of concern, retarders are often used to delay set time. These retarders can result in a lack of small entrained air bubbles (Du and Folliard 2005).

Materials must be stored, batched, mixed, and transported in a manner that supports successful placement. For example, the aggregate chosen for a mixture should be piled, transported, and stored properly to prevent segregation, prevent contamination, and ensure moisture content is controlled (Kosmatka and Wilson 2016). Methods utilized for batching, mixing and transporting concrete should comply with ASTM C94, "Standard Specification for Ready-Mix Concrete" (ASTM 2020). During concrete batching, each individual material must be added within tolerances provided by specifications. Concrete should be mixed thoroughly until all materials are uniformly distributed, and re-mixed within limits if mixture stiffens during transport (Kosmatka and Wilson 2016).

Transportation from the mixer to the site of placement should be done with minimal impact to the original design conditions (such as slump, *w/cm* ratio, air content, etc.). Different drum colors can help with this, as darker colors can help retain solar energy in colder regions and light colors can help prevent excess heat gain in warmer regions (Lane 2013). For hot and cold weather concrete placements there are provisions that must be followed to guarantee expected properties after concrete has set (Taylor et al., 2013). Guidelines for proper concrete placement are outlined in ACI 304R (ACI 2009).

Construction factors such as water added on-site, placement, and curing measures have a direct influence on concrete pore structure. Much like pore structure, vibration can also have an effect on air void systems. Vibration can result in smaller air bubbles forming with larger ones, directly influencing the air void system (Du and Folliard 2005). Improper vibration techniques of concrete can lead to the destruction of the pore structure of concrete through thixotropy, or the lessening of viscosity (Chappuis 1990). By adding water on-site for workability purposes, a contractor can exceed the maximum *w/cm* which will affect pore structure. Proper consolidation helps remove excess voids in the concrete and increases its durability. Guidance on consolidation can be found in ACI 309R (ACI 2005).

Finishing and curing are the final steps in the construction process. Finishing should be performed with as little manipulation as possible, as overworking the concrete surface may reduce the surface air content and cause fine aggregates to rise to the top, increasing the cracking potential. Saw cutting of the groves and joints should be delayed until the concrete is strong enough to prevent coarse aggregate movement. If concrete is not properly cured, water at the surface can be evaporated and cement particles will not have the water needed for proper hydration (Kosmatka and Wilson 2016).

The PEM initiative aims to improve performance of concrete while also catering to the concerns of contractors performing the work. Without being able to effectively transport, place, and finish concrete, even a mixture that has characteristics to support durable performance will suffer from issues associated with improper construction. Workability, flow, pumpability, and finishability are characteristics considered in performance engineered concrete mixture designs (Ley et al. 2014, Cook et al 2014, Wang et al. 2017, Cackler et al. 2017a and 2017b). Durable concrete should be handled with care and placed/finished in accordance with project specifications and quality assurance provisions.

A.2.3 Selected Tests to Evaluate Concrete Durability

Durability tests traditionally aim to evaluate concrete's ability to conduct three mediums: air, water, and electricity (Milla et al. 2020). The less conductive concrete is to these mediums, the more likely the concrete can resist the ingress of harmful substances and deterioration. Water permeability can cause poor freezing and thawing results as well as allow the ingress of chlorides, sulfates, and other deleterious substances into the concrete. It is important to note that the conditioning of the concrete test specimens (particularly moisture saturation level, pore chemistry, and temperature) can greatly influence durability testing results (Nokken et al. 2006).

Historically, tests to evaluate concrete durability performance often take much more time to perform than mechanical property tests, and can require sophisticated test equipment and/or highly trained operators. The longer duration and technical challenges associated with these tests are often cited as a key factor influencing an agency's hesitancy include durability tests in their specifications for mixture design approval and product acceptance. To illustrate this point, one of the most common durability tests, freeze/thaw testing (ASTM C666, “Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing”) generally takes 2.5 months to complete (ASTM 2015), and requires sophisticated and expensive equipment for conditioning and testing. Mechanical tests typically specified in concrete acceptance, namely slump, air content, and compressive strength, can be completed in a fraction of this time (in minutes) with more affordable equipment (in the case of slump and air content). Although easy to perform and rapid to complete, these tests used for acceptance are not good indicators of long-term performance of concrete pavements. The PEM initiative aims to develop and promote durability tests that are much simpler to perform as well as take much less time than current durability tests (Cackler et al. 2017b, AASHTO 2017).

In the PEM initiative, concrete durability is characterized by a number of performance measures. AASHTO PP 84 presents a list of characteristics that influence the durability of the concrete. This standard capitalizes upon recent advancements in testing technologies, specifying some recently developed tests that are more rapid and easier to perform, and have been linked to durability performance in other tests and/or field performance (AASHTO 2019). In AASHTO PP 84, concrete strength, resistance to cracking and warping due to shrinkage, freeze-thaw durability, resistance to chemical deicers, aggregate stability, and workability are presented as the primary focus areas associated with durability performance (AASHTO 2019). Additionally, properties such as low absorption, diffusion, and other transport related properties have an influence on the pavement’s durability. AASHTO PP 84 provisions can be used in specifications during the mixture design phase, in mixture qualification testing, and in some cases, acceptance testing performed during and after construction. By providing a suite of tests for each performance measure, AASHTO PP 84 allows agencies flexibility in the type of test (more complex vs. simple, highly correlated to field performance vs. more loosely correlated) and risk tolerance.

As the industry aims to minimize restrictive prescriptive specification requirements and moves towards incorporating selected performance requirements, contractors are expected to ensure mixture production and quality control (Cackler et al. 2017b, AASHTO 2019). Current PEM specifications change the narrative of quality control (QC) for contractors encouraging enhanced practices to improve concrete performance and potentially reduce agency burden for quality assurance (QA) (Cackler et al. 2017b).

A.2.3.1 Shrinkage Tests

AASHTO PP 84 identifies volume of paste, unrestrained volume change testing, restrained shrinkage, and cracking potential as influencing factors for the pavement’s ability to resist warping and cracking caused by shrinkage. Of the suggested tests, only one should be selected for project specifications. AASHTO PP 84 includes prescriptive options for reducing shrinkage, which are limiting paste content of the concrete to 25%, or testing for unrestrained volume change (AASHTO 2019). AASHTO T 160, “Standard Method of Test for Length Change of Hardened Hydraulic Cement Mortar and Concrete” (also ASTM C157, “Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete”) is the specified test method for unrestrained volume testing, with a target value of less than 420 microstrain at 28 days (AASHTO 2017, ASTM 2017). AASHTO PP 84 Section 6.4.2 provides alternative performance specifications for testing. As opposed to the prescriptive specification for the same test, target values are 360, 420, or 480 microstrain at 91 days depending on the application. This linear (volumetric) shrinkage test is commonly utilized and easily performed, although in recent years the shortcomings of this test (including its inability to capture early age autogenous and chemical shrinkage potential) have led many agencies to question its usefulness (ASTM 2019, Weiss 2017).

Previous editions of AASHTO PP 84 (2017) provided provisions for use of restrained shrinkage testing, although this has been removed from the 2019 edition of AASHTO PP 84 as additional research in this area is ongoing. Restrained shrinkage testing was specified using either AASHTO T 334, “Standard Method of Test for Estimating the Cracking Tendency of Concrete”, or AASHTO T 363, “Standard Method of Test for Evaluating Stress Development and Cracking Potential due to Restrained Volume Change Using a Dual Ring Test” (AASHTO 2017). These tests are similar to ASTM C1581, “Standard Test Method for Determining Age at Cracking and Induced Tensile Stress Characteristics of Mortar and Concrete under Restrained Shrinkage” (ASTM 2018). If specifying using AASHTO T 334, the suggested target value was no cracking at 180 days (AASHTO 2017). AASHTO T 363 should have stress results less than 60% of splitting tensile strength for 7 days. Computational programs can also be used to evaluate cracking potential (AASHTO 2008). Computational programs should have a determined cracking probability of less than 5%, 20%, or 50% depending on curing conditions and the application.

A.2.3.2 Permeability Tests

Water and Air Permeability

Permeability of concrete is directly related to pore structure and concrete density. The denser the paste matrix is developed within the concrete, the less permeable the concrete mixture. Water absorption testing is used to determine the permeability of concrete in regard to water. ASTM C1585 "Measurement of Rate of Absorption of Water by Hydraulic-Cement Concretes" outlines the test procedures for finding the concrete's sorptivity by means of ponding water on one surface of a specimen (ASTM 2013). Sorptivity is defined as the action of absorbing and transmitting water by means of capillary force in a porous material (Hall 1989). The test procedures in ASTM C1585 are used to evaluate concrete absorption with only one face exposed to water. This method provides an accurate representation of the surface exposure of a concrete structure or pavement. Sorptivity is influenced by a number of factors including the mixture proportions of the concrete (as well as presence or absence of chemical admixtures or SCMs), physical characteristics and chemical composition of mixture inputs, content of entrained air, methods of concrete placement, methods of finishing, curing quality, age, microcracking in the concrete, surface treatments, and moisture conditions (ASTM 2013).

Air permeability testing is less common due to the low number of tests that can be performed on site (Claisse et al. 2003). Testing for air permeability can be either destructive testing like in Claisse's method, or non-destructive if a device like a Proceq Torrent device is used (Proceq 2019). The theory behind both tests involves applying a vacuum for a set amount of time while measuring pressure changes in the concrete.

A variety of other air and water permeability tests have been developed over the past decades. These methods include those that can be constructed within a laboratory setting and others for which commercial devices are available for purchase. A summary of these tests, along with strengths and limitations is presented in Milla et al. (2020).

Electrical Tests

Over the past decades, electrical tests have emerged as an effective method to evaluate the resistivity (and hence permeability) of concrete. In the 1960's, the FHWA identified reinforcing steel corrosion as an exacerbating factor in premature bridge deck failures. The ensuing investigations linked corrosion to the chloride ion penetration resulting from deicing salts used on the bridge decks (Kassir and Ghosn 2002). Funding was implemented to develop a rapid field test for the identification of concrete permeability. Techniques developed in the early 1970's by Levitt and Figg were useful for measuring concrete permeability relative to water or air, but the FHWA put priority on development of a test to measure chloride ion penetration (Levitt 1970, Figg 1973). In 1982, the AASHTO standard for RCPT was presented and approved as AASHTO T 277, "Standard Method of Test for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration" (AASHTO 2015). ASTM also endorsed the test method in 1991, producing ASTM C1202, "Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration" (ASTM 2018). The test method presented in both of these standards is commonly known as the rapid chloride permeability test, or RCPT.

RCPT is an electrical test method used for measuring electrical conductance of concrete. Electrical conductance provides an indication of the concrete's ability to resist penetration of chloride ions. Testing is performed by passing 60 volts of direct current (DC) through a 50mm thick concrete cylinder. The ends of the cylinder are submerged in solution, one side being sodium chloride and the other being sodium hydroxide (Whiting, 1981, ASTM 2018). Although the RCPT has often shown good correlation to field performance, and has historically been relied upon as a primary durability performance evaluation tool, this test has a number of limitations and shortcomings. As part of the research presented in Mohr, it was determined that location and depth of specimen extraction can have an impact on RCPT results (Mohr et al. 2000). Ruettgers and Vidal (1935) and Cook (1951), and more recently Lomboy and Wang (2009) demonstrated that permeability was significantly increased as *w/cm* increased. RCPT results have also been correlated to *w/cm* by Ruettgers et al. (1935), Cook (1951), Clear and Hay (1973), and Kondo et al. (1974).

Although the results presented research by Mohr and others correlate compressive strength with RCPT results, this is often not the case. The precision and bias portion of ASTM C1202 highlight the prevalent variability in this test method. Variance has been found to be 12.3% for a single operator, and 18.0% for multilaboratory single test results. These values result in results found by testing of different cylinders from the same batch being 34% for a single operator and 42% - 51% for multilaboratory tests. (ASTM 2017). Table A.1 illustrates the RCPT results (in coulombs) associated with different levels of chloride ion permeability and mixture characteristics (ASTM 2018).

Table A.1: Chloride permeability based on charge passed

Charge passed (coulombs)	Chloride permeability	Typical of
> 4,000	High	High water-cement ratio, conventional (≥ 0.6) PCC
2,000 – 4,000	Moderate	Moderate water-cement ratio, conventional (0.4 – 0.5) PCC
1,000 – 2,000	Low	Low water-cement ratio, conventional (< 0.4) PCC
100 – 1,000	Very Low	Latex-modified concrete Internally sealed concrete
< 100	Negligible	Polymer impregnated concrete Polymer concrete

Until recently, the RCPT was considered the most rapid way to determine concrete's tendency to allow chloride ion permeability. The emergence of new electrical tests provided a new, rapid means to evaluate concrete's permeability using electrical methods that take only minutes to obtain on a standard compressive strength testing cylinder. Methods of measuring electrical resistivity of concrete include the disc method, the Wenner method (Figure A.2), and the use of electrodes (among others). Additionally, with prior planning, resistivity measurements can be performed by embedding metal electrodes prior to casting. Alternatively, unplanned methods of field measurement can be performed without the use of embedded electrodes (Polder 2001). Due to the costs and time required for testing, these test methods are used less often as part of a quality control plan (Kessler et al. 2005).

Surface resistivity testing using the Wenner probe (Figure A.2) offers a vast improvement in time and cost, requiring minutes to obtain results, compared to the several days required to perform the RCPT (AASHTO 2015). Surface resistivity tests measure the electrical resistivity of water-saturated concrete, and can be used to evaluate a wide array of concrete characteristics (Morris et al. 1996). In fact, Polder's study was able to relate the likelihood of steel reinforcement corrosion to the resistivity of various concrete samples and structures (Polder 2001).

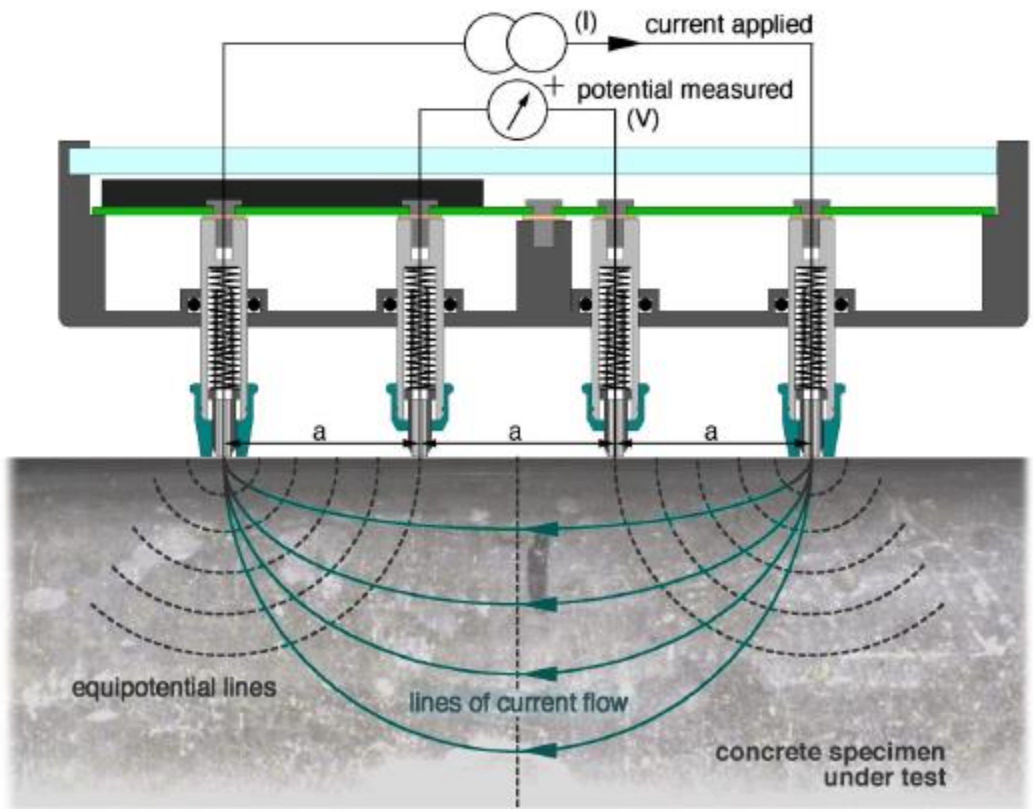

Figure A.2: Measurement of surface resistivity using the Wenner probe (from Proceq).

The test method for measuring surface resistivity is outlined in AASHTO T 358. A four-pin Wenner probe is used to pass an alternating current (AC) across the surface of a concrete structure or specimen. The two outer pins provide the current flow, and the potential difference is measured between the two inner pins. The results are presented in kΩ-cm (AASHTO 2017). Recently, results obtained through surface resistivity testing have been shown to correlate well with test results from RCPT (Rupnow and Icenogle 2012). Factors that influence surface resistivity results include moisture content, composition of the concrete, permeability, age, and temperature (Morris et al. 1996, Polder 2001, Presuel-Moreno et al. 2010, Liu et al. 2010). Table A.2, presented below, provides chloride ion penetration levels based on surface resistivity results.

Table A.2: Chloride ion penetration based on surface resistivity results

Chloride ion penetration	Surface Resistivity Test	
	100-by-200-mm (4-by-8-in.) cylinder (kΩ-cm) a = 1.5	150-by-300-mm (6-by-12-in.) cylinder (kΩ-cm) a = 1.5
High	<12	<9.5
Moderate	12 – 21	9.5 – 16.5
Low	21 – 37	16.5 – 29
Very low	37 – 254	29 – 199
Negligible	>254	>199
a = Wenner probe tip spacing		

The Rupnow and Icenogle study (2012) involved comparing 14 and 28-day average surface resistivity results with average 58-day RCPT results. The study found that the correlation between the results was strong, with a coefficient of determination (R^2) of 0.89. In this study, implementation of surface resistivity testing in lieu of RCPT estimated costs savings for LADOTD in the first year of implementation to be $101,000 in personnel costs. Estimated savings for contractor QC costs were estimated to be $1.5 million. These savings were primarily attributed to the reduction in man hours needed to perform testing, which was estimated to be approximately 4.1% of the man hours needed to perform RCPT. Cost of running the test had similar savings, with surface resistivity testing costing approximately 4.7% of costs associated with RCPT (Rupnow and Icenogle 2012).

Resistivity of concrete, disregarding outside factors, is influenced namely by the resistivity of the pore solution in the concrete's voids, the degree of saturation, and the volume and layout of the pore network (Spragg et al. 2011). Similar to surface resistivity, bulk resistivity testing is performed by passing an electrical current through a saturated concrete specimen. The key difference lies in the method of which the current is passed. Bulk resistivity is an electrical testing method which sends the current along the longitudinal axis of a concrete cylinder via plate electrodes (Polder 2001, Newlands et al. 2008). The standard for this testing method is outlined in ASTM C1760, "Standard Test Method for Bulk Electrical Conductivity of Hardened Concrete" (ASTM 2012).

Bulk conductivity is the inverse of resistivity, and because of this, there is a strong correlation between results found from these two tests. Experimentation to determine the correlation and variance between the tests was performed by Spragg et al. (2011). This testing program consisted of running both surface resistivity and bulk conductivity tests on twelve mixtures at three ages (28, 56, and 91 days). Data was also collected at 12 separate laboratories on the same mixtures at the same ages. The data collected by Spragg's laboratory had an R^2 value of 0.9997 (less than 2% difference) between measured resistivity and calculated cylinder resistivity. The coefficient of variance (COV) within Spragg's laboratory was 4.36%, and the COV across all participating labs was 13.22% (Spragg et al. 2011).

Formation Factor

The formation factor of concrete is a ratio of the self-diffusion coefficient to the microstructural diffusion coefficient, which characterizes pore structure (Snyder et al. 2000, Snyder 2001). This number is used to describe the layout of pores within the concrete. Geometry of the pores as well as how they are connected are also influencing factors to pore structure and subsequently the formation factor (Weiss et al. 2013, Weiss 2014, Weiss et al. 2016, Weiss 2019). Calculation of formation factor is done by dividing the electrical resistivity of the saturated concrete by the resistivity of the pore solution (Weiss et al. 2016). Research by also indicates that mixture characteristics such as composition of the cementitious materials used (namely alkali contents), degree of cement hydration at the point which the measurement is taken, and mixture proportions (AASHTO 2019).

The formation factor can alternatively be calculated by taking the inverse of the product of porosity and pore solution resistivity (Weiss et al. 2016). Saturated formation factors can be correlated to other electrical resistivity testing such as RCPT, surface resistivity, and bulk conductivity testing as shown in AASHTO PP 84 (Rupnow and Icenogle 2012). Sections 6.6.1.2 - 6.6.2.1 of AASHTO PP 84 outline recommendations for prescriptive and performance specifications related to the formation factor (AASHTO 2019). The prescriptive provisions of the specification recommend a formation factor value of greater than 500 if freeze/thaw conditions and deicers are negligible. A recommended value of greater than or equal to 1000 if the concrete will be subjected to deicing and freeze/thaw conditions. Table A.3 presents chloride ion penetrability levels associated with different formation factor, RCPT, and resistivity results. The performance portion of the specification allows the designer to choose acceptable saturated formation factor values as a function of the desired service life and exposure conditions of the concrete. These values are presented in Table A.4.

Table A.3: Prescriptive values for F factor, RCPT, and resistivity (AASHTO 2019)

Chloride ion penetrability	Greatest saturated formation factor	Lowest saturated formation factor	Minimum charge passed at 6 hours	Maximum charge passed at 6 hours	Greatest resistivity	Lowest resistivity
	-	-	Coulombs	Coulombs	Ω M	Ω M
High	500	-	4000	-	50	-
Moderate	1000	500	2000	4000	100	50
Low	2000	1000	1000	2000	200	100
Very low	20000	2000	100	1000	2000	200
Negligible	-	20000	0	100	-	2000

Table A.4: Performance values for saturated F factor (AASHTO 2019)

Exposure conditions	Saturated F factor limits	
	Desired service life (years)	
	25 – 35	> 35
Non-freeze-thaw and no deicers	> 500	> 1,000
Freeze-thaw and deicer exposure	> 1,000	> 2,000

Current electrical tests, including the surface resistivity method, are influenced by temperature, moisture, leaching, and degree of saturation, leading to repeatability issues attributed to the conditioning measures implemented (Snyder et al. 2000, Weiss et al. 2013, Qiao et al. 2018, Weiss 2019). One proposed method, the Bucket Test, seeks to eliminate variance caused by different conditioning methods. The Bucket Test is a procedure developed by researchers at Oregon State University, and the method includes measuring the electrical resistivity and mass change of 4in x 8in concrete cylinders that have been submerged in a solution that mimics that of typical concrete pore solution. An advantage of the Bucket Test over previously developed saturation tests (sealed samples, vacuum saturation, moist curing room) is that only matrix voids are saturated (i.e. gel and capillary), without affecting air voids (Weiss 2019). The Bucket Test also provides rapid results (5 days or less), and not only provides information about the formation factor, but also about the point of critical saturation, or nick point. The nick point is the point at which concrete has reached a critical saturation, at which freeze-thaw damage becomes an inevitable risk (Weiss et al. 2016).

Transport properties are addressed in AASHTO PP 84 Section 6.6, and relate to the concrete's tendency to allow penetration from various mediums. The *w/cm*, formation factor, and ionic penetration are identified as influencing factors for transport properties (AASHTO 2019). A *w/cm* of less than 0.50 (if concrete is not subjected to freeze/thaw conditions or deicers) or less than 0.45 (if freeze/thaw conditions or deicers are a risk) are suggested as prescriptive specification approaches in PP 84 Section 6.6.1.1. Target values for RCPT, resistivity, and F factor are specified in PP 84 Section 6.6.1.2, and are shown in Table A.3. The performance specification portion for transport properties identifies F factor values found using AASHTO T 358 or AASHTO TP 119, "Standard Method of Test for Electrical Resistivity of a Concrete Cylinder Tested in a Uniaxial Resistance Test" depending on desired ionic penetration depth over the desired service life (AASHTO 2015, 2019).

A.3 Statistical Techniques Supporting Analysis to Link Material Characteristics and Early Age Test Data to Field Performance

The need for advances in predicting the long-term performance of a mixture design has prompted several states to direct funds to research projects on creating performance related standards as well as performance prediction modeling. In this section, approaches for that modeling will be presented, along with information on what is needed to create these models.

A.3.1 Types of Variables

Within the datasets used for this study, there are many different variables available. These include (but are not limited to) the following:

- Materials used in the mixture design, and the respective amounts of each
- Particular material types (for example, different types of pozzolans)
- Names of suppliers of mixture materials
- Early age test results, such as air content, slump, compressive strength, and/or flexural strength
- Measurements of either the bridge deck or the pavement section, such as length, width, etc.
- Information about the location, structure type, and main material type of bridges
- Information about the location, age, and rating of pavement sections

Variables used in the modeling process can be divided into different types. *Continuous* (also known as *interval* or *quantitative*) variables take on any value within the range, as they are measured on a smooth scale rather than a stepped scale. The precision of the data is limited to the measuring equipment, not the method of collecting it. Variables in the mixture design data set that fit this category include cement content, aggregate content, and others. Discrete variables can be numbers or labeled as numbers, but there is no smooth transition from category to category or value to value. They can also denote or describe non-numeric *qualitative* values, such as material type, bridge system, superstructure type, etc. (all categories in the BMS Network Master). The distinction between continuous and discrete variables is not always apparent, especially in cases were the difference between discrete values is small and where the continuous variables are cut at certain limits (Tabachnick and Fidell 2007).

A.3.2 Types of Models

The goal of the modeling process in this study is to determine relationships between variables – i.e., between mixture design characteristics, early age performance test results, and long term field performance. These relationships could exist between the amount of impact fly ash has on slump, the individual components of the mixture design and specific measures of long term performance inspection values, or any number of other potential relationships. Therefore, modeling procedures that compare the individual variables, or fields, are useful in this research. The first modeling type to be used in this study is canonical correlation, which shows the relationship between large sets of variables. Regression modeling can then be used to get more specific on the impact of a single field (Tabachnick and Fidell 2007).

A.3.3 Identification of Significant Predictor Variables

Due to the number of variables present in the mixture design and early age data sets, determining the significant predictor variables, and their individual weights of importance, is necessary for the overall analysis of the data. To do this, *Canonical Correlation* can be used to find the relationship between sets of data. Canonical correlation is a method for determining the relationship between two sets of variables. In this technique, if one set is known to be the predictor or independent set, and the other is known to be the criterion or dependent set, then the goal is to determine how the first effects the second. Considering the following two equations:

$$W_1 = a_{11}X_1 + a_{12}X_2 + \cdots + a_{1p}X_p \quad \text{(A.1)}$$

$$V_1 = b_{11}Y_1 + b_{12}Y_2 + \cdots + b_{1p}Y_p \quad \text{(A.2)}$$

Where:
W_1 = Linear combination of the X variables
V_1 = Linear combination of the Y variables
C_1 = The correlation between W_1 and V_1 (canonical correlation)

The goal of the two sets of equations is to find the values for $a_{11}, a_{12}, \dots, a_{1p}$ and $b_{11}, b_{12}, \dots, b_{1p}$ such that C_1 exists at its maximum value. This step is then repeated for W_2 and V_2, and so on and so forth until W_m and V_m. In summary, the objective of canonical correlation is to identify the m set of canonical variates, (W_1, V_1), (W_2, V_2),… (W_m, V_m) such that the corresponding canonical correlations, C_1, C_2,…,C_m are maximized (Sharma 1996).

The next step is determining the statistical significance of the canonical correlations. The null and alternative hypotheses for assessing this significance are:

$$H_o: C_1 = C_2 = \cdots = C_m = 0 \tag{A.3}$$
$$H_a: C_1 \neq C_2 \neq \cdots \neq C_m \neq 0 \tag{A.4}$$

To test this hypothesis (the null states that all canonical correlations are equal to zero), a number of test statistics can be used. However refined the final values from the canonical correlation are, they not useful unless they can be practically interpreted. This is a common issue associated with use of this method. The results have been described as "…often mathematically elegant but uninterpretable" (Tabachnick and Fidell 2007), which makes them difficult for inclusion in this study. Because of this, the results procured from the data while using this method are not the most directly useful. While it is a very useful process and can be done by hand, the number of computations lead it to be impractical to be completed by hand. Several software packages are available that can perform canonical correlation, such as SAS, SPSS, MATLAB, and SYSTAT.

A.3.4 Regression Analysis

Regression analysis is the use of statistical methods to determine the relationship (if one exists) between a dependent variable and several corresponding independent variables. For a simple linear regression model, only one independent variable exists, meaning that the dependent variable changes at a constant rate as the independent variable changes. This can be shown as the equation of a straight line. Ideally, this line will "fit" the data in the scatter plot, but not everything will fall perfectly on the line. To demonstrate this, the sample coefficient of determination R^2 is used. R^2 indicates how well the equation fits the data, and ranges in value from 0 to 1 (with higher values indicating a better fit) (Dowdy et al. 2004).

$$Y' = A + BX \tag{A.5}$$

Where: *Y'* = Predicted value (dependent variable)
A = The value of *Y* when *X* is equal to zero
B = The slope of the best-fit line
X = Value from which *Y'* is predicted (independent variable)

In order to solve for the equation, values for both *A* and *B* must be determined. *B* is the bivariate regression coefficient, and is the ratio of the covariance of the variables (*X* and *Y*) and the variance of the one from which predictions are made (*X*), as well as the slope of the best fit line. Once *B* has been determined using equation A.6, the value of the x-intercept, *A*, can be found using equation A.7 (Tabachnick and Fidell 2007).

$$B = \frac{N \sum XY - (\sum X)(\sum Y)}{N \sum X^2 - (\sum X)^2} \tag{A.6}$$

Where: *B* = Bivariate regression coefficient
X = Independent variable
Y = Dependent variable

$$A = \bar{Y} - B\bar{X} \tag{A.7}$$

Where: *A* = X-intercept
X = Mean of the predicted variable
Y = Mean of the predictor variable

Multiple regression is an extension of the principles of simple linear regression, with the largest difference being the use of multiple independent variables instead of just one. Instead of one single bivariate regression coefficient, each independent variable has their own, in an effort to cause *Y* to be as accurate as possible. As such, the regression equation is represented by the following equation:

$$Y' = A + B_1X_1 + B_2X_2 + \cdots + B_kX_k \tag{A.8}$$

Where: *Y'* = Predicted value (dependent variable)

A = The value of Y when all X values equal zero
B_n = Regression coefficient for the n-th variable
X_n = n-th independent variable
k = Number of independent variables

With an inflated number of variables, the relationship between the individual variables can cause portions of the equation to become redundant. This is known as multicollinearity, where one or more of the independent variables are highly correlated with each other. If not addressed, multicollinearlity is problematic in the final model because these variables represent redundant information and are not all needed. If left in the final model, they will inflate the size of the error terms, and can weaken the analysis (Tabachnick and Fidell 2007). To measure the scale of the impact of multicollinearity, a variance inflation factor (VIF) should be computed for each independent variable during the regression analysis. VIF is calculated using the following equation:

$$VIF_j = \frac{1}{1 - R_j^2} \tag{A.9}$$

Where: VIF_j = Variance Inflation Factor (for variable j)
R^2_j = Coefficient of Determination (for variable j)

In this equation, the coefficient of determination (R^2) is determined from the regression of each independent variable on the other independent variables that are being tested (Rawlings et al. 1998). Variables that show high correlation with other variables included in the model must be removed one at a time, rerunning the linear regression analysis each time to generate updated VIF values for the remaining independent variables. With a commonly used threshold of 10 as the target value (Rawlings et al. 1998), the previously mentioned step should be performed until all VIF values are below the threshold. In some cases, the regression analysis may call for the removal of seemingly important variables, labeling them with higher VIF values. While statistically it may make sense to remove them, knowledge of typical concrete property influencers may contradict this. Accordingly, at a certain level, multicollinearity can be ignored. There are at least three situations in which a high VIF value is not an issue and can be ignored (Allison 2012):

1. The variables with high VIFs are control variables, and the variables of interest do not have high VIFs.
2. The high VIFs are caused by the inclusion of powers or products of other variables.

The variables with high VIFs are indicator (dummy) variables that represent a categorical variable with three or more categories. By these guidelines, any time that a variable was slated for removal but still fit one of these three situations, can be kept in the dataset.

In regression modeling with noisy, high-variability data, the R value (and thus the R-squared value) may be lower than expected. R-squared represents the variability, or the scatter around the regression line. Low R^2 values become an issue when trying to create precise predictive equations, but these low R^2 values do not necessarily mean the variables are unrelated. Even in cases where the R-squared is low, low P values can still indicate the relationship between the significant predictors and the response variable (Minitab Blog Editor 2014). Therefore, in cases where predictive equations are not necessary, but determination of variable relationships are, low R-squared values are acceptable, but attaining the highest R^2 value possible will still be preferable.

While regression can be performed by hand, when analysis is being performed with a large set of data it would become tedious, and the potential for mistakes is higher. As such, there are several computer programs that can aid in these calculations and can provide a best-fit equation for the data. Common computer programs for this include Minitab, MATLAB, SAS, SPSS, and SYSTAT. Each program has its benefits (from more output data to better user interface), but all can handle large amounts of data when performing regression analysis.

A.3.5 T-Test

The t-test is a statistical method for determining if the difference in the mean of two groups are significantly different from each other; i.e., is the amount of difference significant when considering the sample size and the standard deviation of each group. In this case, the null and alternative hypothesis are written as the following equations:

$$H_0 : \mu_1 = \mu_2 \tag{A.10}$$

$$H_a : \mu_1 \neq \mu_2 \tag{A.11}$$

Where μ_1, μ_2 are the means of the respective groups of data.

To test the null hypothesis, values for the t-value must be computed with the information provided in the dataset. There are two potential equations for finding the t-value that will be used in this study: one for equal variance, also known as the Student's t (Eq. A.12), and one for unequal variance, also known as Welch's t (Eq. A.14) (NIST/SEMATECH 2012).

Student's t:

$$T = \frac{\overline{Y}_1 - \overline{Y}_2}{s_p\sqrt{\frac{1}{N_1} + \frac{1}{N_2}}} \tag{A.12}$$

Where:

$$s_p = \sqrt{\frac{(N_1 - 1)s_1^2 + (N_2 - 1)s_2^2}{N_1 + N_2 - 2}} \tag{A.13}$$

Welch's t:

$$T = \frac{\overline{Y}_1 - \overline{Y}_2}{\sqrt{\frac{s_1^2}{N_1} + \frac{s_2^2}{N_2}}} \tag{A.14}$$

Where: $N_1, N_2 =$ Sample sizes
$\overline{Y}_1, \overline{Y}_2 =$ Sample means
$s_1^2, s_2^2 =$ Sample variances

At a significance level "α", the null hypothesis that the two means are equal is rejected if $|T| > t_{1-\alpha/2}$ at the calculated degree of freedom (υ). This degree of freedom is calculated using Eq. A.15 if the variances are equal, and using Eq. A.16 if the variances are not equal.

$$\upsilon = N_1 + N_2 - 2 \tag{A.15}$$

$$\upsilon = \frac{\left(\frac{s_1^2}{N_1} + \frac{s_2^2}{N_2}\right)^2}{\frac{\left(\frac{s_1^2}{N_1}\right)^2}{(N_1 - 1)} + \frac{\left(\frac{s_2^2}{N_2}\right)^2}{(N_2 - 1)}} \tag{A.16}$$

Where: $N_1, N_2 =$ Sample sizes
$s_1^2, s_2^2 =$ Sample variances

If the null hypothesis is rejected, the difference in the means between the groups is considered to be significant. If the final result is to fail to reject the null hypothesis, the difference between the means is not large enough to be significant at the given confidence level.

Calculations to determine the t-value, as well as the degree of freedom, can be performed using a variety of computer programs, with Microsoft Excel being used for this study. Variance of the mean can be assumed as equal if the data allows for it, but for this study the variance will be checked and not assumed. This can be performed in Minitab, as a command under the ANOVA family of commands.

A.3.6 Group Difference Modeling

Similar to the results obtained from the t-test, performing group comparisons using One-Way ANOVA allows for testing the difference in the mean between pairs of groups. The error rate for the comparison can range between 0.001 (99.999% confidence) and 50 (50% confidence) (Minitab 2018). Within the Minitab software, there are four options for data groups with assumed equal variance (Tukey, Fisher, Dunnett, and Hsu MCB), and one for data groups with unequal variance (Games-Howell) (Devore 2011). For this research, the following will be used:

- Dunnett's Method: Used to create confidence intervals for group differences between means of each factor and the mean of a singular control group. If an interval contains zero, then there is no significance in the difference between the means. This method assumes equal variance between the data groups.
- Games-Howell: The only available group comparison method available on Minitab for groups with equal variance not assumed, the method provides a similar confidence interval between the means. If that interval contains zero, the there is no significance in the difference between the means.

These methods provide not only the written confidence intervals, but also graphs to allow for easy interpretation. Since these methods will be used in conjunction with the t-test, and therefore are a secondary check rather than the main analysis method, the mathematical derivations of these methods will not be discussed.

A.3.7 Review of Previous Studies

In a field investigation performed at The Pennsylvania University, researchers used field data to identify factors that contributed to (increasing or reducing) early age cracking the state bridge decks, as well as assessing the long-term durability effects of those cracks. This study was performed by combining data from older bridges as well as from newly constructed decks. In attempting to establish trends between various mixture design factors and early-age cracking of various concrete classes, it was determined that higher cementitious material content results in higher probability of cracking in early-age concrete (as displayed by Class AAA bridge decks, which are high cement level mixtures). Use of one-way analysis or variance (ANOVA) also implied that 7-day compressive strength significantly affected the crack density at the 0.05 significance level. Higher compressive strengths at the time tended to lead to higher likelihood of cracking (Manafpour et al. 2016).

A.4 Specification Provisions for Resistivity

A number of states have begun to explore the benefits of surface resistivity as an electrical resistance test to predict concrete durability. Louisiana Department of Transportation and Development (LADOTD) is one of the first states to specify the use of surface resistivity for acceptance of concrete. LADOTD 2016 Standard Specifications for Roads and Bridges manual requires surface resistivity testing on all major structural class concrete (LADOTD 2016). Louisiana's testing method, DOTD TR 233, "Surface Resistivity Indication of Concrete's Ability to Resist Chloride Ion Penetration" is similar to the test method outlined in AASHTO T 358 (AASHTO 2017). LADOTD requires a minimum surface resistivity of 22 kΩ-cm at 28 days using 4in x 8in cylinders for Class A1 mixtures (4,500 psi mass concrete), and 56 days for class A2 (6,500 psi mass concrete) and A3 (9,000 psi mass concrete) mixtures. Table A.5 shows LADOTD's chloride ion penetrability ratings based on surface resistivity test results. It should be noted that LADOTD TR 233 requires an adjustment factor of 1.1 if samples are cured in limewater due to the average 10% reduction in resistivity that results (LADOTD 2018). This provision is consistent with AASHTO T 358. If surface resistivity results do not meet these requirements, LADOTD reserves the right to withhold a percentage of contract price based on results or require removal and replacement of the concrete, as shown in Table A.6.

Table A.5: LADOTD chloride ion penetrability and associated surface resistivity values (LADOTD 2018)

Chloride ion penetration	Surface resistivity test	
	4 inch X 8 inch cylinder (kΩ-cm) a=1.5	6 inch X 12 inch cylinder (kΩ-cm) a=1.5
High	< 12.0	< 9.5
Moderate	12 .0 – 21.0	9.5 – 16.5
Low	21.0 – 37.0	16.5 – 29.0
Very low	37.0 – 254.0	29.0 – 199.0
Negligible	> 254.0	> 199.0
a = Wenner probe tip spacing		

Table A.6: LADOTD acceptance and payment schedules associated with surface resistivity values (LADOTD 2018)

Surface resistivity per lot, kΩ-cm (28 to 31 days: A1 mixes) (56 to 59 days: A2 & A3 Mixes)	
Class A1, A2, A3, S, P1, P2, P3, S & MASS(A1,A2,A3)	Percent of contract price
22.0 & above	100
20.0 – 21.9	98
18.0 – 19.9	90
below 18.0	50 or remove and replace

A review of standards currently implemented (or being proposed for implementation) by a number of state highway agencies was performed to determine provide insight into currently utilized specification targets for RCPT and surface resistivity. In total, 12 states currently utilizing (or proposing use of) RCPT and/or surface resistivity in their specifications were identified at the time of this work. The implementation level of these specifications ranged from project special provisions and to fully implemented specifications. A summary table of the states that include requirements for paving and bridge concrete mixtures is presented in Table 5.2 (in body of report). A table summarizing all state RCPT and surface resistivity requirements is shown in Table A.7.

Table A.7: Summary of agency specification RCPT and surface resistivity requirements

State/ Standard	**RCPT Specification**			**Resistivity Specification**		
	Concrete Type	**Require-ment (coulombs)**	**Age (days)**	**Concrete Type**	**Require ment (kΩ-cm)**	**Age (days)**
Florida DOT special circumstances. Implemented AASHTO T 358 in January 2017	-	-	-	Ternary blend - extremely aggressive environment	> 29	28
	-	-	-	Ternary blend - moderately aggressive environment	17 - 29	28
	-	-	-	Ternary blend - slightly aggressive environment	< 17	28
	-	-	-	Structural Concretes: Class IV, V, V (special), VI with use of silica fume, ultrafine fly ash, or metakaolin	≥ 29	28
	-	-	-	Ultra-high performance repair material for vertical surfaces	≥ 22	28
	-	-	-	Special fillers for cathodic protection	Can be 15 or less	28
	-	-	-	Special fillers for non-cathodic protection	≥ 22	28
Kansas DOT special provisions	Concrete classified as high chloride risk	> 4000	28	Concrete classified as high chloride risk	< 7	28
	Concrete classified as moderate chloride risk	2000 - 4000	28	Concrete classified as moderate chloride risk	7 - 13	28

	Concrete classified as low chloride risk	1000 - 2000	28	Concrete classified as low chloride risk	13 - 24	28
	Concrete classified as very low chloride risk	100 - 1000	28	Concrete classified as very low chloride risk	24 - 190	28
	Concrete classified as negligible chloride risk	0 - 100	28	Concrete classified as negligible chloride risk	> 190	28
Louisiana DOTD structural class concrete	-	-	-	Structural Concretes: Class A1, A2, A3; Prestressed Concretes: Class P1, P2, P3; CIP Structural: Class S	> 22	28
	-	-	-	Structural Mass Concretes: Class Mass A1, A2, A3	> 22	56
Montana DOT	-	-	-	Mix trial batches for Class "Deck" (superstructures, deck slabs, barriers) and "Overlay S-F" (silica fume overlays)	> 21	28
New Hampshire DOT (for bridge decks, abutment backwalls) (SRT = surface resistivity test in kΩ-cm)	-	-	-	Class AA (Pay factor 1.05 - 0.06 (10 - SRT))	≥ 5 and ≤ 10	56
	-	-	-	Class AA (Pay factor 1.05)	> 10 and ≤ 35	56
	-	-	-	Class AA (Pay factor 1.05 + 0.0004347 (150 - SRT))	> 35 and ≤ 150	56
	-	-	-	Class AA (Pay factor 1.0)	> 150	56
	-	-	-	Prestressed and member concrete	> 15	56
New Jersey DOT	-	-	-	HPC Design and Verification Requirements	≥ 36	56
	-	-	-	HPC Acceptance Requirements	≥ 19	56
	-	-	-	Concrete classified as high chloride risk	< 9	56
	-	-	-	Concrete classified as moderate chloride risk	9 - 20	56
	-	-	-	Concrete classified as low chloride risk	20 - 48	56
	-	-	-	Concrete classified as very low chloride risk	48 - 817	56
	-	-	-	Concrete classified as negligible chloride risk	> 817	56
New York DOT proposed thresholds for design mix performance criteria where specified	-	-	-	Superstructures and substructures	> 24	28
	-	-	-	Footings, piles, drilled shafts, underground applications, sign bases, etc.	> 14	28
	-	-	-	Pavements, sidewalks, gutters, curbs, barriers, headwalls, drainage elements, pipe inverts, maintenance repair	> 16.5	28
New York DOT performance engineered concrete mixtures for pavements based on application requirements	Pay factor - 100%	≤ 1000	28	Pay factor - 100%	≥ 37	28
	Pay factor - 87.5%	> 1000 and ≤ 1500	28	Pay factor - 87.5%	< 37 and ≥ 27	28
	Pay factor - 75%	>1500 and ≤ 2500	28	Pay factor - 75%	< 27 and ≥ 19	28
	Reject concrete	>2500	28	Reject concrete	< 19	28
Rhode Island DOT concrete pre-qualification requirements	Structural and prestressed/ precast elements: Class HP	≤ 2000	28	Structural and prestressed/ precast elements: Class HP	≥ 15	28
	Mass Concrete: Class MC²	≤ 3000	28	Mass Concrete: Class MC²	≥ 15	28
	Structural and prestressed/	≤ 1000	28 day accele	Structural and prestressed/ precast elements: Class HP	≥ 21	56

	precast elements: Class HP		rated cure			
	Mass concrete: Class MC²	≤ 1500	28 day accele rated cure	Mass concrete: Class MC²	≥ 21	56
Texas DOT	Pavement, structures, and other concrete construction	< 1500	56	-	-	-
	Pavement, structures, and other concrete construction	< 1500	28 day accele rated cure	-	-	-
UTAH DOT mix requirements	-	-	-	Class AA (LSF), AA (LS), AA (ES). (AA= bridge decks, LS= low shrinkage, LSF= low shrinkage with fibers, ES = Early strength. AA(LS) used for bridge decks & approach slabs, AA (AE) = other structural elements)	Must have "low to negligibl e risk" accordin g to AASHT O T 358	
Virginia DOT design maximum lab permeability Note: [XXXX]* = design maximum lab permeability over tidal waters	A5 prestressed and other special designs	1500 [1500]*	28	-	-	-
	A4 general	2500 [2000]*	28	-	-	-
	Low shrinkage A4 mod	2500 [2000]*	28	-	-	-
	A4 post & rails	2500 [2000]*	28	-	-	-
	A3 general	3500 [2000]*	28	-	-	-
	A3a paving	3500 [3500]*	28	-	-	-
	A3b paving	3500 [3500]*	28	-	-	-
	B2 massive or lightly reinforced	NA [NA]*	28	-	-	-
	C1 massive unreinforced	NA [NA]*	28	-	-	-
	T3 tremie seal	NA [NA]*	28	-	-	-
	latex hydraulic cement concrete overlay	1500 [1500]*	28	-	-	-
	silica fume, silica fume/class f fly ash or silica fume/slag concrete overlay	1500 [1500]*	28	-	-	-
	class F fly ash or slag overlay	1500 [1500]*	28	-	-	-
West Virginia DOT supplemental specs	Class S-P concrete (self-consolidating for precast/ prestressed applications	≤ 2000	28	-	-	-
	Class S-P concrete (self-consolidating for precast/ prestressed applications	≤ 1500	56	-	-	-
	Bridges	< 750	90	-	-	-

As shown in Table A.7, the type of specification and related requirements vary greatly state to state. For both RCPT and surface resistivity, test dates include 28 and 56 day requirements, as well as West Virginia's DOT including a 90 day RCPT bridge requirement (WVDOT 2016). The six states that have RCPT requirements are as follows: Kansas (KDOT 2015), New York (NYDOT 2018), Rhode Island (RIDOT 2018), Texas (TDOT 2004), Virginia (VDOT 2016), and West Virginia (WVDOH 2016). Of these states, four states (Rhode Island, Texas, Virginia, and West Virginia) specify specific limits which must be met for certain classes of concrete. Kansas and New York specify RCPT based upon the application of the concrete, with New York including a pay factor adjustment if the desired values are not met. Texas and West Virginia are the two states that have specifications at later ages, with Texas specifying at 56 days (unless an accelerated cure is used). West Virginia includes separate requirements at 28 and 56 days for "Class S-P" concrete, and a 90 day requirement for bridge applications. It should be noted that Kansas's specification uses the same values set forth in the AASHTO and ASTM standards for RCPT.

Nine out of the twelve states include some form of a resistivity requirement at either 28 or 56 days. These states include Florida (FDOT 2018), Kansas (KDOT 2015), Louisiana (LADOTD 2016), Montana (MDOT 2014), New Hampshire (NHDOT 2016), New Jersey (Nassif et al. 2015), New York (NYDOT 2018), Rhode Island (RIDOT 2018), and Utah (UDOT 2018). Kansas and New York specify resistivity in the same manner as RCPT, based on application requirements with New York including a pay factor. Florida, Kansas, Montana, New York, and Utah require various surface resistivity targets at 28 days, while New Hampshire and New Jersey set their requirements at 56 days. Both Louisiana and Rhode Island have separate requirements for typical and mass concrete applications at both 28 and 56 days.

The most aggressive specifications for both RCPT and surface resistivity are for concretes with one of three characteristics or service considerations: specifications requiring target values be met at later ages, concrete utilizing SCMs, and concrete serving in high chloride risk environments. For RCPT, the three most rigorous requirements are Virginia's 28 day requirements for overlays with latex or SCMs (1,500 coulombs), New York's 28 day PEM pavement requirements (1,000 coulombs), and West Virginia's 90 day bridge specification (750 coulombs). Likely the most difficult to achieve resistivity specifications are Florida's 28 day requirements for ternary blend concretes serving in extremely aggressive environments and structural concretes (29 kΩ-cm), New Jersey's 56 day high performance concrete design & verification requirement (36 kΩ-cm), and New York's PEM pavement requirement at 28 days (37 kΩ-cm).

A.5 Specification Provisions for Early Age Opening to Traffic

As specifications for roads and bridges were developed around the United States, most were originally based on other states specifications and agency experiences instead of engineering analysis. This practice was generally true until around the mid-1990s, when specifications for opening roads to traffic began to be based on engineering properties (Cole and Okamoto 1995). Although agency experience heavily guided specification development for many years, recently many states across the U.S. utilize improved specification provisions, and some address the impact of SCMs on concrete performance by including provisions for slower strength gain and improved performance targets. However, the standard specifications from NCDOT for roads and bridges only includes substitutional requirements, as the NCDOT manual states that up thirty percent of cement can be substituted with SCMs at a one to one ratio (NCDOT 2018).

The primary specification requirement that could potentially impact a contractor's ability to move forward with a project after concrete placement is the minimum required concrete strength that must be achieved in order to open roads and bridges to traffic along with handling construction traffic and equipment. Early age strength requirements are essential to consider when improving specifications because PEM mixtures should allow adequate strength gain to provide the required strength while also allowing contractors to progress at a reasonable rate. Ultimately, contractors need to feel comfortable utilizing PEM concrete mixtures, which will often utilize SCMs to meet performance test targets.

To aid with quality control and quality assurance, improved methods for evaluating concrete placement are essential to ensure specified compressive strength is reached for opening pavements to traffic. Maturity concepts include non-destructive testing to estimate in-place concrete performance (Kosmatka and Wilson 2016, Hanson 2019, Garber 2019). During the Fall 2019 National Concrete Consortium meeting, maturity methods to assist with monitoring of early age strength gain was a key topic for presentation and discussion. Specifications from states that conduct mostly cold weather, and mass concrete placement were highlighted, and each put emphasis on monitoring the temperature during these types of conditions to ensure the concrete will perform (reaching specified compressive strength) as expected (Garber 2019). Advantages to improving maturity evaluation systems include increased safety, improved construction methods, efficiency, and consistency (Garber 2019). Maturity systems should include field early strength predictions, schedule of sawing and curing activities (as these directly affect concrete strength gain once it is set), and a plan if cracking occurs (Garber 2019).

Maturity concepts involve a maturity-strength curve produced by contractors to use to estimate in place strength and compare with actual strength using laboratory testing (Hanson 2019). Contractors utilizing this system will be able to monitor how well the mixture that was delivered and placed compares to the mixture design using sensors, ensuring the concrete placed is performing properly (Garber 2019). Several technologies were discussed in this presentation to aid contractors with maturity evaluations including embedded and non-embedded Bluetooth sensors, thermocouple systems, and combination systems, all of which improve the quality of concrete placement and methodology (Garber 2019).

Specifically, Iowa Department of Transportation (IowaDOT) has implemented a maturity system as discussed for in a NCC presentation entitled, “Maturity for Opening PCC Pavements: Iowa Experience” by Todd Hanson (Hanson 2019). This involved creating maturity curves for flexural and compressive strength in order to use for strength and temperature validation (Hanson 2019). Although, getting contractors to cast test specimens and pay for more expensive field maturity devices, Iowa allowed contractors to use curve validations instead of developing new curves along with allowing minor changes to mixtures giving contractors some flexibility. This method has reduced construction times and costs (benefitting the owner, contractor, and the public), along with accelerating staged construction since roads can be opened earlier (Hanson 2019).

North Carolina Department of Transportation

NCDOT has specified overall standard requirements for concrete that include slump, air content, compressive, and flexural strength at 28 days as shown in Table A.8 (NCDOT 2018). These standards do not provide performance targets for concrete mixtures, or modified targets for mixtures containing SCMs. As far as standards specific to SCMs, NCDOT limits the use of fly ash as a substitution for cement up to 30 percent at a one pound of fly ash to one pound of cement as stated in Section 1000-3. This is a recent change to specifications, which formerly limited fly ash replacement rates to 20% at a substitution of 1.2 pounds of fly ash to each 1.0 pound of cement replaced (NCDOT 2012). In the same section, NCDOT specifies the use of blast furnace slag as a substitute for cement can be used up to 50 percent pound for pound. Also, it is stated in Section 1024-5 that fly ash must meet ASTM C618 for Class F or Class C, and loss on ignition cannot exceed four percent. In addition, Class C fly ash cannot be used in portland cement concrete that has alkali content of 0.4 percent (NCDOT 2018).

Table A.8: NCDOT Requirements for Concrete Mixtures (NCDOT 2018)

Class of Concrete	Min. Comp. Strength at 28 days	Maximum Water-Cement Ratio				Consistency Max. Slump		Cement Content			
		Air-Entrained Concrete		Non Air-Entrained Concrete		Vibrated	Non-Vibrated	Vibrated		Non-Vibrated	
		Rounded Aggre-gate	Angular Aggre-gate	Rounded Aggre-gate	Angular Aggre-gate			Min.	Max	Min	Max.
Units	*psi*					*inch*	*inch*	*lb/cy*	*lb/cy*	*lb/cy*	*lb/cy*
AA	4,500	0.381	0.426	-	-	3.5	-	639	715	-	-
AA Slip Form	4,500	0.381	0.426	-	-	1.5	-	639	715	-	-
Drilled Pier	4,500	-	-	0.450	0.450	-	5-7 dry 7-9 wet	-	-	640	800
A	3,000	0.488	0.532	0.550	0.594	3.5	4	564	-	602	-
B	2,500	0.488	0.567	0.559	0.630	1.5 ma-chine placed 2.5 hand place	4	508	-	545	
Sand Light-weight	4,500	-	0.420	-	-	4	-	715	-	-	-
Latex Modified	3,000 7 day	0.400	0.400	-	-	6	-	658	-	-	-
Flowable Fill excavatable	150 max. at 56 days	as needed	as needed	as needed	as needed	-	Flow -able	-	-	40	100
Flowable Fill non-excavatable	125	as needed	as needed	as needed	as needed	-	Flow -able	-	-	100	as need-ed
Pavement	4,500 design, field 650 flexural, design only	0.559	0.559	-	-	1.5 slip form 3.0 hand place	-	526	-	-	-
Precast	See Table 1077-1	as needed	as needed	-	-	6	as need-ed	as need-ed	as need-ed	as need-ed	as need-ed
Prestress	per contract	See Table 1078-1	See Table 1078-1	-	-	8	-	564	as need-ed	-	-

The change in the specification to allow an increased (30%) SCM content will likely impact early age performance. Class A and pavement mixtures are the primary focuses of the NCDOT due to the higher strength requirements of each class ensuring road and bridge safety as roads and bridges are expected to reach 4,500 psi by 28-days and at least 3,000 psi prior to opening roads to traffic which will be further discussed later in this chapter (NCDOT 2018).

Although NCDOT has acceptable values for performance of concrete at a given age, contractors are interested in requirements for opening pavements to construction and regular traffic. For existing structures traffic must be maintained and the posted load limits must be observed. The NCDOT standard specification 420-20 “Placing Load on Structure Members” states that structures must cure for at least 7 days prior to loading. In addition to curing, construction equipment and vehicles cannot load structures until 28- day strength is reached or a compressive strength of 3,000 psi is obtained. To remove formwork for bridge decks, beams, and girders a compressive strength of 3,000 psi is required. For regular traffic, structural pavements must have a minimum flexural strength of 650 psi and a minimum compressive strength of 4,500 psi within 28 days (NCDOT 2018).

These requirements do not include cold weather concrete placement. If concrete is placed in weather below 35 degrees Fahrenheit, and contains fly ash or GGBFS, the concrete must be insulated and protected for seven days prior to loading. Placing mixtures in cold weather, mixtures containing fly ash require a mixture of 572 pcy of cement and at least 172 pcy fly ash for insulation. Concrete mixtures including GGFBS require a mix of 465 pcy of cement and 250 pcy of GGFBS for insulation as stated in Section 420-7 (C) of the NCDOT standard specifications for roads and bridges (NCDOT 2018).

In section 105-5, NCDOT presents equipment load restrictions for bridges as shown in Table A.9. Equipment should not exceed these maximum limits along with listed maximums for existing structures.

Table A.9: NCDOT Equipment Load Restrictions for Bridges (NCDOT 2018)

Property	Maximum Load in Pounds
Axle load	36,000
Axle load on tandem axles	30,000
Gross load	90,000

A number of other states have standard specifications provisions for mixtures containing SCMs, including mixtures containing relatively high SCM contents. These are discussed subsequently in order to compare with NCDOT specification provisions, and identify specification approaches that could be used to help modify NCDOT specifications to better address PEMs.

Louisiana Department of Transportation and Development (LaDOTD)

LaDOTD Standard Specifications for Roads and Bridges states in Section 901.08 "Composition of Concrete" for all concrete mixes use of fly ash is limited to a maximum of 25 percent weight of cement for concrete pipe, 20 percent weight of cement for minor structures and pavements, and 15 percent weight of cement for structural concrete depending on the class of concrete. These standards state that records of any concrete material (fly ash, cement, micro-silica, granulated blast furnace slag, etc.) deliveries must be tracked by the contractor, and require trial mixes to determine performance and compatibility of the concrete materials (LaDOTD 2016).

Along with trial mixtures, the contractor is expected to test and send results for slump, unit weight, air content, set times, compressive strength and flexural strength for pavements at 3, 7, and 28 days for state verification. In addition, all structural concrete with the exception of minor structures, must use surface resistivity to determine permeability per DOTD TR 233 standard. Also, LaDOTD set standards specifically for fly ash in structural concrete in Section 901.08.2 "Cementitious Material Substitution." For instance, for structural binary mixtures (combination of portland cement and one additional cementitious replacement, such as fly ash or GGBFS, the maximum permissible substitution rate for fly ash is 30 percent and 50 percent for GGBFS. For ternary concrete mixtures (combination of portland cement and two additional cementitious replacements including fly ash class C and/or F, and GGBFS), the maximum permissible substitution rate is different depending on the Type of cement used. LaDOTD states, "…for ternary mixtures containing Type I, II, III, 1L portland cement, the maximum substitution rate is 70 percent of cement" and "using Type IP or IS portland cement, the maximum substitution rate is 40 percent" (LaDOTD 2016).

Compressive strength required for construction loads are explained in Section 601.03.13 of the LaDOTD Standard Specifications for Roads and Bridges Manual. For instance, heavy equipment is not permitted on pavements until a minimum compressive strength of 3,000 psi is reached. Also, traffic is not permitted on concrete pavements until 14 days after setting or test specimens made in accordance with standard 601.03.7 have reached a compressive strength of 3,000 psi tested in accordance with DOTD TR 230 or a flexural strength of 550 psi tested in accordance with AASHTO T 97. Any concrete that is supporting formwork must reach 3,000 psi compressive strength prior to placing concrete as per Section 805.05.3. On the other hand, bridge deck concrete must reach a minimum of 4,000 psi before reinforcement, forms, concrete, or metal railings can be installed as per Section 810.03 (LaDOTD 2016).

Minnesota Department of Transportation (MnDOT)

MnDOT is unique because of newly advanced standards to improve overall concrete durability and longevity utilizing SCMs and PEMs. In 1996, the state released a new maximum water cement ratio of 0.40 and maximum cementitious value of six hundred pounds per cubic yard for concrete (Sutter et al. 2018). In addition, the standard was amended to include maximum substitution of fly ash for portland cement to 25 percent, which was not included prior to 2018. After these changes were made, research showed that pavements under this standard were smoother at a given year of pavement life, and the road condition deteriorated slower in comparison to pavements constructed prior to this water cement ratio standard (Sutter et al. 2018). Also, core samples from pavements under the new standard were tested using ASTM C457/C457M S*tandard Test Method for Microscopical Determination of Parameters of the Air-Void System in*

Hardened Concrete, and results indicated "on average, an increase of air content and improved air void system" allowing the concrete to have increased "resistance to freeze/thaw deterioration" (Sutter et al. 2018).

Table A.10 shows minimum curing periods, strength requirements, and methods for testing in-place concrete strength prior to loading structures with construction vehicles and equipment (with the exclusion of mass concrete structures) (MnDOT 2018). Construction equipment on pavements, loaded or empty hauling equipment is only permitted on the Permeable Asphalt Stabilized Stress Relief Course (PASSRC) and only the paver, roller, and bituminous haul are permitted on the Permeable Asphalt Stabilized Base (PASB). Only Minnesota permitted trucks are permitted to drive up to the PASB, drop off material, and must immediately move after dumping per Standard Section 2363.3. Prior to opening a pavement slab to regular traffic, the concrete must cure for 7 days, or reach a minimum compressive strength of 3,000 psi, or flexural strength based on thickness as shown in Table A.11, whichever happens first as stated in Standard Section 2301.3 (MnDOT 2018).

Table A.10: MnDOT Curing Requirements for Concrete Bridge Elements (MnDOT 2018)

Bridge Element	Minimum Curing Period	Minimum Period For Form Cure	Minimum Strength Required to Pull Forms, psi	Minimum Strength to Apply Loads, % of Required †	Method Allowed to determine in-place concrete strength
Bridge superstructures, unless otherwise specified	96 hrs	24 hrs	2000 ǂ	65	Maturity or Control Cylinders
Slab Span Superstructure	7 days	8 days	See special provisions	See special provisions	Maturity or Control Cylinders
Diaphragms and end webs not a part of box girders and cast before the bridge slab	72 hrs	24 hrs	2000 ǂ	45	Maturity or Control Cylinders
Pier Caps	72 hrs	72 hrs	2000 ǂ	65	Maturity or Control Cylinders
Retaining Walls	72 hrs	12 hrs *	Self-supporting	100	Maturity or Control Cylinders
Barriers and Parapets	72 hrs	-	Self-supporting	45 ‖	Maturity or Control Cylinders
Sections not included in superstructures, unless otherwise specified	72 hrs	24 hrs	2000 ǂ	45	Maturity or Control Cylinders
Bridge Decks	7 days	-	-	100	-
Bridge Deck Underside	7 days	8 days	2000	100	-

* When weather conditions require cold weather protection in accordance with 2401.3.G.5, "Protection Against Cold Weather," increase form curing to a minimum of 24 hours.
‖ Achieve 4000 psi prior to use as a traffic barrier.
† Applied loads include but are not limited to equipment, beams, backfilling, or successive concrete placements.
ǂThe Engineer will require verification of the minimum strength when air temperatures drop below 40° F during the curing period or when the mix design includes greater than 15% cement substitution. The minimum strength requirement does not apply to bulkheads and edge of deck forms.

Table A.11: MnDOT Minimum Requirements for Opening Pavements to General Traffic (MnDOT 2018)

Slab Thickness, in	Flexural Strength, psi
≤7.0	500
7.5	480
8.0	460
8.5	440
9.0	390
≥ 9.5	350

Additionally, MnDOT specifications contain mixture design requirements for concrete that include maximum allowable SCMs percentages based on the use of the concrete, maximum w/c ratios, maximum cementitious material content, along with other requirements as shown in Table A.12. Table A.13 provides MnDOT's design requirements specifically for high early strength concrete, with strength requirements for opening roads to traffic.

Table A.12: Minnesota Concrete Mix Design Requirements (MnDOT 2018)

Concrete Grade	Mix Number	Intended Use *	Maximum w/c ratio †	Maximum Cementitious Content (lbs/yd^3)	Maximum %SCM (Fly Ash/ Slag/Ternary)	Slump Range	Minimum 28-day Compressive Strength, f'c	3137 Spec.
B Bridge Substructure	3B52 ‡	Abutment, stems, wingwalls, paving brackets, pier columns and caps, pier struts	0.45	750	30/35/40	2 - 5"	4000 psi	2.D.1
F Flatwork	3F32 ‡	Slipform curb and gutter	0.42	750	30/35/0	½ - 3" #	4500 psi	2.D.1
	3F52 ‡ 3F52EX 3F53EX 3F52CO ‖	Walks, curb and gutter, slope paving, median walks, driveway entrances, ADA pedestrian walks	0.45	750	25/30/0	2 - 5"	4500 psi	2.D.1
G General Concrete	1G52 ‡	Footings and pilecap	0.55	750	30/35/40	2 - 5"	4500 psi	2.D.1
	3G52 ‡	Footings, pilecap, walls, cast-in-place manholes and catch basins, fence posts, signal bases, light pole foundations, erosion control structures, cast-in-place box culverts, culvert headwalls, open flumes, cast-in-place wall stems	0.45	750	30/35/40	2 - 5"	4500 psi	2.D.1
M Median Barrier	3M12	Slipform barrier, median barrier, non-bridge	0.42	750	30/35/40	½ - 1" #	4500 psi	2.D.1
	3M52	Barrier, median barrier, non-bridge	0.45	750	30/35/40	2 - 5"	4500 psi	2.D.1
P Piling	1P62 ‡	Piling, spread footing leveling pad	0.68	750	30/35/40	3 - 6"	3000 psi	2.D.1
R Pavement Rehabilitation	3R52 ‡	CPR - Full depth concrete repairs, concrete base	0.45	750	30/35/40	2 - 5"	4000 psi	2.D.3
S Bridge Superstructure	3S12	Slipform bridge barrier, parapets, end post	0.42	750	30/35/40	½ - 1" #	4000 psi	2.D.2
	3S52	Bridge median barrier, raised median, pilaster, curb, sidewalk, approach panel, formed bridge barrier, parapet, end post, collar	0.45	750	30/35/40	2 - 5"	4000 psi	2.D.2
X Miscellaneous Bridge	1X62	Cofferdam seals, rock sockets, drilled shafts	0.45	750	30/35/40	3 - 6"	5000 psi	2.D.1
	3X62	Drilled shafts above frost line	0.45	750	30/35/40	3 - 6"	5000 psi	2.D.1
Y Bridge Deck §	3Y42-M 3Y42-S	Bridge decks, integral abutment diaphragms, pier continuity diaphragms, expansion joint replacement mix	0.45	750	30/35/40	2 - 4"	4000 psi	2.D.2
	3Y47 **	Deck patching mix	0.45	750	30/35/40	2 - 4"	4000 psi	2.D.2

* If the intended use is not included elsewhere in the Specification or Special Provisions, use mix 3G52, unless otherwise directed by the Engineer.
‖ Identify specific color used on the certificate of compliance. Colored concrete is only allowed when specified in the plans or the Contract.
† The minimum water/cement (w/c) ratio is 0.30.
‡ The Contractor may choose to use the Coarse Aggregate Designation "1" for the 4th digit in accordance with Table 2461-3.
# Adjust slump in accordance with 2461.3.G.7.a for slipform concrete placement.
§ The "-S" indicates a bridge deck with a structural slab and "-M" indicates a monolithic bridge deck.
** Mix 3Y47 requires the use of Coarse Aggregate Designation "7" or "3" for the 4th digit in accordance with Table 2461-3.

Table A.13: Minnesota High Early Strength Design Requirements (MnDOT 2018)

Mix Number	Concrete Grades Allowed	Minimum Time to Opening	Maximum w/c ratio	Maximum Cementitious Content (lbs/ yd³) *	Slump Range	Minimum Strength to Opening	Minimum 28-day Compressive Strength, f'c	3137 Spec.
3HE32	F	48 hrs	0.42	750	1 – 3" ‖	3000 psi	4500 psi	2.D.1
3HE52	B, F, G	48 hrs	0.42	750	2 – 5"	3000 psi	4500 psi	2.D.1
3YHE52	Y (Repairs Only)	48 hrs	0.42	750	2 – 5"	3000 psi	4000 psi	2.D.2
3RHE52	R (Repairs Only)	48 hrs	0.42	750	2 – 5"	3000 psi	4000 psi	2.D.3

* Supplementary Cementitious Materials allowed.
‖ Adjust slump in accordance with 2461.3.G.7.a,"Concrete Placed by the Slip-form Method."

State of New York Department of Transportation (NYSDOT)

Similarly, NYSDOT has developed more advanced standards inclusive to SCMs. Waste materials are encouraged and at times required for concrete mixtures in this state, as long as the waste material is performance verified, readily available, and does not harm the environment as stated in Section 106-05 entitled "Recycled Materials" (NYDOT 2019). Pozzolanic material is required as a partial replacement for portland cement in Class DP, G, and HP concrete in New York, and is allowed as partial replacement for all concrete classes except Class F as stated in Section 501-2.02 "Material Requirements". Class DP concrete is a mixture of cement, fly ash micro-silica, fine and coarse aggregate, air entraining agent admixture and is used for concrete structures. Class G concrete is a low shrinkage fiber-reinforced structural concrete. Class HP is High Performance concrete utilized for concrete structures. Table A.14 shows concrete classes and allowable amounts of cement substitution with fly ash for each class (NYSDOT 2019).

Table A.14: New York Allowable Pozzolan Substitutions (NYSDOT 2019)

Concrete Class Specified	Substitute Cement by Mass With	Class Substitution Allowed
A, C, E, H	15-20% Class F Fly Ash (711-10)	HP[1]
I, J	15-20% Class F Fly Ash (711-10)	-
D	15-20% Class F Fly Ash (711-10)	DP[1]
G[2] and GG[2]	20% Class F Fly Ash (711-10)	-
F	No Substitution Allowed	-

NOTES:
1. Class HP and DP concrete may be substituted to mitigate ASR as listed above. Classes HP and DP require the replacement of portland cement with 20% pozzolan and 6% microsilica. The pozzolan may be either Class C or F Fly Ash (§711-10) or Ground Granulated Blast Furnace Slag (§711-12).
2. Classes G and GG require the replacement of portland cement with 20% pozzolan. The mitigation of ASR in Classes G and GG must be accomplished using Class F Fly Ash (§711-10).

In regard to allowing loads on newly constructed bridges and roads, compressive strength results are used to determine when loading can begin unless otherwise stated by the regional engineer. Even if early loading is requested, the regional engineer will base decision for loading on compressive strength results. Table A.15 shows minimum wait times for loading based on the structure type, but are not applicable for concrete with fly ash, GGBFS, or concrete placed in ambient temperatures less than 60 degrees Fahrenheit as stated in Section 555-03.08 (NYDOT 2019).

Table A.15: Minimum Time for Form Removal and Loading Limitations for Substructures in New York (NYSDOT 2019)

SUBSTRUCTURE ELEMENT	STRIPPING (2)	FORMING NEXT PLACEMENT	LOADING
All Footings	2 days	2 days	4 days before next placement
Abutment stems, backwalls	2 days if less than 10 feet (avg.). Add 1 day for each additional 5 feet to 5 days, maximum.	2 days	5 days before placing backwall on stem. 7 days before backfilling, 14 days before placing superstructure loads. (3)
Pier Columns, Pier Plinths	2 days if less than 10 feet high (avg.). Add 1 day for each additional 5 feet.	4 days – columns 2 days if forming pedestal	Columns – 7 days before placing cap beam. Plinth- 2 days before pedestal placement. 21 days before placing superstructure loads. (3)
Pier cap beams	8 days (bottom) 3 days (sides)	2 days	5 days before pedestal placement. 21 days before placing superstructure loads. (3)
All pedestals	2 days	—	7 days (class A) 3 days (class F) (4)
Wingwalls or Retaining walls	Same as abutment stems.	—	14 days before backfilling (3)
Arch centers Centering under beams	8 days	—	14 day (3)

All construction vehicles must be in accordance with the Vehicle and Traffic Law Section 385, along with complying with the limits provided by the contract. Any vehicles or equipment over the legal gross weight limits, must be approved and operate under Section 385 as well. In addition, any over-weight equipment must be approved by the contract Engineer prior to loading structures (NYDOT 2019).

When class C concrete is specified for pavements, Section 502-3.18 states roads can be opened to construction traffic and equipment 7 days after placement, or 3 days if contract Engineer approves and test cylinders prove to have a minimum compressive strength of 2500 psi in accordance with Section 502-3.18C. As far as general traffic, if placed between June 1 and September 15, roads can be opened after 10 days, and if placed outside this window general traffic is allowed after 15 days according to the same section. If the contract Engineer approves, the roads can be opened within 4 days if cylinders tested in accordance with Section 502-3.18C reach a minimum compressive strength of 3,000 psi. Also in section 502-2.02 of the standard specifications, High Early Strength (HES) Concrete can be used when early age opening is required or requested. Table A.16 provides the HES concrete mix requirements, which includes opening roads to traffic (NYDOT 2019).

Table A.16: High Early Strength Concrete Requirements for New York (NYSDOT 2019)

Property	Minimum	Desired	Maximum
28 Day Compressive Strength	4000 psi	-	-
Opening Compressive Strength	2500 psi	-	-
Freeze-Thaw Loss (Test 502-3P, 3% NaCl)	-	0.0 %	3.0 %
Plastic Air Content	5.0 %	6.5 %	8.0 %
Hardened Air Content	5.0%	6.5 %	8.0 %
Water – Cement Ratio (w/c)	-	-	0.44
Slump[2]	1 in	-	6 in

Florida Department of Transportation (FDOT)

FDOT requires fly ash in all classes of concrete except for use of the following in an "aggressive" environment: Class I (3,000psi), Class I (3,000psi pavement), and Class II (3,400psi) as stated in Section 346-2.3 (FDOT 2019). In the same section, it states that SCMs may be used as an equal weight replacement for portland cement within total cementitious materials limitations, meaning the total of SCM and portland cement must stay within limits. Table A.17 describes the concrete mixture proportions for cementitious materials based on application, the environment conditions are considered aggressive unless otherwise noted. Section 346-4 includes a master proportion table shown as Table A.18, limiting the amount of total cementitious material and w/c ratio sorted by class of concrete. In Section 346-2.2, FDOT specifies cement types for structures based on environmental use as shown in Table A.19. Also, FDOT specifies minimum 28-day strength and slump target values for each class of concrete, as shown in Table A.20 with emphasis on Class I (pavement) and Class II (bridge deck) as those at pertinent to this research.

Table A.17: Maximum Permissible Florida DOT Cementitious Materials and Mixture Proportions (%) (FDOT 2019)

Application	Portland Cement	Fly Ash Type F	Slag	Highly Reactive Pozzolans		
				Silica Fume	Metakaolin	Ultra-Fine Fly Ash
General Use	70-82	18-30				
	66-78	15-25		7-9		
	66-78	15-25			8-12	
	66-78	15-25				8-12
	30-40	10-20	50-60			
	30-75(1)		25-70(1)			
	30-50		50-70			
	36-43		50-55	7-9		
	33-42		50-55		8-12	
	33-42		50-55			8-12
Precast Prestressed	70-85(1)	15-30(1)				
	70-82	18-30				
	66-78	15-25		7-9		
	66-78	15-25			8-12	
	66-78	15-25				8-12
	30-40	10-20	50-60			
	30-50		50-70			
	36-43		50-55	7-9		
	33-42		50-55		8-12	
	33-42		50-55			8-12
Drilled Shaft	63-67	33-37				
	38-42		58-62			
	30-40	10-20	50-60			
Mass Concrete	50-82(2)	18-50(2)				
	50-65(3)	35-50(3)				
	66-78	15-25		7-9		
	66-78	15-25			8-12	
	66-78	15-25				8-12
	30-40	10-20	50-60			
	30-50		50-70			
	36-43		50-55	7-9		
	33-42		50-55		8-12	
	33-42		50-55			8-12

(1) Slightly Aggressive and Moderately Aggressive environments.
(2) Concrete Core Temperature T≤165°F.
(3) Concrete Core Temperature T≥165°F.

Table A.18: Florida DOT Concrete Master Proportions (FDOT 2019)

Class of Concrete	Minimum Total Cementitious Materials Content pounds per cubic yard	Maximum Water to Cementitious Materials Ratio pounds per pounds*
I	470	0.53
I (Pavement)	470	0.50
II	470	0.53
II (Bridge Deck)	611	0.44
III	611	0.44
III (Seal)	611	0.53
IV	658	0.41**
IV (Drilled Shaft)	658	0.41
V (Special)	752	0.37**
V	752	0.37**
VI	752	0.37**
VII	752	0.37**

*The calculation of the water to cementitious materials ratio (w/cm) is based on the total cementitious material including cement and any supplemental cementitious materials that are used in the mix.
** When silica fume or metakaolin is used, the maximum water to cementitious material ratio will be 0.35. When the use of ultrafine fly ash is required, the maximum water to cementitious material ratio will be 0.30.

Table A.19: Florida DOT Cement Use by Environmental Classification (FDOT 2019)

Component	Slightly Aggressive Environment	Moderately Aggressive Environment	Extremely Aggressive Environment
Bridge Superstructures			
Precast Superstructure and Prestressed Elements	Type I or Type III	Type I, Type IL, Type II, Type III, Type IP, or Type IS	Type II (MH), Type IL, or Ternary Blend

Component	Slightly Aggressive Environment	Moderately Aggressive Environment	Extremely Aggressive Environment
Bridge Superstructures			
Cast In Place	Type I	Type I, Type IL, Type II, Type IP, or Type IS	Type II (MH), Type IL, or Ternary Blend
Bridge Substructures, Drainage Structures and other Structures			
All Elements	Type I or Type III	Type I, Type IL, Type II, Type IP, or Type IS	Type II (MH), Type IL, or Ternary Blend

Notes:
1. Cements used in a more aggressive environment may also be used in a less aggressive environment.
2. Type III cement may be used in an Extremely Aggressive Environment for precast superstructure and prestressed elements when the ambient temperature at the time of concrete placement is 60°F and below.

Table A.20: FDOT Concrete class, Compressive Strength, and Slump Requirements (FDOT 2019)

TABLE 3
Concrete Class, Compressive Strength, and Slump

Class of Concrete	Specified Minimum Strength (28-day) (psi)	Target Slump Value (inches) (c)
Structural Concrete		
I [(a)]	3,000	3 [(b)]
I (Pavement)	3,000	2
II [(a)]	3,400	3 [(b)]
II (Bridge Deck)	4,500	3 [(b)]
III [(e)]	5,000	3 [(b)]
III (Seal)	3,000	8
IV [(d)(f)]	5,500	3 [(b)]
IV (Drilled Shaft)	4,000	8.5
V (Special) [(d)(f)]	6,000	3 [(b)]
V [(d)(f)]	6,500	3 [(b)]
VI [(d)(f)]	8,500	3 [(b)]
VII [(d)(f)]	10,000	3 [(b)]

(a) For precast three-sided culverts, box culverts, endwalls, inlets, manholes and junction boxes, the target slump value and air content will not apply. The maximum allowable slump is 6 inches, except as noted in (b). The Contractor is permitted to use concrete meeting the requirements of ASTM C478 4,000 psi in lieu of Class I or Class II concrete for precast endwalls, inlets, manholes and junction boxes.
(b) The Engineer may allow a maximum target slump of 7 inches when a Type F, G, I or II admixture is used. When flowing concrete is used, the target slump is 9 inches.
(c) For a reduction in the target slump for slip-form operations, submit a revision to the mix design to the Engineer. The target slump for slip-form mix is 1.50 inches.
(d) When silica fume, ultrafine fly ash, metakaolin, or a ternary blend cement is used in Class IV, Class V, Class V (Special), Class VI, or Class VII concrete, ensure that the concrete meets or exceeds a resistivity of 29 KOhm-cm at 28 days, when tested in accordance with AASHTO T358. Submit three 4 x 8 inch cylindrical test specimens to the Engineer for resistivity testing before mix design approval. Take the resistivity test specimens from the concrete of the laboratory trial batch or from the field trial batch of at least 3 cubic yards. Verify the mix proportioning of the design mix and take representative samples of trial batch concrete for the required plastic and hardened property tests. Cure the field trial batch specimens similar to the standard laboratory curing methods. Submit the resistivity test specimens at least 7 calendar days prior to the scheduled 28 day test. The average resistivity of the three cylinders, eight readings per cylinder, is an indicator of the permeability of the concrete mix.
(e) When precast three-sided culverts, box culverts, endwalls, inlets, manholes or junction boxes require a Class III concrete, the minimum cementitious materials is 470 pounds per cubic yard. Do not apply the air content range and the maximum target slump shall be 6 inches, except as allowed in (b).
(f) Highly reactive pozzolans may be used outside the lower specified ranges to enhance strength and workability. Testing in accordance with AASHTO T358 is not required.

For any road, street, or bridge (including temporary bridges owned by FDOT), equipment cannot be operated in excess of maximum weights specified in Florida Highway Control, Commercial Motor Vehicle Manual, or in excess of posted lower weight limits established legally as per Section 7-7.2 of the FDOT specification manual (FDOT 2019). Fresh concrete must be cured continuously for 72 hours. Unless the project engineer approves earlier opening, fresh concrete must be cured at least 14 days prior to opening structures to traffic. The project contractor can open any portion of a structure to vehicular or pedestrian traffic as long as the project engineer approves as per Section 7-15. Generally, the engineer will approve early opening to traffic only if concrete samples made in accordance with ASTM C31 and tested in accordance with ASTM C39 prove to be at least 2,200 psi as stated in Section 350-16. The pavement must be protected from all operations (including construction equipment loading) until specified time has elapsed. For bridge decks and slabs, concrete must be wheeled in order to avoid construction loading, and concrete has to cure for at least 14 days prior to opening road to traffic or approved by project engineer with a verified minimum compressive strength of 1,600 psi as per Section 400-17.1.

Iowa Department of Transportation (IowaDOT)

In the Standard Specifications, IowaDOT states in Materials I.M. Section 491.17 that all fly ash and GGFBS must be selected from an approved source and must be in accordance with AASHTO M 295 (IowaDOT 2015). As per standard section 4108.01 fly ash must be either Class F or Class C, and Class F must be tested for pozzolanic activity with lime. The allowable fly ash and slag substitution is dependent on the type of mixture and purpose of mixture. For low traffic pavements class A-mixtures are used, while for most pavement and bridge decks class-C mixtures are used. For bridge deck overlays, blended cements, slag, and fly ash is required in the mixtures as per standard IM-529, "Portland Cement Concrete Proportions" and the maximum *w/cm* ratio is 0.42. Any concrete made using class V aggregates, which are fine and coarse feldspathic rocks, must follow Section 4117, "Class V Aggregates for Portland Cement Concrete" shown below in Table A.21. Fly ash is limited to a substitution rate of 20% and slag is limited to a rate of 20%, with up to 50% total mineral admixture substitution for concrete structures as per section 2403, "Structural Concrete." For concrete bridge decks, as stated in section 2412 of the standard specifications, the maximum allowable substitution rates shown in Table A.22 are adhered. For concrete pavements mixtures, fly ash is limited to a substitution rate of 20% and GGFBS is limited to 35% with a maximum of 40% total mineral admixture as per standard section 2301, "Portland Cement Concrete Pavement." For blended cements such as Type IP or IS, only fly ash is permitted as a substitution (IowaDOT 2015).

Table A.21: IowaDOT Cement Types and Substitution for Portland Cement Concrete with Class V Aggregates (IowaDOT 2015)

Cement Type	Min. Required Substitution	Max. Allowable Substitution
Type I, Type II	20% Class F Fly Ash	25% Class F Fly Ash
Type I, Type II	25% GGBFS	35% GGBFS
Type IS, IP	---	20% Class C Fly Ash

Table A.22: IowaDOT Maximum Allowable Substitution Rates for Concrete Bridge Decks (IowaDOT 2015)

Cement Type	Maximum Allowable Substitution[a]	Time Period
Type I, Type II	35% GGBFS 20% Fly Ash	March 16 through October 15
Type IS, IP	0% GGBFS 20% Fly Ash	March 16 through October 15
Type I, II, IS, IP	0% GGBFS 0% Fly Ash	October 16 through March 15

Construction equipment and other external loads must be simple compressive loads only for concrete structures, and must not exceed allowable loads designated by the designer. Prior to loading concrete structures unless otherwise noted, the concrete must reach the ages shown in Table A.23, and reach a minimum of 575 psi flexural strength as per section 2403. For concrete pavements, the maturity method can be used to expedite and determine when loads can be applied. The maturity method was discussed in Chapter 2, Section 2.1.1.3. Otherwise pavements must be in accordance with the age and strengths shown in Table A.24 (IowaDOT 2015).

Table A.23: IowaDOT Minimum Age Requirements for Loading Concrete Structures (IowaDOT 2015)

Portland cement (Type I and Type II with or without Class C fly ash)	7 calendar days
With Class F fly ash substitution	8 calendar days
Class M mix (with or without Class C or Class F fly ash)	3 calendar days
If strength is not determined (regardless of type of cement or class of fly ash)	14 calendar days

Table A.24: Minimum Flexural Strength for Opening Concrete Pavements (IowaDOT 2015)

Strength Class of Concrete	Minimum Age	psi (MPa)
A	14 calendar days[(a)]	500 (3.45)
B	14 calendar days	400 (2.80)
C	7 calendar days[(b)]	500 (3.45)
M	48 hours[(c)]	500 (3.45)

(a) 10 calendar days for concrete 8 inches (200 mm) thick or more.
(b) 5 calendar days for concrete 9 inches (230 mm) thick or more.
(c) Pavement may be opened for use prior to 48 hours when minimum flexural strength requirements are met.

Illinois Department of Transportation (IDOT) and Illinois Tollway Authority

The Illinois Department of Transportation (IDOT) and Illinois Tollway standard specifications are summarized together since the Illinois Tollway follows IDOT with the exception of the supplemental specifications provided by the Tollway for special provisions. There are no supplemental provisions for portland cement concrete, thus the following specifications apply to both IDOT and the Illinois Tollway Authority. Section 1020.04 states that portland-pozzolan cement, portland limestone cement or any other combination of finely divided minerals and cement, must contain at least 400 pcy of OPC (IDOT 2016). Class PV is designated for paving mixtures and BS is designated for bridge structure mixtures as shown in Table A.25. Table A.25 also presents the mix design criteria for bridge and pavement mixtures in Illinois. For PV and BS class mixtures Class F fly replacement rates are not to exceed 25%, and limited to 30% for Class C fly ashes as per section 1020.05 (c)(1) in the IDOT standard specifications.

Table A.25: Mix Design Criteria for IDOT (IDOT 2016)

TABLE 1. CLASSES OF CONCRETE AND MIX DESIGN CRITERIA

Class of Conc.	Use	Specification Section Reference	Cement Factor cwt/cu yd (3) Min.	Cement Factor cwt/cu yd (3) Max	Water / Cement Ratio lb/lb	Slump in. (4)	Mix Design Compressive Strength (Flexural Strength) psi, minimum Days 3	14	28	Air Content %	Coarse Aggregate Gradations (14)
PV	Pavement Base Course Base Course Widening Driveway Pavement Shoulders Shoulder Curb	420 or 421 353 354 423 483 662	5.65 (1) 6.05 (2)	7.05	0.32 - 0.42	2 - 4 (5)	Ty III 3500 (650)	3500 (650)		5.0 - 8.0 (5)	CA 5 & CA 7, CA 5 & CA 11, CA 7, CA 11, or CA 14
PP	Pavement Patching Bridge Deck Patching (10)	442					3200 (600) Article 701.17(e)(3)b.				CA 7, CA 11, CA 13, CA 14, or CA 16
	PP-1		6.50 6.20 (Ty III)	7.50 7.20 (Ty III)	0.32 - 0.44	2 - 4	at 48 hours			4.0 - 7.0	
	PP-2		7.35	8.20	0.32 - 0.38	2 - 6	at 24 hours			4.0 - 6.0	
	PP-3		7.35 (Ty III) (8)	7.35 (Ty III) (8)	0.32 - 0.35	2 - 4	at 16 hours			4.0 - 6.0	
	PP-4		6.00 (9)	6.25 (9)	0.32 - 0.50	2 - 6	at 8 hours			4.0 - 6.0	
	PP-5		6.75 (9)	6.75 (9)	0.32 - 0.40	2 - 8	at 4 hours			4.0 - 6.0	
RR	Railroad Crossing	422	6.50 6.20 (Ty III)	7.50 7.20 (Ty III)	0.32 - 0.44	2 - 4	3500 (650) at 48 hours			4.0 - 7.0	CA 7, CA 11, or CA 14
BS	Bridge Superstructure Bridge Approach Slab	503	6.05	7.05	0.32 - 0.44	2 - 4 (5)		4000 (675)		5.0 - 8.0 (5)	CA 7, CA 11, or CA 14 (7)
PC	Various Precast Concrete Items Wet Cast Dry Cast	1042	5.65 5.65 (TY III)	7.05 7.05 (TY III)	0.32 - 0.44 0.25 - 0.40	1 - 4 0 - 1	See Section 1042			5.0 - 8.0 N/A	CA7, CA11,CA 13, CA 14, CA 16, or CA 7 & CA 16
PS	Precast Prestressed Members Precast Prestressed Piles and Extensions	504 512	5.65 5.65 (TY III)	7.05 7.05 (TY III)	0.32 - 0.44	1 - 4			Plans 5000	5.0 - 8.0	CA 11 (11), CA 13, CA 14 (11), or CA 16
	Precast Prestressed Sight Screen	639							3500		

IDOT specifies in Section 107.29 that the project engineer will determine when/if a concrete pavement or structure is to be opened to regular traffic (IDOT 2016). Also, Section 707.17 (c)(5) of IDOT's standard specifications states pavements will not be opened to regular traffic until 650 psi flexural strength is met or 3,500 psi compressive strength is met. If these tests are not conducted, concrete pavements cannot be opened until 14 days after placement for OPC, and 28 days for concrete mixtures with fly ash or GGFBS. This section mentions all traffic (including construction traffic) should be limited to legal axle loads. For structural concrete (i.e. Class BS concrete in this case) a minimum of 4,000 psi compressive strength or required flexural strength as determined by the project engineer must be met prior to loading concrete structures. As shown in Table A.25, this is to be tested at 14 days. As per the minimum curing schedule shown in section 1020.13, pavements must cure at least 3 days, and bridge decks must cure at least 7 prior to opening to traffic (IDOT 2016).

It should be noted that although concrete strength is a traditional method to ensure a pavement or bridge component can be subjected to traffic or other loads, the potential for a component to be distressed is also affected by other factors such as base thickness/strength, subgrade strength and reinforcement. Similarly, although concrete strength has been somewhat linked to durability at times, other performance variables are just as essential in determining durability, such as shrinkage as discussed in this report.

A.6 Specification Provisions for Volumetric Shrinkage

North Carolina Department of Transportation

NCDOT standard specifications state in section 420-15 to properly cure concrete structures for a minimum of seven days and take all necessary precautions to avoid shrinkage cracking including wind screens, temporary liquid moisture barriers, or early application of wet coverings (NCDOT 2018). In hot weather, concrete temperatures must be controlled to prevent plastic cracking and as stated in section 1078-9 of the standard specifications, if shrinkage cracks occur during or after placement, the project engineer determines if removal or remediation is required. Otherwise, there is no specific target or testing required for unrestrained shrinkage.

LaDOTD

The LaDOTD standard specifications state in section 901.11.2 that concrete placed in high temperatures (hot weather) must be designed, placed, and cured properly to avoid plastic shrinking (LaDOTD 2016). The only target specification for shrinkage is for undersealing or slab-jacking pavements and for structural concrete patching, where the shrinkage after four days must not change more than 0.13 percent in length and no more than 0.07 percent in length as per ASTM C157 testing procedure (LaDOTD 2017).

NYDOT

NYDOT standard specifications state in section 718-06, for High Performing (HP) concrete length change due to shrinkage must be less than 600 microstrain tested in accordance with AASTHO T160-97, “Standard Method of Test for Length Change of Hardened Hydraulic Cement Mortar and Concrete” (ASTM C157) at 56 days (NYDOT 2018). For other concrete classes, there is no specified target maximum for shrinkage.

IowaDOT

The IowaDOT standard specifications do not specify target or standards for shrinkage resistance for concrete pavements and structures except for ultra-high performing concrete. For this type of concrete the initial shrinkage (tested after initial set) should be less than 766 micro-strain tested in accordance with ASTM C150, as stated in special provisions section 150289 (IowaDOT 2015).

IDOT and Illinois Tollway Authority

In the Standard Specifications for Road and Bridge Construction, IDOT specifies shrinkage targets for the following concrete applications. For rapid hardening cement, shrinkage is limited to 0.050 percent in accordance to ASTM C 596, “Standard Test Method for Drying Shrinkage of Mortar Containing Hydraulic Cement” (IDOT 2016). Other than concrete mixtures using rapid hardening cement, targets for shrinkage specification were not found in the IDOT specifications or the supplementary specifications for the Illinois Tollway.

An ACI webinar about the Illinois Tollway Authority, discusses implementation of performance specifications in current and future projects and shrinkage is discussed (Gancarz 2018). For HPC (structural concrete use) contractors have two options for shrinkage mitigation. The first option is to use shrinkage reducing admixtures at a rate of 1.5 gallons/cy and limit the cementitious materials to less than 605 lb/cy total cementitious material content or the other option is to provide test results of the ring test (ASTM C 596) proving drying shrinkage has been mitigated. For pavements and structures, the Illinois Tollway has identified reduced cementitious material contents, and increased use of SCMs as essential to producing durable concrete mixtures (Gancarz 2018).

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APPENDIX B – SUPPORTING MATERIAL FOR LABORATORY TESTING PROGRAM AND RESULTS (Chapter 4)

Holcim

NSF Certified to NSF/ANSI 61 & 372

Material Certification Report

Material: Portland Cement
Type: I-II (MH)

Test Period: 17-Jun-2018 to 17-Jun-2018
Date Issued: 13-Jul-2018

Certification

This cement meets the specifications of ASTM C150 and AASHTO M85 for Type I-II (MH) cement.

General Information

Supplier:	Holcim (US) Inc. d/b/a LafargeHolcim US	Source Location:	Holly Hill Plant Silo: 8
Address:	8700 West Bryn Mawr Ave Chicago, IL 60631		2173 Gardner Blvd Holly Hill, SC 29059
Contact:		Contact:	Scott Poaps / (803) 496-2995

The following is based on average test data during the test period. The data is typical of product shipped from this source; individual shipments may vary.

Test Data on ASTM Standard Requirements

Chemical			Physical		
Item	**Limit [1]**	**Result**	**Item**	**Limit [1]**	**Result**
SiO_2 (%)	-	19.2	Air Content (%)	12 max	6
Al_2O_3 (%)	6.0 max	4.5	Blaine Fineness (m^2/kg)	260-430	384
Fe_2O_3 (%)	6.0 max	3.4			
CaO (%)	-	63.8	Autoclave Expansion (%) (C151)	0.80 max	0.01
MgO (%)	6.0 max	1.3	Compressive Strength MPa (psi)		
SO_3 (%) [2]	3.0 max	2.9	3 day	10.0 (1450) min	31.2 (4530)
Loss on Ignition (%) [5]	3.5 max	2.6	7 day	17.0 (2470) min	38.3 (5550)
Insoluble Residue (%)	1.50 max	0.54			
CO_2 (%)	-	1.4	Initial Vicat (minutes)	45-375	120
$CaCO_3$ in Limestone (%)	70 min	87			
Potential Phase Compositions [3]:			Mortar Bar Expansion (%) (C1038)	0.020 max	0.005
C_3S (%)	-	63			
C_2S (%)	-	7			
C_3A (%)	8 max	6			
C_4AF (%)	-	10			
$C_3S + 4.75C_3A$ (%)	100 max	93			

Test Data on ASTM Optional Requirements

Chemical			Physical		
Item	**Limit [1]**	**Result**	**Item**	**Limit [1]**	**Result**
Equivalent Alkalies (%)	0.60 max	0.52	Heat of Hydration kJ/kg (cal/g) (ASTM C1702) 3 Days [4]	-	308 (74)

Notes (*1-9)

1 - Dashes in the Limit / Result columns mean Not Applicable.
2 - It is permissible to exceed the specification limit provided that ASTM C1038 Mortar Bar Expansion does not exceed 0.020% at 14 days.
3 - Adjusted per Annex A1.6 of ASTM C150 and AASHTO M85.
4 - Test results represent the most recent value and is provided for information only.
5 - Limit = 3.0 when limestone is not an ingredient in the final cement product
6/17/2018
Grind 168/169

Additional Data

Item	Limestone	Inorganic Processing Addition	Base Cement Phase Composition	Result
Amount (%)	3.7	-	C_3S (%)	66
SiO_2 (%)	3.0	-	C_2S (%)	7
Al_2O_3 (%)	0.8	-	C_3A (%)	7
Fe_2O_3 (%)	0.6	-	C_4AF (%)	11
CaO (%)	52.4	-		
SO_3 (%)	0.1	-		

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Version: 180412

Scott Poaps

Scott Poaps,
Quality Manager

Figure B.1: Mill report for OPC

Material Certification Report

Brand: Envirocore™ Family of Products
Material: Blended Cement
Type: IL (10)

Test Period: 07-Jun-2018 to 07-Jun-2018
Date Issued: 13-Jul-2018

Certification

This cement meets the specifications of ASTM C595 and AASHTO M240 for Type IL cement.

General Information

Supplier:	Holcim (US) Inc. d/b/a LafargeHolcim US	Source Location:	Holly Hill Plant Silo: 7
Address:	8700 West Bryn Mawr Ave Chicago, IL 60631		2173 Gardner Blvd Holly Hill, SC 29059
Contact:		Contact:	Scott Poaps / (803) 496-2995

The following is based on average test data during the test period. The data is typical of product shipped from this source; individual shipments may vary.

Test Data on ASTM Standard Requirements

Chemical Item	Limit [1]	Result	Physical Item	Limit [1]	Result
MgO (%)	-	1.3	+45 um (No. 325) Sieve (%)	-	1.3
SO3 (%) [2]	3.0 max	3.3	Blaine Fineness (m^2/kg)	-	518
Loss on Ignition (%)	10.0 max	5.8	Autoclave (%) (C151)	-0.20-0.80	-0.01
			Initial Vicat (minutes)	45-420	115
$CaCO_3$ in Limestone (%)	70.0 min	86.5	Air Content (%)	12 max	6
			Compressive Strength MPa (psi)		
			3 day	13.0 (1890) min	36.8 (5340)
			7 day	20.0 (2900) min	42.8 (6210)
			28 day (previous month's data)	25.0 (3630) min	51.9 (7530)
			Mortar Bar Expansion (%) (C1038)	0.020 max	0.004

Test Data on ASTM Optional Requirements

Chemical Item	Limit [1]	Result	Physical Item	Limit [1]	Result

Notes (*1-9)

[1] Dashes in the limits columns means Not Applicable
[2] It is permissible to exceed the specification limit provided that ASTM C1038 Mortar Bar Expansion does not exceed 0.020% at 14 days.
This data may have been reported on previous mill certificates. It is typical of the cement being currently shipped.
6/7/2018
Grind 158/159

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Version: 180412

Scott Poaps

Scott Poaps,
Quality Manager

Figure B.2: Mill report PLC

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

REPORT OF FLY ASH TESTS

Date Sampled: DS 11/23-12/11
Manufacturer: Roxboro

Start Date: November 23, 2015
End Date: December 11, 2015
Date Received: December 16, 2015

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 testing report

Table B.1: Sieve analysis for coarse aggregate

Sieve Size	% Passing	ASTM C33 Specification (% Passing)
1 in	100	100
¾ in	98	98
½ in	59	59
3/8 in	36	36
No. 4	5	5
No. 8	3	3
No. 200		

Table B.2: Sieve analysis for fine aggregate

Sieve Size	% Passing	ASTM C33 Specification (% passing)
3/8 in	100	100
No. 4	100	95-100
No. 8	98	80-100
No. 16	86	50-85
No. 30	41	25-60
No. 50	9	5-30
No. 100	2	0-10
No. 200	1.0	0-3

Table B.3: Fresh concrete test results (average of two batches comprising a single mixture)

Designation	Slump (in.)	Air content (%)	Unit weight (pcf)
H-700-0	8.0	5.2	137.1
H-560-140	8.0	5.2	136.4
H-650-0	6.5	6.0	141.4
H-520-130	7.0	5.5	138.0
H-600-0	2.5	5.8	138.7
H-480-120	3.0	6.0	139.4
H-420-180	3.8	6.0	136.1
M-700-0	5.0	5.5	141.6
M-560-140	4.25	6.0	136.6
M-650-0	2.5	5.7	142.4
M-520-130	3.0	5.5	139.7
M-600-0	1.0	6.0	140.5
M-480-120	1.5	5.0	139.6
M-420-180	2.0	6.0	138.1
M-600P-0	0.8	5.5	141.1
M-480P-120	1.0	5.1	140.5
M-420P-180	1.5	5.9	137.0
L-700-0	2.25	6.0	143.9
L-560-140	1.8	5.0	140.3
L-650-0	1.0	6.0	141.8
L-520-130	1.0	5.0	141.6
L-600-0	1.0	5.5	142.6
L-480-120	0.8	5.5	142.0
L-420-180	1.0	5.2	142.0

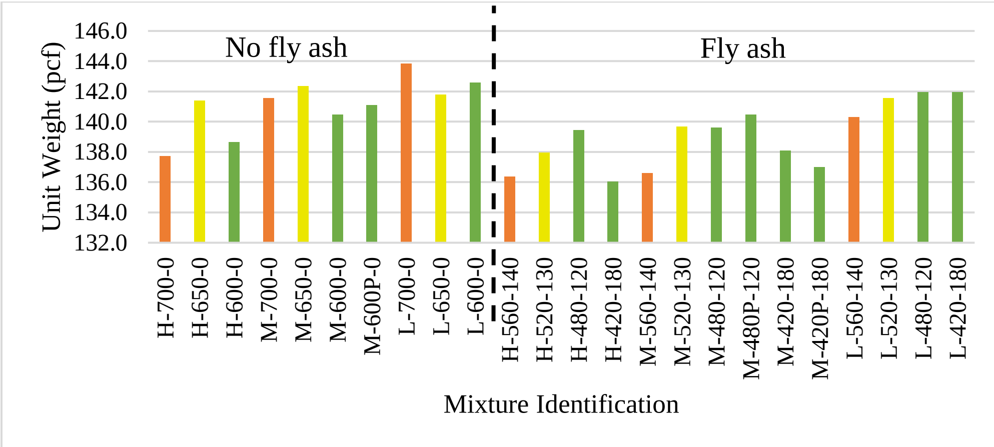

Figure B.4: Fresh unit weight test results

Table B.4: 28-day compressive strength results

Mixture identification	28 day compressive strength			Average compressive strength (psi)	Standard deviation
	1	2	3		
H-700-0	5,075	5,669	5,394	5,379	297.3
H-560-140	4,544	5,131	5,306	4,994	399.1
H-650-0	6,113	6,440	6,216	6,256	167.2
H-520-130	5,466	5,007	5,483	5,319	270.0
H-600-0	5,016	5,381	6,085	5,494	543.4
H-480-120	3,870	4,114	3,962	3,982	123.2
H-420-180	3,862	5,007	4,114	4,328	601.7
M-700-0	6,330	6,874	6,860	6,688	310.1
M-560-140	5,284	5,270	6,510	5,688	711.9
M-650-0	6,600	7,046	6,572	6,739	265.9
M-520-130	6,162	6,626	6,337	6,375	234.3
M-600-0	5,264	5,813	6,541	5,873	640.6
M-600P-0	6,531	6,388	5,933	6,284	312.3
M-480-120	4,567	5,290	6,313	5,390	877.3
M-480P-120	6,358	6,294	6,593	6,415	157.4
M-420-180	4,835	4,602	5,584	5,007	513.1
M-420P-180	5,226	4,719	5,328	5,091	326.2
L-700-0	8,348	7,303	7,916	7,856	525.1
L-560-140	6,528	6,261	7,398	6,729	594.6
L-650-0	7,810	7,690	8,473	7,991	421.7
L-520-130	7,694	7,056	6,859	7,203	436.5
L-600-0	6,989	6,742	7,299	7,010	279.1
L-480-120	7,318	7,136	5,988	6,814	721.1
L-420-180	5,980	6,054	6,650	6,228	367.3

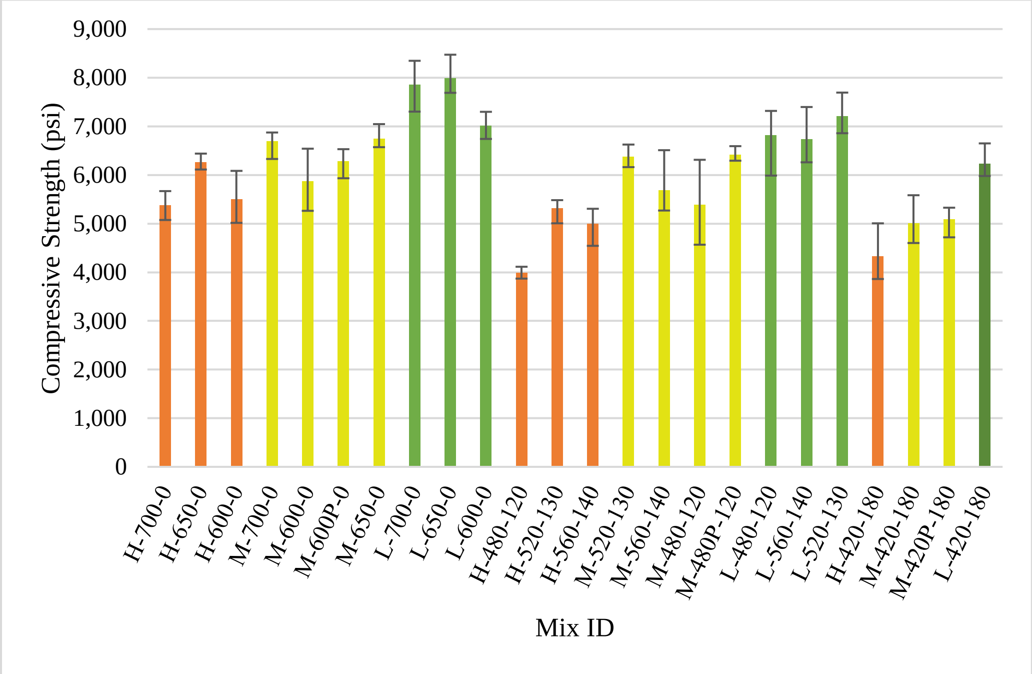

Figure B.5: 28-day compressive strength test results with variability

Table B.5: 28-day MOR results

Mixture identification	28 day MOR (psi)			Average MOR (psi)	Standard deviation
	1	2	3		
H-600-0	714.2	779.6	740.0	744.6	32.9
H-480-120	683.8	866.3	875.0	808.3	108.0
H-420-180	703.8	765.4	704.2	724.4	35.5
M-600-0	831.7	790.8	842.9	821.8	27.4
M-600P-0	859.2	820.4	747.5	809.0	56.7
M-480-120	780.4	692.9	705.4	726.3	47.3
M-480P-120	687.1	735.0	737.5	719.9	28.4
M-420-180	654.6	792.5	732.5	726.5	69.2
M-420P-180	604.6	669.6	767.5	680.6	82.0
L-600-0	703.3	868.8	878.8	816.9	98.5
L-480-120	654.2	759.6	740.4	718.1	56.2
L-420-180	898.8	749.6	797.9	815.4	76.1

Table B.6: 28-day MOE results

Mixture identification	28 day MOE (psi)		Average MOE (psi)	Standard deviation
	1	2		
H-700-0	3,389,412	2,700,545	3,044,979	487,102
H-560-140	2,464,897	2,884,458	2,674,677	296,674
H-650-0	3,601,875	3,698,410	3,650,142	68,261
H-520-130	3,132,694	2,979,134	3,055,914	108,584
H-600-0	2,951,483	3,008,643	2,980,063	40,418
H-480-120	2,703,661	2,349,676	2,526,668	250,305
H-420-180	2,518,430	2,403,926	2,461,178	80,967
M-700-0	3,459,243	3,678,499	3,568,871	155,037
M-560-140	3,451,607	3,274,101	3,362,854	125,516
M-650-0	3,604,745	3,806,583	3,705,664	142,721
M-520-130	3,816,814	3,423,214	3,620,014	278,317
M-600-0	3,254,569	3,541,713	3,398,141	203,041
M-600P-0	3,310,487	3,394,322	3,352,404	59,280
M-480-120	3,169,587	2,983,306	3,076,447	131,720
M-480P-120	3,390,621	3,513,363	3,451,992	86,792
M-420-180	3,098,216	3,162,973	3,130,595	45,790
M-420P-180	3,215,984	2,791,732	3,003,858	299,991
L-700-0	3,750,468	3,901,068	3,825,768	106,490
L-560-140	3,741,828	3,570,978	3,656,403	120,809
L-650-0	4,428,320	4,206,100	4,317,210	157,133
L-520-130	3,639,087	3,624,984	3,632,035	9,973
L-600-0	3,899,451	3,622,778	3,761,114	195,637
L-480-120	2,698,745	3,474,744	3,086,744	548,714
L-420-180	3,279,346	3,202,280	3,240,813	54,494

Table B.7: 28-day Poisson's ratio results

Mixture identification	28 day Poisson's ratio		Average Poisson's ratio	Standard deviation
	1	2		
H-700-0	0.21	0.21	0.21	0.00
H-560-140	0.20	0.19	0.20	0.01
H-650-0	0.20	0.22	0.21	0.01
H-520-130	0.23	0.22	0.23	0.01
H-600-0	0.18	0.20	0.19	0.01
H-480-120	0.20	0.20	0.20	0.00
H-420-180	0.19	0.24	0.22	0.04
M-700-0	0.23	0.24	0.24	0.01
M-560-140	0.18	0.18	0.18	0.00
M-650-0	0.19	0.20	0.20	0.01
M-520-130	0.20	0.19	0.20	0.01
M-600-0	0.19	0.22	0.21	0.02
M-600P-0	0.19	0.19	0.19	0.00
M-480-120	0.20	0.19	0.20	0.01
M-480P-120	0.22	0.23	0.23	0.01
M-420-180	0.19	0.19	0.19	0.00
M-420P-180	0.19	0.20	0.20	0.01
L-700-0	0.15	0.19	0.17	0.03
L-560-140	0.20	0.19	0.20	0.01
L-650-0	0.18	0.19	0.19	0.01
L-520-130	0.22	0.20	0.21	0.01
L-600-0	0.19	0.18	0.19	0.01
L-480-120	0.20	0.23	0.22	0.02
L-420-180	0.20	0.20	0.20	0.00

Table B.8: 28-day surface resistivity results

Mixture identification	28 day surface resistivity (kΩ-cm)			Average surface resistivity (kΩ-cm)	Standard deviation
	1	2	3		
H-700-0	6.8	7.5	7.6	7.3	0.44
H-560-140	6.7	6.5	6.6	6.6	0.10
H-650-0	8.9	8.6	8.4	8.7	0.24
H-520-130	10.3	10.8	10.8	10.6	0.28
H-600-0	8.8	8.4	7.1	8.1	0.87
H-480-120	9.3	9.0	10.1	9.5	0.57
H-420-180	9.7	11.6	12.2	11.2	1.30
M-700-0	10.8	10.8	11.2	10.9	0.22
M-560-140	6.3	7.2	5.7	6.4	0.75
M-650-0	10.5	10.9	10.7	10.7	0.21
M-520-130	12.0	12.3	12.1	12.1	0.14
M-600-0	10.2	9.7	10.0	10.0	0.25
M-600P-0	10.4	9.7	11.7	10.6	1.01
M-480-120	9.6	9.0	9.7	9.4	0.39
M-480P-120	7.0	6.7	6.1	6.6	0.46
M-420-180	6.4	6.0	5.9	6.1	0.26
M-420P-180	6.1	6.4	6.4	6.3	0.17
L-700-0	8.7	9.9	9.2	9.3	0.58
L-560-140	12.1	12.4	12.5	12.3	0.22
L-650-0	14.9	14.4	15.1	14.8	0.36
L-520-130	13.1	12.9	13.4	13.1	0.25
L-600-0	9.0	9.3	11.4	9.9	1.31
L-480-120	9.2	8.8	9.3	9.1	0.23
L-420-180	8.3	8.4	8.5	8.4	0.10

Table B.9: 28-day bulk resistivity results

Mixture identification	28 day bulk resistivity (kΩ-cm)			Average bulk resistivity (kΩ-cm)	Standard deviation
	1	2	3		
H-700-0	5.69	4.49	5.11	5.10	0.60
H-560-140	5.68	4.62	4.54	4.94	0.64
H-650-0	5.20	4.06	5.82	5.02	0.89
H-520-130	6.92	6.22	7.37	6.83	0.58
H-600-0	5.57	4.59	5.53	5.23	0.55
H-480-120	7.41	6.78	7.74	7.31	0.49
H-420-180	10.13	9.51	9.45	9.70	0.37
M-700-0	7.46	6.70	7.40	7.19	0.43
M-560-140	5.25	4.42	4.69	4.79	0.42
M-650-0	7.09	6.63	7.28	7.00	0.34
M-520-130	8.86	8.32	8.10	8.43	0.39
M-600-0	7.39	6.77	7.12	7.09	0.31
M-600P-0	7.36	6.92	7.45	7.24	0.28
M-480-120	6.50	5.81	6.90	6.41	0.55
M-480P-120	5.60	5.02	5.09	5.24	0.32
M-420-180	5.73	4.91	5.65	5.43	0.45
M-420P-180	6.22	5.61	5.60	5.81	0.35
L-700-0	8.12	7.55	7.77	7.81	0.28
L-560-140	10.25	9.96	10.08	10.10	0.15
L-650-0	13.65	13.27	13.60	13.51	0.21
L-520-130	11.84	11.63	11.74	11.73	0.10
L-600-0	8.48	8.03	8.05	8.19	0.25
L-480-120	7.59	7.07	7.62	7.42	0.31
L-420-180	5.45	4.54	6.15	5.38	0.81

Table B.10: 28-day RCPT results

Mixture identification	28 day RCPT (coulombs)		Average RCPT (coulombs)	Standard deviation
	1	2		
H-700-0	4,105	4,463	4,253	253.1
H-560-140	3,647	4,112	3,860	328.8
H-650-0	5,134	4,422	4,687	503.5
H-520-130	4,391	4,709	4,480	224.9
H-600-0	4,250	4,040	4,159	148.5
H-480-120	3,818	3,682	3,766	96.2
H-420-180	3,445	3,709	3,571	186.7
M-700-0	4,566	4,369	4,479	139.3
M-560-140	4,291	4,454	4,354	115.3
M-650-0	3,280	3,698	3,506	295.6
M-520-130	4,379	4,143	4,247	166.9
M-600-0	3,747	4,028	3,943	198.7
M-600P-0	3,932	3,695	3,897	167.6
M-480-120	3,741	3,547	3,632	137.2
M-480P-120	3,837	3,672	3,746	116.7
M-420-180	3,435	3,323	3,391	79.2
M-420P-180	3,376	3,690	3,514	222.0
L-700-0	4,886	4,663	4,766	157.7
L-560-140	3,925	4,212	4,094	202.9
L-650-0	4,147	4,275	4,239	90.5
L-520-130	2,721	2,420	2,532	212.8
L-600-0	3,435	3,651	3,572	152.7
L-480-120	3,058	2,881	2,987	125.2
L-420-180	2,956	2,818	2,879	97.6

Table B.11: Unrestrained shrinkage test results in average percent (%) length change

Mix ID	Percent Change in Length (%) (micro-strain)			
	28-day	8-week	16-week	32-week
H-700-0	0.0312 (312)	0.0382 (382)	0.0424 (424)	0.0504 (504)
H-560-140	0.0301 (301)	0.0376 (376)	0.0424 (424)	0.0937 (937)
H-520-130	0.0286 (286)	0.0342 (342)	0.0439 (439)	-
H-600-0-2	0.0261 (261)	0.0322 (322)	0.0429 (429)	0.0829 (829)
H-480-120-2	0.0258 (258)	0.0329 (329)	0.0420 (420)	0.0683 (683)
H-420-180-2	0.0246 (246)	0.0336 (336)	0.0439 (439)	0.0592 (592)
M-700-0	0.0322 (322)	0.0401 (401)	0.0498 (498)	0.0567 (567)
M-650-0	0.0310 (310)	0.0380 (380)	0.0462 (462)	0.0515 (515)
M-560-140	0.0318 (318)	0.0387 (387)	0.0448 (448)	0.1185 (1185)
M-520-130	0.0304 (304)	0.0389 (389)	0.0389 (389)	-
M-600-0-2	0.0274 (274)	0.0328 (328)	0.0378 (378)	0.0835 (835)
M-480-120-2	0.0279 (279)	0.0339 (339)	0.0401 (401)	0.0788 (788)
M-420-180-2	0.0292 (292)	0.0361 (361)	0.0415 (415)	0.0618 (618)
M-600P-0-2	0.0284 (284)	0.0355 (355)	0.0455 (455)	0.6340 (6340)
M-480P-120-2	0.0287 (287)	0.0348 (348)	0.0415 (415)	0.0638 (638)
M-420P-180-2	0.0269 (269)	0.0333 (333)	0.0390 (390)	0.0570 (570)
L-700-0	0.0314 (314)	0.0414 (414)	0.0513 (513)	-
L-650-0	0.0333 (333)	0.0401 (401)	0.0483 (483)	0.1140 (1140)
L-560-140	0.0347 (347)	0.0447 (447)	0.0546 (546)	-
L-520-130	0.0318 (318)	0.0414 (414)	0.0501 (501)	-
L-600-0-2	0.0298 (298)	0.0371 (371)	0.0430 (430)	0.0703 (703)
L-480-120-2	0.0304 (304)	0.0375 (375)	0.0437 (437)	0.0964 (964)
L-420-180-2	0.0309 (309)	0.0367 (367)	0.0419 (419)	0.0599 (599)

APPENDIX C – SUPPORTING INFORMATION FOR DEVELOPMENT OF SUGGESTED SPECIFICATION PROVISIONS (Chapter 5)

Table C.1: Complete state summary of RCPT and surface resistivity requirements

State/ Standard	RCPT Specification			Resistivity Specification		
	Concrete Type	**Requirement (coulombs)**	**Age**	**Concrete Type**	**Requirement (kΩ-cm)**	**Age**
Virginia DOT design maximum lab permeability Note: [XXXX]* = design maximum lab permeability over tidal waters	A5 prestressed and other special designs	1500 [1500]*	28 days	-	-	-
	A4 general	2500 [2000]*	28 days	-	-	-
	Low shrinkage A4 mod	2500 [2000]*	28 days	-	-	-
	A4 post & rails	2500 [2000]*	28 days	-	-	-
	A3 general	3500 [2000]*	28 days	-	-	-
	A3a paving	3500 [3500]*	28 days	-	-	-
	A3b paving	3500 [3500]*	28 days	-	-	-
	B2 massive or lightly reinforced	NA [NA]*	28 days	-	-	-
	C1 massive unreinforced	NA [NA]*	28 days	-	-	-
	T3 tremie seal	NA [NA]*	28 days	-	-	-
	latex hydraulic cement concrete overlay	1500 [1500]*	28 days	-	-	-
	silica fume, silica fume/class f fly ash or silica fume/slag concrete overlay	1500 [1500]*	28 days	-	-	-
	class F fly ash or slag overlay	1500 [1500]*	28 days	-	-	-
Florida DOT special circumstances. Implemented AASHTO T 358 in January 2017	-	-	-	Ternary blend - extremely aggressive environment	> 29	28 days
	-	-	-	Ternary blend - moderately aggressive environment	17 - 29	28 days
	-	-	-	Ternary blend - slightly aggressive environment	< 17	28 days
	-	-	-	Structural Concretes: Class IV, V, V (special), VI with use of silica fume, ultrafine fly ash, or metakaolin	≥ 29	28 days
	-	-	-	Ultra-high performance repair material for vertical surfaces	≥ 22	28 days
	-	-	-	Special fillers for cathodic protection	Can be 15 or less	28 days
	-	-	-	Special fillers for non-cathodic protection	≥ 22	28 days

New Hampshire DOT (for bridge decks, abutment backwalls) (SRT = surface resistivity test in kΩ-cm)	-	-	-	Class AA (Pay factor 1.05 - 0.06 (10 - SRT))	≥ 5 and ≤ 10	56 days
	-	-	-	Class AA (Pay factor 1.05)	> 10 and ≤ 35	56 days
	-	-	-	Class AA (Pay factor 1.05 + 0.0004347 (150 - SRT))	> 35 and ≤ 150	56 days
	-	-	-	Class AA (Pay factor 1.0)	> 150	56 days
	-	-	-	Prestressed and member concrete	> 15	56 days
Louisiana DOTD structural class concrete	-	-	-	Structural Concretes: Class A1, A2, A3; Prestressed Concretes: Class P1, P2, P3; CIP Structural: Class S	> 22	28 days
	-	-	-	Structural Mass Concretes: Class Mass A1, A2, A3	> 22	56 days
Kansas DOT special provisions	Concrete classified as high chloride risk	> 4000	28 days	Concrete classified as high chloride risk	< 7	28 days
	Concrete classified as moderate chloride risk	2000 - 4000	28 days	Concrete classified as moderate chloride risk	7 - 13	28 days
	Concrete classified as low chloride risk	1000 - 2000	28 days	Concrete classified as low chloride risk	13 - 24	28 days
	Concrete classified as very low chloride risk	100 - 1000	28 days	Concrete classified as very low chloride risk	24 - 190	28 days
	Concrete classified as negligible chloride risk	0 - 100	28 days	Concrete classified as negligible chloride risk	> 190	28 days
New Jersey DOT	-	-	-	HPC Design and Verification Requirements	≥ 36	56 days
	-	-	-	HPC Acceptance Requirements	≥ 19	56 days
	-	-	-	Concrete classified as high chloride risk	< 9	56 days
	-	-	-	Concrete classified as moderate chloride risk	9 - 20	56 days
	-	-	-	Concrete classified as low chloride risk	20 - 48	56 days
	-	-	-	Concrete classified as very low chloride risk	48 - 817	56 days
	-	-	-	Concrete classified as negligible chloride risk	> 817	56 days
New York DOT proposed thresholds for design mix performance	-	-	-	Superstructures and substructures	> 24	28 days
	-	-	-	Footings, piles, drilled shafts, underground applications, sign bases, etc.	> 14	28 days

criteria where specified	-	-	-	Pavements, sidewalks, gutters, curbs, barriers, headwalls, drainage elements, pipe inverts, maintenance repair	> 16.5	28 days
New York DOT performance engineered concrete mixtures for pavements based on application requirements	Pay factor - 100%	≤ 1000	28 days	Pay factor - 100%	≥ 37	28 days
	Pay factor - 87.5%	> 1000 and ≤ 1500	28 days	Pay factor - 87.5%	< 37 and ≥ 27	28 days
	Pay factor - 75%	>1500 and ≤ 2500	28 days	Pay factor - 75%	< 27 and ≥ 19	28 days
	Reject concrete	>2500	28 days	Reject concrete	< 19	28 days
Rhode Island DOT concrete pre-qualification requirements	Structural and prestressed/ precast elements: Class HP	≤ 2000	28 days	Structural and prestressed/ precast elements: Class HP	≥ 15	28 days
	Mass Concrete: Class MC[2]	≤ 3000	28 days	Mass Concrete: Class MC[2]	≥ 15	28 days
	Structural and prestressed/ precast elements: Class HP	≤ 1000	28 day accelerated cure	Structural and prestressed/ precast elements: Class HP	≥ 21	56 days
	Mass concrete: Class MC[2]	≤ 1500	28 day accelerated cure	Mass concrete: Class MC[2]	≥ 21	56 days
Texas DOT	Pavement, structures, and other concrete construction	< 1500	56 days	-	-	-
	Pavement, structures, and other concrete construction	< 1500	28 day accelerated cure	-	-	-
UTAH DOT mix requirements	-	-	-	Class AA (LSF), AA (LS), AA (ES). (AA= bridge decks, LS= low shrinkage, LSF= low shrinkage with fibers, ES = Early strength. AA(LS) used for bridge decks & approach slabs, AA (AE) = other structural elements)	Must have "low to negligible risk" according to AASHTO T 358	
West Virginia DOT supplemental specs	Class S-P concrete (self-consolidating for precast/ prestressed applications	≤ 2000	28 days	-	-	-
	Class S-P concrete (self-consolidating for precast/ prestressed applications	≤ 1500	56 days	-	-	-
	Bridges	< 750	90 days	-	-	-

Montana DOT	-	-	-	Mix trial batches for Class "Deck" (superstructures, deck slabs, barriers) and "Overlay S-F" (silica fume overlays)	> 21	28 days

Table C.2: Bridge mixtures passing/not passing at 28 and 56 days for performance targets 18.0 kΩ-cm and 17.0 kΩ-cm

Target value	Meeting 18.0 kΩ-cm		Not meeting 18.0 kΩ-cm		Meeting 17.0 kΩ-cm		Not meeting 17.0 kΩ-cm	
Age	28 day	56 day	28 day	56 day	28 day	56 day	28 day	56 day
Mixture identification	CF	H-520-130	H-700-0	H-700-0	CF	H-520-130	H-700-0	H-700-0
		M-520-130	H-650-0	H-650-0		M-520-130	H-650-0	H-650-0
		L-520-130	M-700-0	M-700-0		L-650-0	M-700-0	M-700-0
		CF	M-650-0	M-650-0		L-520-130	M-650-0	M-650-0
			L-700-0	L-700-0		CF	L-700-0	L-700-0
			L-650-0	L-650-0		CC	L-650-0	H-560-140
			CC	CC			CC	M-560-140
			H-560-140	H-560-140			H-560-140	L-560-140
			H-520-130	M-560-140			H-520-130	
			M-560-140	L-560-140			M-560-140	
			M-520-130				M-520-130	
			L-560-140				L-560-140	
			L-520-130				L-520-130	

* Note: Mixture CC is the control straight cement AA mixture and mixture CF is the control fly ash mixture from RP 2016-06 (Cavalline et al. 2019).

Table C.3: Bridge mixtures passing/not passing at 28 and 56 days for performance targets 16.0 kΩ-cm and 15.0 kΩ-cm

Target value	Meeting 16.0 kΩ-cm		Not meeting 16.0 kΩ-cm		Meeting 15.0 kΩ-cm		Not meeting 15.0 kΩ-cm	
Age	28 day	56 day	28 day	56 day	28 day	56 day	28 day	56 day
Mixture identification	CF	H-520-130	H-700-0	H-700-0	CF	H-520-130	H-700-0	H-700-0
	CC	M-520-130	H-650-0	H-650-0	CC	M-520-130	H-650-0	H-650-0
		L-520-130	M-700-0	M-700-0		L-520-130	M-700-0	M-700-0
		CF	M-650-0	M-650-0		CF	M-650-0	M-650-0
		L-650-0	L-700-0	L-700-0		L-650-0	L-700-0	L-700-0
		CC	L-650-0	H-560-140		CC	L-650-0	H-560-140
		L-560-140	H-560-140	M-560-140		L-560-140	H-560-140	
			H-520-130			M-560-140	H-520-130	
			M-560-140				M-560-140	
			M-520-130				M-520-130	
			L-560-140				L-560-140	
			L-520-130				L-520-130	

* Note: Mixture CC is the control straight cement AA mixture and mixture CF is the control fly ash mixture from RP 2016-06 (Cavalline et al. 2019).

Table C.4: Bridge mixtures passing/not passing at 28 and 56 days for performance targets 14.0 kΩ-cm and 13.0 kΩ-cm

Target value	Meeting 14.0 kΩ-cm		Not meeting 14.0 kΩ-cm		Meeting 13.0 kΩ-cm		Not meeting 13.0 kΩ-cm	
Age	28 day	56 day	28 day	56 day	28 day	56 day	28 day	56 day
Mixture identification	CC	L-650-0	H-700-0	H-700-0	CC	L-650-0	H-700-0	H-700-0
	CF	CC	H-650-0	H-650-0	CF	CC	H-650-0	H-650-0
	L-650-0	H-520-130	M-700-0	M-700-0	L-650-0	H-520-130	M-700-0	M-700-0
		M-560-140	M-650-0	M-650-0	L-520-0	M-560-140	M-650-0	M-650-0
		M-520-130	L-700-0	L-700-0		M-520-130	L-700-0	L-700-0
		L-560-140	H-560-140			L-560-140	H-560-140	
		L-520-130	H-520-130			L-520-130	H-520-130	
		CF	M-560-140			CF	M-560-140	
		H-560-0	M-520-130			H-560-0	M-520-130	
			L-560-140				L-560-140	
			L-520-130					

* Note: Mixture CC is the control straight cement AA mixture and mixture CF is the control fly ash mixture from RP 2016-06 (Cavalline et al. 2019).

Table C.5: Bridge mixtures passing /not passing at 28 and 56 days for performance targets 13.0 kΩ-cm and 12.0 kΩ-cm

Target value	Meeting 13.0 kΩ-cm		Not meeting 13.0 kΩ-cm		Meeting 12.0 kΩ-cm		Not meeting 12.0 kΩ-cm	
Age	28 day	56 day	28 day	56 day	28 day	56 day	28 day	56 day
Mixture identification	CC	L-650-0	H-700-0	H-700-0	CC	L-650-0	H-650-0	H-650-0
	CF	CC	H-650-0	H-650-0	CF	CC	M-700-0	M-700-0
	L-650-0	H-520-130	M-700-0	M-700-0	L-650-0	H-520-130	M-650-0	L-700-0
	L-520-0	M-560-140	M-650-0	M-650-0	L-520-0	M-560-140	L-700-0	
		M-520-130	L-700-0	L-700-0	M-520-130	M-520-130	H-560-140	
		L-560-140	H-560-140		L-560-0	L-560-140	H-520-130	
		L-520-130	H-520-130			L-520-130	M-560-140	
		CF	M-560-140			CF	H-700-0	
		H-560-0	M-520-130			H-560-140		
			L-560-140			H-700-0		
						M-650-0		

* Note: Mixture CC is the control straight cement AA mixture and mixture CF is the control fly ash mixture from RP 2016-06 (Cavalline et al. 2019).

Table C.6: Bridge mixtures passing and failing at 28 and 56 days for performance target 11.0 kΩ-cm

Target value	Meeting 11.0 kΩ-cm		Not meeting 11.0 kΩ-cm	
Age	28 day	56 day	28 day	56 day
Mixture identification	CC	L-650-0	H-700-0	H-650-0
	CF	CC	H-650-0	M-700-0
	L-650-0	H-520-130	M-700-0	L-700-0
	L-520-0	M-560-140	M-650-0	
	M-520-130	M-520-130	L-700-0	
	L-560-0	L-560-140	H-560-140	
		L-520-130	H-520-130	
		CF	M-560-140	
		H-560-0		
		H-700-0		
		M-650-0		

Table C.7: Analysis of pavement and structural mixtures passing selected volumetric shrinkage targets at 28 days

Target Values	350με	420με	600με
Age	28 days		
Mixtures passing target value	H-600-0	M.B.N.M.	P.BL.N.M
	M-600-0	P.B.N.N.	
	M-600P-0	P.B.N.M.	
	L-600-0	M.BL.N.M.	
	C.A.N.M		
	M.A.N.M		
	P.A.N.M		
	P.A.N.N		
	C.B.N.M		
	C.BL.N.M		
	P.BL.N.N		
	H-480-120		
	M-480-120		
	M-480P-120		
	L-480-120		
	P.A.A.M		
	P.A.B.M		
	P.B.A.M		
	P.B.B.M		
	P.BL.A.M		
	P.BL.B.M		
	H-420-180		
	M-420-180		
	M-420P-180		
	L-420-180		
Percent Passing	83%	97%	100%

Table C.8: Pavement and structural mixtures meeting and not meeting 350 με performance target at 28 days

Target Values	Meeting 350με target	Exceeding 350με target
Age	28 days	28 days
Mixture Identification	H-600-0	M.B.N.M
	M-600-0	P.B.N.M
	M-600P-0	P.B.N.N
	L-600-0	M.BL.N.M
	C.A.N.M	P.BL.N.M
	M.A.N.M	
	P.A.N.M	
	P.A.N.N	
	C.B.N.M	
	C.BL.N.M	
	P.BL.N.N	
	H-480-120	
	M-480-120	
	M-480P-120	
	L-480-120	
	P.A.A.M	
	P.A.B.M	
	P.B.A.M	
	P.B.B.M	
	P.BL.A.M	
	P.BL.B.M	
	H-420-180	
	M-420-180	
	M-420P-180	
	L-420-180	

Table C.9: Pavement and structural mixtures meeting and not meeting a 420 με volumetric shrinkage performance target at 28 days

Target Values	Under 420με target	Not Under 420με target
Age	28 days	28 days
Mixture Identification	H-600-0	P.BL.N.M
	M-600-0	
	M-600P-0	
	L-600-0	
	C.A.N.M	
	M.A.N.M	
	P.A.N.M	
	P.A.N.N	
	C.B.N.M	
	C.BL.N.M	
	P.BL.N.N	
	H-480-120	
	M-480-120	
	M-480P-120	
	L-480-120	
	P.A.A.M	
	P.A.B.M	
	P.B.A.M	
	P.B.B.M	
	P.BL.A.M	
	P.BL.B.M	
	H-420-180	
	M-420-180	
	M-420P-180	
	L-420-180	