DEVELOPMENT OF A SURFACE RESISTIVITY SPECIFICATION FOR DURABLE CONCRETE

by

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ABSTRACT

ROBERT BLAKE BIGGERS. Development of a Surface Resistivity Specification for Durable Concrete. (Under the direction of DR. TARA L. CAVALLINE)

Surface resistivity testing is a state-of-the-art method of testing concrete's ability to resist chloride ion penetration. This electrical test is performed by passing an electrical current through a saturated concrete specimen, and has recently gained steady support as a superior testing method to currently specified durability tests such as the rapid chloride permeability test (RCPT) which is time consuming and exhibits significant variation. Over the past two decades research and implementation of surface resistivity has provided insight into the benefits of this test, namely cost and time savings, for durability testing of concrete. The goals of this research were to evaluate the factors affecting surface resistivity measurements of concrete produced with North Carolina materials, as well as to identify target values indicative of durable concrete performance for North Carolina Department of Transportation (NCDOT) performance-based specifications.

Twenty-four different concrete mixtures were produced using materials typical of that specified by the NCDOT for use in concrete bridge and pavement construction. The mixtures included variations to the w/cm, cementitious material content, fly ash replacement percentage, and substitution of a portland limestone cement. Mixtures were also proportioned in a way that would represent a range of designs typical of concretes used in both structural (e.g., bridge) and pavement construction. These mixtures were used to evaluate fresh and hardened mechanical and durability properties, and to support the development of a surface resistivity specification and performance targets.

Compared to mixtures with higher water to cementitious material (w/cm) ratios, test results for mixtures with lower w/cm showed the generally established trend that lower w/cm ratios provide benefits to both mechanical and durability performance. Mixtures including fly ash also exhibited superior durability performance, although exhibit delayed development of mechanical strength at early ages as expected. In order to allow these fly ash mixtures to develop the proven improved durability performances, revision of the NCDOT 28 day 4,500 psi compressive strength specification should be considered. NCDOT's decision to increase allowable fly ash replacement rates from 20% to 30% should have little to no impact to mechanical properties at later ages. NCDOT's decision to allow use of portland limestone cement (PLC) in mixtures was supported, as the influence of PLC on mechanical and durability properties was not significant. Mixtures with improved durability performance could be promoted by NCDOT through specification provisions promoting lower w/cm mixtures, which could readily be achieved through use of WRAs, fly ash, and optimized aggregate gradations.

Results of this study were used to develop a surface resistivity specification for use by NCDOT. Performance specifications 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. The implementation of a surface resistivity specification could prove to be beneficial in the areas of early-age durability indication, as well as time and cost savings for use in future applications for North Carolina's roadway and bridge infrastructure.

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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF ABBREVIATIONS

AAR	Alkali-Aggregate Reaction
AASHTO	American Association of State Highway and Transportation Officials
AC	Alternating Current
ACI	American Concrete Institute
ACPA	American Concrete Pavement Association
AEA	Air entraining admixture
ASCE	American Society of Civil Engineers
ASR	Alkali-Silica Reaction
ASTM	American Society for Testing and Materials
C3S	Tricalcium silicate
$Ca(OH)_2$	Calcium hydroxide
cf	cubic feet
COV	Coefficient of Variance
cwt	hundredweight
DC	Direct Current
DOT	Department of Transportation
FHWA	Federal Highway Administration
F/T	freeze/thaw
g	grams
in	inch

ITZ	interfacial transitition zone
Kg	kilograms
KOH	Potassium hydroxide
kΩ-cm	kilohms per centimeter
LADOTD	Louisiana Department of Transportation and Development
mm	Millimeter
MOE	Modulus of Elasticity
MOR	Modulus of Rupture
NaOH	Sodium hydroxide
NCC	National Concrete Consortium
NCDOT	North Carolina Department of Transportation
NCPTC	National Concrete Pavement Technology Center
OPC	ordinary portland cement
PCA	Portland Cement Association
pcf	pounds per cubic foot
pcy	pounds per cubic yard
PEM	Performance Engineered Mixture
PLC	portland limestone cement
Psi	Pounds per Square Inch
QA/QC	Quality Assurance/Quality Control
RCPT	Rapid Chloride Permeability Test
R^2	Coefficient of determination
SAM	Super Air Meter

SCMs	Supplementary Cementitious Materials
SHA	State Highway Administration
TRB	Transportation Research Board
VDOT	Virginia Department of Transportation
WRA	water reducing admixture
w/cm	Water to Cementitious Material Ratio
ºF	Degrees Fahrenheit

CHAPTER 1: INTRODUCTION

1.1 Background and Significance

Concrete accounts for between 50% and 75% by weight of America's infrastructure (ASCE 2019). Concrete has been heavily utilized in building and infrastructure construction in all regions of the country, and has been for nearly one hundred years. A construction boom occurred in America following World War II, creating a large portion of America's infrastructure that is still in use. The majority of the concrete structures and systems built during this time were designed to have a useful life of roughly fifty years (ASCE 2019). This lifespan can vary based on factors such as concrete type, application, and use.

Regardless of variations in exact useful life, these structures are nearly all approaching the end of their respective lifespans. Federal funding for infrastructure projects has been on a decline in the last three decades. At its highest point, federal spending on transportation and water infrastructure equated for approximately 6% of federal spending. As of 2017, federal spending for the same category accounted for only about 2.5% (Congressional Budget Office 2018). According to the American Society of Civil Engineers (ASCE) infrastructure report card, the collective grade assigned for America's infrastructure is a "D+" on a typical A-F grading scale, which gives a "poor, at risk" evaluation (ASCE 2019). This score is indicative of the declining state of America's infrastructure when compared to the "C" grade given in 1988, the first year that infrastructure evaluation was performed in this manner. In 2001, the ASCE reported an estimated $1.3 trillion dollar investment would be required over a five year period to make the necessary improvements to America's infrastructure. In the year 2017, the reported ten year investment needed to make the necessary improvements would be roughly $4.59 trillion dollars (ASCE 2019).

A combination of environmental and economic reasons has led the Federal Highway Administration (FHWA) to encourage and fund research and technology transfer tools to support implementation of performance specifications and methodologies that can provide a more durable infrastructure (FHWA 2019). The Performance Engineered Mixture (PEM) initiative aims to update the outdated practices specified to bring specifications up to date with the advanced materials, testing methods, and other available resources of today (Cackler et al. 2017). Many of these advancements will be discussed in Chapter 2 of this thesis. In addition to the FHWA and many state transportation agencies, a number of other agencies are involved in promoting the PEM initiative, including industry groups such as American Concrete Paving Association (ACPA), Portland Cement Association (PCA), and other members of the National Concrete Consortium (NCC). Currently, a number of researchers at universities and other entities are involved in conducting research supporting PEM concrete.

The American Association of State Highway and Transportation Officials (AASHTO) has published a standard practice document, AASHTO PP 84, "Standard Practice for Developing Performance Engineered Concrete Pavement Mixtures", supporting the efforts of the PEM movement. AASHTO PP 84 outlines better ways of designing, accepting, and testing concrete through a collection of specifications that provide performance-based alternatives to the traditional prescriptive specification approaches. Prescriptive specifications (also known as materials specifications, methods specifications, or "recipe" specifications) are those that outline means and methods, materials, and characteristic requirements. Alternatively, performance specifications are a set of requirements related to functionality based upon specific application. Performance specifications also include suggested methods for testing and criteria for acceptance to uphold the requirements for the application (NRMCA 2019). AASHTO PP 84 includes prescriptive as well as alternative performance specifications for materials, sampling, proportioning, acceptance, quality control, and reporting recommendations. Although AASHTO PP 84 was initially designed for use on concrete pavement specifications, the suggested methodologies can be applied to other concrete applications, such as bridge decks.

Concrete has long been specified based upon slump, air content, and compressive strength. Although these three parameters are important, they cannot reliably be linked to producing concrete that is constructible or durable over a service life (Cackler et al. 2017). Similar to many other state highway agencies, the North Carolina Department of Transportation (NCDOT)'s effort to transition from prescriptive to performance based concrete specifications include two major factors, appropriate material selection and proportioning, and tests for acceptance criteria. Performance based specifications for concrete allow contractors the flexibility to develop mixture designs and produce concrete meeting performance requirements, while also allowing the contractor and supplier to economize and optimize mixture designs for cost and sustainability (Cackler et al. 2017). Specifications based on performance criteria also consider construction related challenges, such as workability and issues related to temperature.

The NCDOT has recognized the large disconnect between design and testing methods, a large portion of which were developed many years ago, and wishes to modify QA/QC protocols and specifications to fulfill the desires and requirements of today. Research has been conducted and is ongoing to explore various design and testing methods to improve future repairs and additions to North Carolina's infrastructure (Cavalline et al. 2013, 2018, 2019). A major goal is to reduce the occurrences of concrete structures and pavements not meeting their designed useful life span, extending the lifespan of newly constructed bridges and pavements, and reducing maintenance and repair actions.

One key characteristic influencing the rate at which structures deteriorate is the durability performance of the concrete. Durability related issues, often caused by ingress of aggressive agents (such as salts and sulfates) or material incompatibilities, often result in the need to perform major repairs, and in some cases can cause premature failure of the structure (TRB 2013). Most conventional test methods to evaluate concrete durability are fairly slow and tedious, but also cannot be evaluated until well after the concrete has been placed. The link between test results and field performance is also not clear, in some instances (ASTM 2018, Rupnow and Icenogle 2012). The need for more reliable, rapid testing methods (discussed in Chapter 2 of this document) along with the knowledge that durability has a major bearing on long-term structure health is driving the desire to move toward specifying concrete based on performance-based measures.

1.2 Introduction to Electrical Measurements for Durability Assessment

There are many factors that can have an influence on the durability of concrete pavements. Characteristics such as permeability, resistance to freeze/thaw (F/T) deterioration, and resistance to chemical attack have a large impact on concrete's durability (TRB 2013, Mehta and Montiero 2014, Kosmatka et al. 2011). The major issue pertaining to evaluation of concrete durability is that the related tests take weeks to be performed and evaluated. This means that if test results indicate the durability of a concrete structure is going to be poor, the possibility of the structure already experiencing problems, or experiencing problems in the future is more likely. Costs associated with corrective measures to improve likelihood of success for a structure also increase greatly as the concrete hardens in place. Rapid tests to evaluate durability at a younger structure age, or (more optimally) prior to construction have begun to emerge, which is a major point of focus for this research.

Concrete durability can be evaluated in a number of ways, with one of the most common methods involving attempting to pass a substance with known properties through the concrete, and measuring the rate at which these substances pass. Resistance to electrical current and to chloride penetration are two of the most tested characteristics for durability. The Rapid Chloride Permeability Test (RCPT), as outlined in American Society for Testing and Materials (ASTM) standard C1202, "Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration" and AASHTO T 277, "Standard Method of Test for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration," is used to evaluate concrete's ability to resist chloride ingress, since chloride is a major contributor to corrosion (Whiting 1981). In this test, a 60 volt electrical current is passed through a 50mm slice of a concrete core (ASTM 2018). The slice has both faces exposed to a chemical solution, one end being a solution of sodium chloride, and the other containing a solution of sodium hydroxide.

The charge passed through the specimen is measured in coulombs, which provides an indication of the concrete's resistance to chloride ion penetration (AASHTO 2015).

The surface resistivity test is similar to the RCPT test in that it uses electrical current passed through saturated concrete to get an indication of the concrete's resistance to chloride ion penetration. The test is performed by passing an AC current along the surface of a concrete cylinder by means of a Wenner probe. The Wenner probe has four pins, with the outer two pins generating a current flow along the pins. The inner two pins are used for measuring the resultant potential difference, which is measured in kilohms per centimeter (kΩ-cm) to get an indication of the resistance to chloride ion penetration. A test method for use of the Wenner probe to measure concrete's surface resistivity has been developed into AASHTO Standard T 358-17, "Surface Resistivity Indication of Concrete's Ability to Resist Chloride Ion Penetration (AASHTO 2017)." Testing of bulk electrical conductivity of concrete using plate electrodes has also been found to be related to measures found through RCPT testing. A test method for determining bulk conductivity is outlined in ASTM C1760, "Standard Test Method for Bulk Electrical Conductivity of Hardened Concrete." The resistivity of concrete has been shown to be influenced by several components of the concrete mixture including aggregate size and type (Morris et al. 1996), sample geometry, temperature, moisture conditions, probe geometry (Spragg et al. 2011), curing conditions, and surface moisture (Kessler et al. 2008).

Many states have achieved success implementing specifications with various surface resistivity targets as measured by AASHTO T 358, "Standard Method of Test for Surface Resistivity Indication of Concrete's Ability to Resist Chloride Ion Penetration"

(AASHTO 2017). Often, these specifications provide different resistivity targets for different types of mixtures (e.g., structural concrete, pavement concrete, self-consolidating concrete, etc.). Due to increasing use of supplementary cementitious materials (SCMs) such as fly ash, which slow the development of both strength and electrical resistance, current resistivity specifications often use specimens tested at ages beyond the conventional 28 days (such as ages of 56 or 90 days) to accommodate the slower SCM hydration.

Over the past several decades, concrete mixtures have become more complex, often involving multiple SCMs, slag, or interground limestone (PLC), and other ternary blends. The composition of materials such as fly ash is variable, and changes with coal combustion and processing technology. These changes can potentially impact concrete's pore system characteristics (e.g. size, volume), interconnectivity of the pores, and pore solution chemistry (Weiss 2014). Therefore, the electrical resistance of concrete's pore solution may impact the measured (bulk) resistance of a concrete specimen. This suggests that specifications currently specifying resistivity measurements per AASHTO T 358 may be improved by adjusting measurements for pore solution resistivity. Research exploring the role of pore solution chemistry and concrete permeability on concrete resistivity is currently a focus of ongoing study (Weiss 2019). The ratio between the resistivity of the bulk concrete to the resistivity of the pore solution (known as the formation factor) has been a suggested approach to modifying the measured (bulk) resistivity of concrete specimens, and is specified in AASHTO PP 84 (AASHTO 2017).

1.3 Objectives and Scope

The objective of this research was to gain a better understanding of the resistivity measurements indicative of quality concrete for use in pavements and bridges using materials and mixture proportions typical of those used in North Carolina. The first objective of this research was to test concrete to identify the effect various mixture materials and proportions had on strength, rapid chloride penetrability, and resistivity. Additionally, a preliminary dataset was created to explore the ability to determine the formation factor per the procedure recommended for use in AASHTO PP 84. Data collected from the laboratory testing portion of this project was supplemented with selected test results from previous UNC Charlotte concrete research projects using typical NCDOT highway mixtures and materials. Data analysis was performed to understand the influence of several mixture design and proportioning characteristics (water to cement ratio (w/cm), cementitious content, and fly ash replacement rate) on strength, rapid chloride permeability, and resistivity test results.

In conjunction with the data analysis portion on representative North Carolina paving and bridge mixtures, a review of RCPT and resistivity specifications implemented by various departments of transportation (DOTs) was performed. Findings of a review of state RCPT resistivity specifications, along with results of analysis of data obtained by this research team, were used to identify target values for acceptable resistivity testing results for use in concrete bridge and pavement projects for North Carolina. Concurrent with this laboratory testing, surface resistivity testing was performed by an industry partner at a pilot project (reconstruction of a 5-mile stretch of I-85 outside of Charlotte, NC). Data obtained in the field, along with input from the industry partner regarding testing frequency and other practical recommendations, was used to assist in the development of a draft specification. As part of the draft specification, recommendations for the NCDOT to move toward implementing resistivity testing as a quality assurance and quality control (QA/QC) tool to support the durable and sustainable concrete initiative were developed.

CHAPTER 2: LITERATURE REVIEW

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 (TRB 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. The idea of improved material selection and testing programs to support durable concrete is the basis for AASHTO’s desire to move from prescriptive specifications to performance-based speculations (AASHTO 2017). The PEM initiative, supported by FHWA, aims to tackle two of the most common causes of failure in concrete pavements: cracking and deterioration caused by deleterious substances.

To be durable, concrete must withstand distress from a variety of aggressive agents and environmental conditions, as well as service loads. The PCA Design and Control of Concrete Mixtures book defines concrete durability 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). Tests to evaluate these durability factors provide an indication of concrete's ability to withstand the deterioration mechanisms likely to cause premature failure. 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 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.

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.

2.1.1 Performance Requirements for Durable Concrete

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). 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). 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.

2.1.1.1 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 (ACI 2013). Plastic shrinkage is related to moisture loss in the concrete. The most common way to combat moisture loss in concrete is the curing process. Curing of concrete is accomplished in a variety of ways including covering the concrete with a material saturated in water, misting of concrete, and membrane curing. If moisture loss is significant, tensile stress increases along the surface of the concrete (ACI 2008). Settlement cracks typically occur following the initial set of concrete. The most common causes of settlement cracks are reinforcing placement, poor concrete consolidation, and often bleeding. As denser mixture components begin to segregate with gravity, settlement cracks can begin to appear as concrete is setting. Cracking is directly related to increased permeability as air and liquids can penetrate the surface into the concrete's mass. ASTM C1581, "Standard Test Method for Determining Age at Cracking and Induced Tensile Stress Characteristics of Mortar and Concrete under Restrained Shrinkage" outlines one type of procedure (restrained cracking) for testing to evaluate age and tensile strengths at which restrained concrete cracks (ASTM 2018). Another more commonly utilized test for linear (volumetric) shrinkage is ASTM C157, "Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete", 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 2017, Weiss 2017).

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 corroding. 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.

2.1.1.2 Resistance to Deleterious Substances

Transport of fluids and gases into concrete through durability-related distresses in concrete often containing chloride ions for corrosion or sulfate ions for sulfate attack (AASHTO 2017). Until recently, it has been difficult to determine permeability and resistivity of concrete. These tests often are not performed until later ages, which can lead to removal and rework of concrete with poor results. The PEM initiative suggests durability tests that provide better insight into concrete's ability to resist transport of deleterious substances from the surface into the structure (AASHTO 2017). Absorption testing, chloride ion penetration testing, and electrical surface resistivity testing are tests demonstrated to be more indicative of long-term durability than currently specified mechanical tests (these tests are described subsequently in Section 3.8.1). The effects of deleterious substances on concrete are dependent largely on pore structure within the concrete. The rate, extent, and effect of the transport are also influenced by presence of cracks and the microclimate at the surface of the concrete (ACI 2008).

2.2 Characteristics of Durable Concrete

Concrete durability is characterized by a number of performance measures. The AASHTO PP 84 specification for performance engineered concrete presents a list of characteristics that influence the durability of the concrete. 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 main focus of achieving durability (AASHTO 2017). Additionally, properties such as low absorption, diffusion, and other transport related properties have an influence on the pavement's durability. These factors and their associated performance targets must be carefully considered across all phases of the concrete's life. AASHTO PP 84 provisions can be used in specifications to address design considerations, mixture qualification testing, as well as acceptance testing performed during and after construction. Details for selection of which test are also outlined (AASHTO 2017).

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).

Section 6.4 of 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. Section 6.4.1 of 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 2017). ASTM C157 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. AASHTO T 160, “Standard Method of Test for Length Change of Hardened Hydraulic Cement Mortar and Concrete” is suggested for unrestrained volume change 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. Restrained shrinkage testing is 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.” If specifying using AASHTO T 334, the target value is 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.

Freeze-thaw durability is influenced by the concrete’s w/cm ratio, fresh air content (and the air matrix produced), time of critical saturation, and damage caused by deicing salts or calcium oxychlorides (ASTM 2015). AASHTO PP 84 Section 6.5 provides both prescriptive and performance specifications for achieving proper freeze-thaw durability. The prescriptive specifications are outlined in AASHTO PP 84 Section 6.5.1, including w/cm, air content, and Super Air Meter (SAM) number recommendations. AASHTO PP 84 Section 6.5.1.1 identifies that a w/cm ratio of less than 0.45, as well as provisions from either Section 6.5.1.2 or Section 6.5.1.3 be used. Section 6.5.1.2 specifies an air content between 5% - 8% using AASHTO T 152, "Standard Method of Test for Air Content of Freshly Mixed Concrete by the Pressure Method", AASHTO T 196, "Standard Method of Test for Air Content of Freshly Mixed Concrete by the Volumetric Method", or AASHTO TP 118, "Standard Method of Test for Characterization of the Air-Void System of Freshly Mixed Concrete by the Sequential Pressure Method" (AASHTO 2015, 2017). Section 6.5.1.3 specifies an air content greater than 4% as well as a SAM number lower than 0.20 found using AASHTO TP 118. The performance specification for achieving freeze-thaw durability is found in PP 84 Section 6.5.2.1, which identifies testing for time to critical saturation as found using the Bucket Test. The Bucket Test method was developed by researchers at Oregon State University (Weiss 2019). Typically the target for time to critical saturation as 30 years, however can differ per agency preferences.

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 2017). 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 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 can be found in Table 2.1. 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, 2017).

These specifications for transport properties include a set of prescriptive and performance-based specifications for evaluating transport properties, as well as suggested performance targets. The specification includes a list of various tests and values for acceptance. The inclusion of typical prescriptive specifications along with suggested performance-based characteristics allows the designer to select specification provisions and performance thresholds based upon their preferences (or agency preferences) (AASHTO 2017). The advantage of this specification approach is that it allows for the designer to specify performance parameters that are best suited for the conditions the concrete pavements will be exposed to. The prescriptive portion of the section includes choosing a water cement ratio related to the possible deicer applications and freezing and thawing conditions the concrete may undergo. Alternatively, design can be based on testing requirements related to the concrete’s F factor value. This F factor value is related to the results found from electrical resistivity testing. The values required can be found in Table 2.1. A performance based alternative is also presented in section 6.6 of the AASHTO PP 84 specification. This performance specification relates to choosing an F factor that is determined based on the desired depth of ionic penetration given ionic exposure during a specified service life.

Table 2.1 Equivalent values for F factor, RCPT, and resistivity

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	kΩ-cm	kΩ-cm
High	500	-	4,000	-	5	-
Moderate	1,000	500	2,000	4,000	10	5
Low	2,000	1,000	1,000	2,000	20	10
Very Low	20,000	2,000	100	1,000	200	20
Negligible	-	20,000	0	100	-	200

AASHTO PP 84 Section 6.7 outlines tests to evaluate aggregate stability in regard to durability. Aggregate stability is a function of its cracking tendency and AAR potential (AASHTO 2017). Potential susceptibility to freeze-thaw should be tested using ASTM C1646, "Standard Practice for Making and Curing Test Specimens for Evaluating Resistance of Coarse Aggregate to Freezing and Thawing in Entrained-Air Concrete" and AASHTO T 161, "Standard Method of Test for Resistance of Concrete to Rapid Freezing and Thawing", or a local State Highway Administration (SHA) practice (ASTM 2016, AASHTO 2017). Susceptibility to deleterious AAR reactions should be tested using AASHTO R 80, "Standard Practice for Determining the Reactivity of Concrete Aggregates and Selecting Appropriate Measures for Preventing Deleterious Expansion in New Concrete." If alkali-silica reactions (ASR) are of concern, the mitigation strategies in AASHTO R 80 should also be followed (AASHTO 2017).

Although not typically considered as a durability characteristic, AASHTO PP 84 Section 6.8 addresses testing for workability evaluation. Concrete that is not workable causes difficulties in the placement of the concrete, and therefore can cause poor construction and durability performance. The box test method (Cook et al. 2014), and the

Vibrating Kelly Ball (VKelly) test (Wang et al. 2017) are the two specified methods of testing for workability. A box test should produce a ranking of 2 or less, and the VKelly should have results between 15 and 30 millimeters (mm).

2.2.1 Materials

Material selection should be made to 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 et al. 2011). 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. Typical aggregates used in portland cement concrete are dense and inert. This results in water and other liquids moving within the concrete paste. With an appropriately graded aggregate system, the volume of paste can be reduced, decreasing permeability (TRB 2013).

The area in which aggregates and paste meet is called the interfacial transition zone (ITZ), This ITZ is influenced by aggregate composition and size, and is considered to be an at-risk area, as increases in the ITZ size results in an increase in permeability (Mehta and Monteiro 2014). 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. Figure 2.1 serves as a guide to evaluate aggregates and determine risk of an AAR (ACI 2008). When considering aggregates, properties such as gradation, specific gravity, absorption, particle shape/angularity, abrasion/impact resistance, chemical stability, and chemical composition should be considered (ACI 2007).

Figure 2.1 Guide for determination of AAR risk in aggregates

Cement type selected for a concrete element is based upon the concrete's proposed application. Cements should be chosen in a manner which supports long-term durability in addition to mechanical strength. The emergence of SCMs as a replacement for portions of cement has further complicated the discussion regarding cement's role in concrete. SCMs such as fly ash, silica fume, and slag have proven to be adequate, and in fact beneficial, as a cement replacement. SCMs can improved mechanical properties, and generally provide improved durability characteristics as well (Papadakis 2000). In addition to the concrete durability performance benefits resulting from substitution of SCMs for ordinary Portland cement (OPC), many of the beneficial SCMs approved for use in concrete are a byproduct of another industry, providing sustainability benefits (Juenger and Siddique 2015).

Material selection has a direct impact on the permeability of concrete. The paste content (quantity of paste) and the quality of the paste in concrete are two main factors that can affect permeability characteristics. Cementitious materials, including SCMs can produce denser paste structures, and therefore less permeable concretes (Gesoglu et al. 2009). Lower w/cm are also attributed to lower water and electrical permeability (Lomboy and Wang 2009). As concrete cures, pore structures become finer and less permeable as found in a study by Cui and Cahyadi (2001).

Admixtures have become a more important input in concrete design as design expectations have increased over the years. Admixtures can provide benefits to several performance characteristics of concrete, from workability in fresh concrete to color to air content (Kosmatka et al. 2011). 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.

Water reducing admixtures (WRAs) are also commonly used in industry. A water reducer allows for mixtures with low w/cm to remain workable for contractors. 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. It is important to consider interaction between admixtures when considering use in a design, as adverse effects could occur (Kosmatka et al. 2011).

2.2.2 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 et al. 2011). The first 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.

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. 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. 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 benefits to concrete durability and internal structure.

In regard to w/cm ratio, only 30% water by weight of cement is needed to hydrate plain cement particles (Mehta and Monteiro 2014). As seen in the laboratory portion of this project, mixtures with w/cm this low are difficult to consolidate and place. 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 in Section 2.2.1, 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 et al. 2011).

2.2.3 Construction

Performance engineered concrete 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, and pumpability are characteristics considered in performance engineered concrete mixture designs (Ley et al. 2014, Cook et al 2014, Wang et al. 2017). Durable concrete should be handled with care and placed/finished in accordance with project specifications and quality assurance provisions. 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.

Construction factors such as water added on-site, placement, and curing measures have a direct influence on concrete pore structure. 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 (Kosmatka et al. 2011). 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 et al. 2011). The handling and treatment of concrete can also affect the air void systems in concrete. 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). 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).

2.3 Tests to Evaluate Concrete Durability

Tests to evaluate concrete durability performance often take much more time to perform than mechanical property tests. The longer duration and technical challenges associated with these tests is 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 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). 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. 2017, AASHTO 2017).

Durability tests traditionally aim to evaluate concrete's ability to conduct three mediums: air, water, and electricity. 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 in regard to moisture saturation level, pore chemistry, and temperature) can greatly influence durability testing results. Pore structure development, air void system formation, material mixing, placement and consolidation, curing, and minimizing cracking all occur early in concrete placement and testing (ACI 2008).

2.3.1 Permeability Tests

Corrosion of the reinforcing steel (which improves concrete tensile strength) 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. It is believed by many that permeability may be the most important factor related to concrete durability (Baykal 2000, TRB 2013). 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).

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 is. Particle packing by means of optimized gradations and use of SCMs allow designers to develop concretes less susceptible to penetration. Capillary action also has a direct influence on concrete permeability. The tendency of concrete with high absorption properties to absorb water leads to the need to produce less permeable concrete in areas where water, and aggressive agents such as deicing salts, are present. ASTM C1585, "Measurement of Rate of Absorption of Water by Hydraulic-Cement Concretes" attributes absorption at the surface of a concrete pavement to mixture proportions, admixtures and SCMs, composition of the cement and aggregates, entrained air content, curing, degree of hydration/age, presence of cracks, presence of sealants, and placement methods including consolidation and finishing (ASTM 2013).

Water absorption testing is used to determine the permeability of concrete in regard to water. ASTM C1585 outlines the test procedures for finding the concrete's sorptivity by means of ponding water on one surface of a specimen. This is a more representative measure of absorption in relation to a concrete pavement, as saturation would occur from primarily the surface. This is an issue in itself, as the quality of the surface of concrete is highly influenced by the contractor's finishing methods. Improper finishing and curing of concrete are two mistakes commonly made by contractors which have direct effect on durability. Improper finishing of concrete can result in bleed water being worked back into the concrete, but only at the surface. This increased w/cm ratio and decreased air content lead to an increase in concrete permeability, making it more susceptible to deterioration (ACI 2008). Insufficient curing is a product of the desire to put concrete pavements into service in shorter time frames after placement. The effects of insufficient curing are exacerbated at the surface, where moisture can evaporate quicker rather than be retained as is in the center of the concrete.

Air permeability testing is less common in quality control 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. Figure 2.2 below from ACI 201.2 (2008) illustrates transport phenomena in concrete.

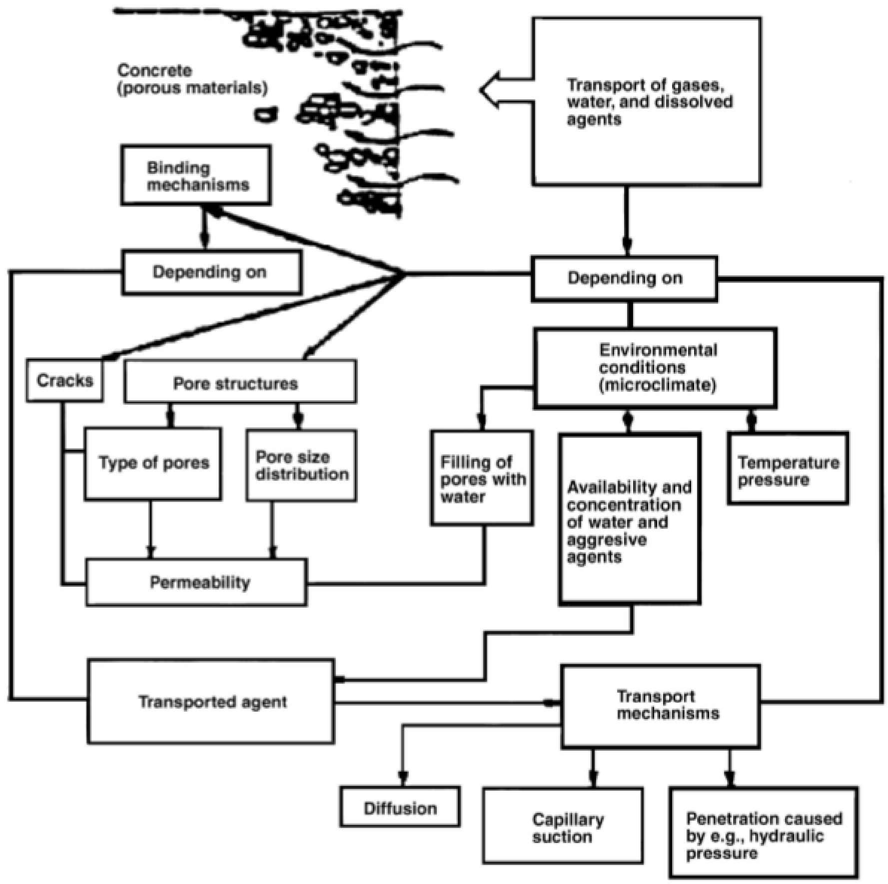

Figure 2.2 Transport phenomena in concrete (from ACI 201.2R-4, originally Schiessl 1992)

2.3.2 Electrical Tests

The implementation of electrical testing for durability properties of concrete allows for a non-destructive alternative to traditional testing. As much of America's infrastructure continues to age and reach the end of its intended useful life, the use of these non-destructive methods allow for earlier detection of deterioration and failures. Using electrical testing methods for evaluation of large concrete structures has been proven useful for identifying several important properties of concrete according to numerous studies (Karhunen et al. 2010).

2.3.2.1 Rapid Chloride Permeability Test (RCPT)

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 (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). These tests are more commonly known as the 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). Cook (1951), as well as Ruettgers and Vidal (1935) 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 2.2 illustrates the RCPT results (in coulombs) associated with different levels of chloride ion permeability and mixture characteristics (ASTM 2018).

Table 2.2 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

2.3.2.2 Surface Resistivity

Until recently, the RCPT was considered the most rapid way to determine concrete’s tendency to allow chloride ion permeability. Surface resistivity measurements are a vast improvement, requiring minutes to obtain results, minutes compared to days for

RCPT (AASHTO 2015). Methods of measuring electrical resistivity of concrete include the disc method, the Wenner method, 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).

One test in particular, the surface resistivity test, has become more common due to ease of the test method, low cost of performance, and the immediate production of results. Surface resistivity is a method of measuring electrical resistivity of water-saturated concrete, and can be used to evaluate a wide array of concrete characteristics (Morris et al. 1996). 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 2011). 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 2.3, presented below, provides chloride ion penetration levels based on surface resistivity results.

Table 2.3 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 (2011, 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 2011).

2.3.2.3 Bulk Resistivity

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.

Bulk conductivity is the inverse of resistivity. Given this information, there should be 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. as discussed in Section 2.3.2.1. 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).

2.3.2.4 Sorptivity

Sorptivity is defined as the action of absorbing and transmitting water by means of capillary force in a porous material (Hall 1989). The sorptivity of concrete is one of the biggest threats to concrete durability, due to the harmful liquids that can be present on the concrete's surface being pulled into the pore structure (Desouza et al. 1998). Desouza also accredited deterioration mechanisms to the sorption of harmful liquids from the surface causing both physical and chemical changes to the concrete. Sorptivity can be measured using ASTM C1585 (ASTM 2013).

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, curing quality, age, microcracking in the concrete, surface treatments, moisture conditions, and quality and methods of concrete placement (ASTM 2013). Durability is heavily influenced by sorptivity, as sorptivity and permeability are the two methods in which most deleterious substances enter concrete (Soutsos 2010). Lowering of sorptivity can be achieved through a variety of considerations when designing the concrete mixture. Inclusion of SCMs in concrete, lowering of w/cm, and designs which consider tighter particle packing will result in better sorptivity characteristics, and concurrently lower permeability, as the two are directly related (Hooten et al. 1993).

2.3.2.5 Tests to Support 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 2014, Weiss et al. 2013, 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 2017).

The formation factor can alternatively be calculated by taking the inverse of the product of porosity and pore solution resistivity (Weiss et al. 2018). 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. The prescriptive portion of the specification recommends 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 (AASHTO 2017). Table 2.4 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 2.5.

Table 2.4 Prescriptive values for F factor, RCPT, and resistivity

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 2.5 Performance values for saturated F factor

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 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).

2.3.3 Resistivity Specifications Used By Other State Agencies

Other states have begun to explore the benefits of surface resistivity as an electrical resistance test. 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 4 in x 8 in 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 2.6 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). 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 2.7.

Table 2.6 LADOTD chloride ion penetrability and associated surface resistivity values

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 2.7 LADOTD acceptance and payment schedules associated with surface resistivity values

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

2.4 Research Needs

As discussed in this chapter, an extensive amount of research has been performed to identify appropriate tests and performance targets to evaluate the durability performance of concrete. The permeability of concrete and its resistance to ingress of deleterious substances can be evaluated in a variety of ways. The most commonly utilized method is RCPT, which is time consuming and has a variety of other limitations. Recently, electrical resistivity has been shown to provide a strong indication of the potential durability performance of concrete. Since chloride ion permeability and electrical resistivity are affected by a number of factors, including local materials and mixture proportions, appropriate targets must be identified if resistivity testing is included in performance specifications for concrete.

NCDOT desires to move towards performance specifications by implementing a resistivity specification. This research has sought to identify, quantify, and evaluate benefits of surface resistivity testing, as provide a recommended specification for surface resistivity testing in North Carolina. Benefits of a surface resistivity specification for NCDOT concrete include cost savings for construction and QA/QC procedures, as well as extend the useful life of concrete pavements and bridges.

Literature review for this project has identified four major needs for research to successfully move toward establishment of a surface resistivity specification for performance engineered concrete. The first research need is to utilize existing data to identify trends in materials and proportions associated with satisfactory and poor concrete performance. The second research need is to establish performance-related criteria related to surface resistivity testing through laboratory testing of conventional NCDOT pavement and bridge mixture designs. The third research need is to provide additional information on the benefits that fly ash and portland limestone cement (PLC) offer for resistivity measures in concrete. The final research need is to provide NCDOT with a draft surface resistivity specification and target performance values, while also outlining recommendations for tasks required to move towards implementation for QA/QC testing.

CHAPTER 3: METHODOLOGY

3.1 Introduction

Implementing performance specifications to improve concrete durability is not possible without first performing research to support identification of the appropriate methods, specification approaches, and target performance criteria. Although the benefits of durability-based tests and specifications discussed in Chapter 2 have been proven, further investigation into the use of these methods and specification provisions is required for use by NCDOT. The testing program for this work was designed to support development and implementation of a resistivity specification to improve concrete durability. 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. In this chapter, the methodology behind the laboratory and testing program for the investigation will be discussed. Identification of mixture types, mixture proportions, batching of fresh concrete, and testing procedures are presented.

3.2 Development of Concrete Mixture Matrix

The mixture matrix was developed in a manner which allows evaluation of results based on changes to proportioning of materials. Three parameters were identified for evaluation which were used as a basis for the remaining material input calculations – w/cm, cementitious material content, and fly ash replacement rate. Utilizing inputs selected to represent a range of potential values bracketing the mixture parameters typical of concrete mixtures historically accepted by NCDOT for bridge (Class AA) and pavement construction, the remaining calculations were performed to develop mixture designs for each of the twenty-four mixtures.

Three w/cm were selected, representing low, medium, and high values typically seen in these types of mixtures. As shown in Figure 3.1, mixtures are grouped by w/cm ratio with the 7 mixtures with a 0.47 w/cm on the left side of the diagram, the 10 mixtures with a 0.42 w/cm in the middle of the diagram, and seven mixtures with a 0.37 w/cm on the right side of the diagram. Cement contents were also varied within the w/cm groups from left to right, with higher cement content mixtures (typical of structural concrete mixtures) shown in orange and lower cement content mixtures (typical of pavement concrete mixtures) in green. The fly ash replacement rates along the left margin result in changes in the amounts of cement and fly ash, represented in pounds within the colored boxes. Considering variations were made only to w/cm and cement contents, coarse aggregate content did not vary across the 24 mixture designs.

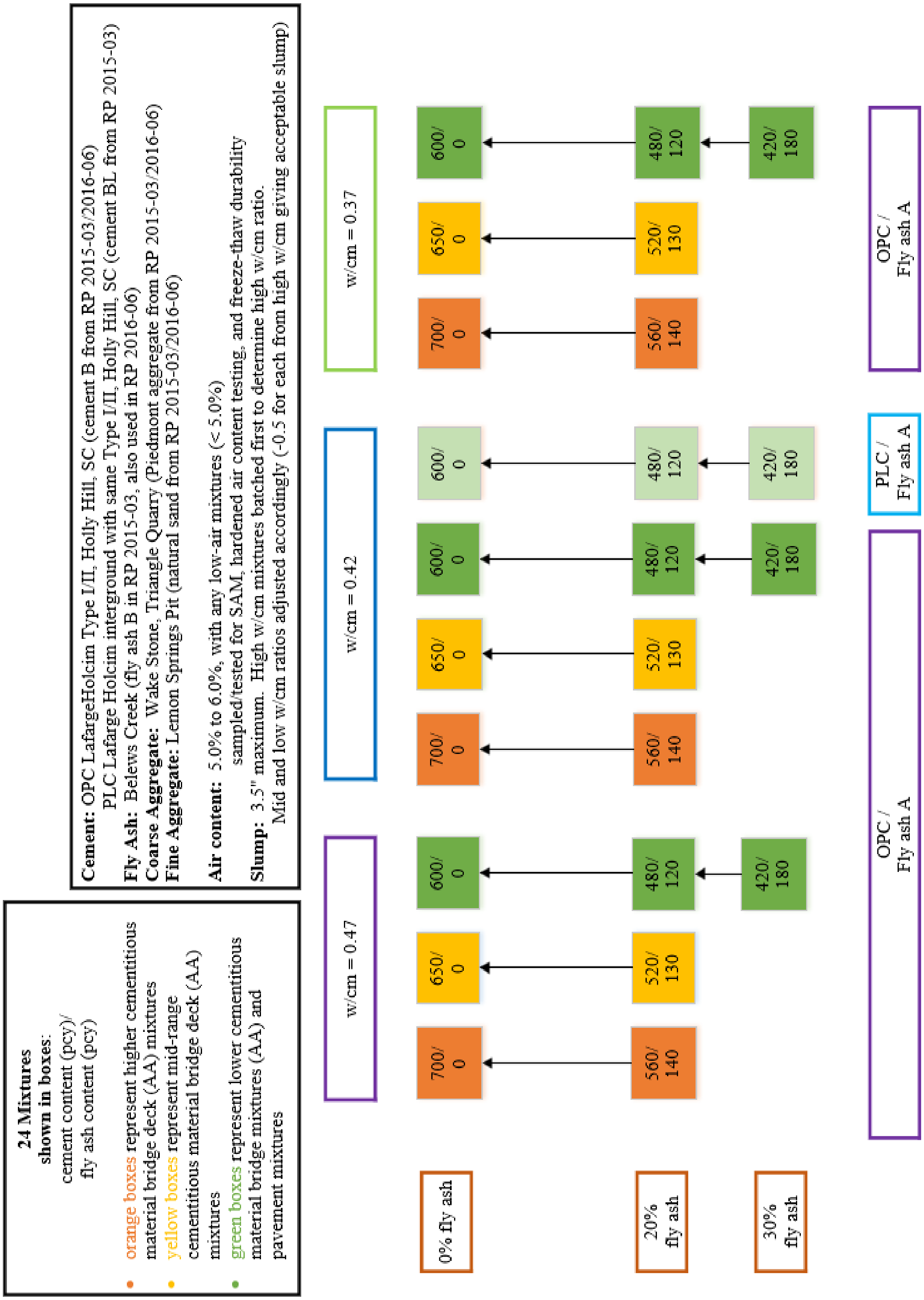

Figure 3.1: Concrete mixture matrix and supporting information

3.2.1 Development of Mixture Design

The NCDOT and research team collaborated to identify 24 target mixtures for testing and batching as part of the laboratory program. All 24 mixtures were designed to meet Class AA bridge deck specifications, with 12 of them having cement contents that could reasonably be considered for paving applications. Materials types and sources including coarse and fine aggregates, fly ash, admixtures, and water were kept consistent across all mixtures. Twenty-one of the mixtures utilized an OPC Type I/II cement sourced from LafargeHolcim in Holly Hill, SC. The remaining 3 mixtures used a Type I/II PLC cement sourced from the same location as mixtures developed and tested as part of previous research for NCDOT RP 2015-03 (Cavalline et al. 2018).

With material type remaining consistent, variation in cementitious material contents, w/cm ratios, and fly ash replacements were used to create mixtures with results at varying replacement levels. Three w/cm ratios were chosen to represent typical, higher than typical, and lower than typical w/cm ratios specified. The high w/cm was chosen to be 0.47, the middle w/cm was chosen to be 0.42, and the low w/cm was chosen to be 0.37. The three different cementitious contents were chosen to represent high, mid-range, and low cementitious contents for Class AA bridge mixtures. A cementitious content of 700 pounds per cubic yard (pcy) was used to represent bridge mixtures with high cement contents. The mid-range cement content mixtures were used to represent typical w/cm specified for bridge deck mixtures, and was 650 pounds pcy. A cement content of 600 pcy was used to represent low cement content bridge deck mixtures as well as paving mixtures.

The remainder of mixture proportions were then computed for each mixture. Utilizing the set inputs established during this process, the mixture proportioning guide outlined in ACI 211.1 was used to calculate the remaining inputs for each mixture design (ACI 2002). Using the ACI 211.1 methodology, the coarse aggregate content was calculated to be 1,659 pcy. Cement contents varied between 420 pcy and 700 pcy depending on w/cm and replacement rate. The amount of fly ash was dependent on cement amount, and varied between 0 and 180 pcy. Fine aggregate amounts ranged from 1,022 pcy and 1,434 pcy. Water contents were calculated based on ACI 211 design procedures, producing water contents ranging between 222 and 329 pcy. Table 3.1 shows mixture characteristics and the resulting mixture proportions.

Table 3.1: Concrete mixture characteristics and 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

3.3 Materials Description and Characterization

The following sections serve as additional information regarding material sources for the concrete produced as part of the laboratory program of this research. Properties related to the materials discussed are also included, and these properties were obtained through experimentation or provided by the material supplier or manufacturer.

3.3.1 Cementitious Material

Three cementitious materials were used for batching of concrete for the laboratory portion of this project: one OPC, one PLC, and one fly ash. Descriptions, characteristics, and sources for each material are provided in the following sections.

3.3.1.1 Portland Cement (OPC)

The OPC selected for the project is one typical of that specified by the NCDOT on paving projects. This OPC was used in 21 of the 24 mixtures produced as part of this research. LafargeHolcim produced the OPC at their manufacturing plant in Holly Hill, SC, and it was shipped to UNC Charlotte. This cement is a Type I/II cement, which meets ASTM C150, "Standard Specification of Portland Cement" (ASTM 2018). The mill reports for the OPC is provided in Appendix A, Figure A.1.

3.3.1.2 Portland Limestone Cement (PLC)

The PLC used in the concrete mixtures batched as part of this research is a type IL cement. This cement was produced at the same manufacturing plant as the OPC in Holly Hill, SC. The PLC was produced according to ASTM C595, "Standard Specification for Blended Hydraulic Cements" using the same clinker as the OPC, as well as less than 15% added limestone (ASTM 2018). The mill reports for OPC are applicable for the PLC, which are provided in Appendix A, Figure A.1

3.3.1.3 Fly Ash

A major focus of this research involves analyzing the effects of fly ash at different replacement rates. Fly ash replacements of 20% and 30% were used to explore the changes in different properties of the fresh and hardened concrete produced, namely surface resistivity. North Carolina Standard Specification section 1024 “Materials for Portland Cement Concrete” currently allows up to 20% replacement by cement mass at a 1:1 replacement ratio (NCDOT 2018). The 30% replacement rate used allows the research team to explore the benefits of increasing this specification to allow a higher fly ash content. Belews Creek Power Plant produced the fly ash at their location in Belews Creek, NC. This fly ash is classified as a Class F fly ash, and information about the ash composition can be found in Appendix A, Figure A.2.

3.3.2 Coarse Aggregate

A coarse aggregate was selected for the batching of concrete mixtures in accordance with NCDOT specification 1014-2, “Aggregate for Portland Cement Concrete – Coarse Aggregate”, as well as ASTM C33, “Standard Specification for Concrete Aggregates” (ASTM 2018, NCDOT 2018). Quarry selection was made by the research team along with NCDOT personnel to represent a coarse aggregate typical of that specified in North Carolina paving mixtures. Another influencing factor in quarry selection was to allow continuity with data obtained from mixtures from previous research projects. Use of the same coarse aggregate sources would allow previously obtained data to be included in this analysis. Aggregate (No. 67) was sourced from the Wake Stone – Triangle Quarry in Cary, North Carolina. Members of the research team transported two cubic yard capacity aggregate hoppers to the quarry, where quarry personnel assisted in loading of aggregate via backhoe and shovel. Material that was collected was then transported back to UNC Charlotte for storage in the same aggregate hoppers until use. Aggregate properties of the granitic gneiss aggregate include a specific gravity of 2.63 and an absorption of 0.40%, with additional information provided in Appendix A, Figures A.3 – A.5 and Table A.1.

3.3.3 Fine Aggregate

The fine aggregate source used for the batching of concrete for this project was selected in a manner similar to the coarse aggregate, using NCDOT specification 1014-1, "Aggregate for Portland Cement Concrete – Fine Aggregate" and ASTM C33. In order to reduce the effect aggregates had on the concrete properties, only one fine aggregate source was used. This fine aggregate is one typical of that currently specified by the NCDOT for paving projects. It is also a fine aggregate that has been used for previous research projects to allow previously collected data to be included in this analysis. Aggregate was obtained by the research team at a natural sand pit quarry in Lemon Springs, North Carolina. Properties of the fine aggregate included a specific gravity of 2.61, an absorption of 0.40%, and a fineness modulus of 2.65. Additional information can be found in Appendix A, Figure A.6 and Table A.2.

3.3.4 Chemical Admixtures

Two admixtures available commercially were used as part of this study, including an AEA and a mid-to-high-range WRA. These admixtures were used to allow the research team to achieve the project requirements of producing concrete with a maximum slump of 3.5 in. and a fresh air content between 5% and 6%. Although NCDOT specification 1000-3(C), "Portland Cement Concrete for Pavement – Slump" requires a maximum slump of 3 in. for hand placed concrete, reasonable variations to the slump were accepted for non-paving mixtures in order to meet the target w/cm (NCDOT 2018).

These variations were deemed acceptable to allow production of concrete with varying aggregate, cementitious material, and water contents.

Although the NCDOT specification 1000-3(B), “Portland Cement Concrete for Pavement – Air Content” specifies an allowable air content of 5% ± 1.5%, a 1% range was used for this project (NCDOT 2018). This tighter range of acceptable air contents better allows the research team to attribute changes in test results to material proportions rather than changes in the air contents of concretes produced.

MasterAir AE 200, a product manufactured by BASF was selected as the AEA for this project. This AEA was used in all 24 mixtures produced in the laboratory batching program. BASF recommends a dosage of between 0.125 and 1.5 fluid ounces per hundredweight (cwt) of cementitious material for this product. The actual range of AEA dosages was 0.42 – 2.99 fluid oz/cwt of cementitious material to produce the required 5.0% - 6.0% air contents.

MasterPolyheed 997’s mid-range WRA, also a product of BASF, was used to achieve workability in 18 of the concrete mixtures for this project. Mixtures that did not require the use of the WRA were provided sufficient workability by means of the mixture design characteristics, namely the w/cm. BASF’s recommendations for dosage of this product ranged between 5 and 15 fluid ounces/cwt of cementitious material for most mixes. The range of WRA dosages for certain mixtures fell outside this range, but mixtures were retained for continuity of project requirements such as consistency in w/cm, fly ash replacement levels, and cementitious contents. The relatively high dosage levels required for some of the mixtures could potentially be attributed to the cementitious materials contents and w/cm ratios included (by design) in the mixture matrix.

3.4 Testing Program

The testing program developed for this project can found in Table 3.2, presented below. Testing was performed on fresh and hardened concrete in accordance with the AASHTO, ASTM, and experimental test procedures shown in Table 3.2. As discussed in Section 3.2.1, 12 of the 24 mixtures were designed to be representative of paving mixtures. Beam specimens for flexural strength (modulus of rupture (MOR)) testing were only cast from mixtures containing lower cementitious contents typical of pavement mixtures.

Table 3.2 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 (flexural strength)	ASTM C78	28	2
	Modulus of elasticity and Poisson's ratio	ASTM C469	28	2
	Hardened air content	ASTM C457 (automated)	N/A	2
	Resistivity	AASHTO T 358	3, 7, 28, 56, 90	3 each age
	Formation factor (via Bucket Test)	Protocol by J. Weiss	35	2
	Shrinkage	ASTM C157	Per standard	3
	Rapid chloride permeability	ASTM C666 (procedure A)	28, 90	2

3.5 Batching and Mixing Procedure

Concrete batching to support this experimental program was performed using a six cubic feet (cf) portable drum mixer. As detailed in the table above, a large array of tests were performed to evaluate the fresh properties of the concrete as well as the mechanical properties and durability performance of the hardened concrete. Calculation of batch size was completed given the tests required, specimens needed, and estimated waste. This value was determined to be 4.11 cf (for batches requiring modulus of rupture beams), and 2.79 cf for the remaining 12 mixtures. Experience of the research team led to the decision to break the paving mixtures into two batches. These batches were produced in two 2.65 cf portions, and the non-paving mixtures were batched in 3.0 cf batches to allow for waste. Compressive strength cylinders were prepared from each batch and tested to ensure consistency between batches of the same mixture.

Batching of concrete was performed in accordance with ASTM C685, “Standard Specification for Concrete Made by Volumetric Batching and Continuous Mixing” (ASTM 2017). For non-paving mixtures, all test specimens listed in Table 3.2 excluding modulus of rupture beams were produced from one batch. Paving mixtures were divided into cylinder mixtures and beam mixtures. Cylinder mixtures included the amount of concrete needed for fresh property tests, as well as hardened air content specimens (2) and cylinders for compressive strength (fifteen 4in x 8in cylinders), modulus of elasticity (MOE) (two 6in x 12in cylinders), RCPT (two 4in x 8in cylinders), and formation factor (two 4in x 8in cylinders). Beam mixtures were used to complete fresh property testing, hardened air content specimens (2), and beams for MOR (3) and shrinkage testing (3).

3.6 Testing of Fresh Concrete Properties

Several key properties were tested following batching of each mixture, while the concrete was still in a fresh state. These properties included the slump, fresh air content, temperature, and fresh density. The SAM was also used for a number of mixtures to determine fresh air content and SAM number, which relates to the air matrix in fresh concrete (Ley 2014). Procedures and standards used for obtaining fresh results are described in the following sections.

3.6.1 Slump

Slump testing was performed for each mixture in accordance with ASTM C143, “Standard Test Method for Slump of Hydraulic-Cement Concrete” (ASTM 2015). The purpose of performing slump tests as a part of the testing program was to evaluate conformance to the parameters set forth in the acceptance criteria of this project. The target slump for this project was 3.5 in. Evaluating changes to mechanical and durability tests associated with design variances was the main focus of the laboratory portion of this project, therefore it was imperative to conform to the designs previously presented. Deviation from the target project slump was accepted within reason, and can be attributed to the characteristics of each mixture design. The slump test was also used during mixing as a simple quality control tool, as the slump of the concretes produced with the high (0.47) w/cm should be much higher than those with the low (0.37) w/cm. Workability of the mixtures with the low w/cm and cementitious contents were an issue due to the associated low slumps. In order to produce mixtures according to design that could be cast into samples, mixtures that were anticipated to have little to no slump were dosed with a WRA.

3.6.2 Air Content

Air content is one of the most important factors in ensuring concrete is durable, as the entrained air matrix is the mechanism in which the concrete mitigates the risk of damage from freeze-thaw conditions. Measurement of the fresh air content was performed in conformance with ASTM C231, “Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method” (ASTM 2017). A range of 5.0% - 6.0% was deemed acceptable for concrete produced for the laboratory portion of this project. Although a range of 5.0% - 8.0% is typically sufficient for entrained air (Neville 2011), the tighter range required for this project was chosen for two reasons. As stated in the previous section, the evaluation test results as a result of material proportions and types was the main focus of this research. Consistent air contents ensured that the proportion of air voids did not vary greatly, which would have had an impact on many of the focus tests of this project. The acceptable air content for this project was also the same used in previous projects completed by the research team. As a result of this continuity, this data can be evaluated and correlated in conjunction with data collected from previous research. To ensure mixtures would have an acceptable entrained air content, an AEA was used. The SAM was also used to test for fresh air content and SAM number for a number of mixtures. Procedures for performing the SAM test are outlined in AASHTO TP 118 (AASHTO 2017).

3.6.3 Unit Weight

The fresh unit weight was the first data collected for each mixture. Unit weight serves as an early indication of whether air content may be too high or low. Testing was performed in accordance with ASTM C138, "Standard Test Method for Density (Unit Weight), Yield, and Air Content (Gravimetric) of Concrete" (ASTM 2017). The same equipment used for pressure method testing of air content was used for determining the unit weight, as the lower portion of the air meter was an apparatus of known weight and volume. Unit weight also served as a way of ensuring proper material proportioning was used in each mixture, as aggregate, cement, and water weights were all varied, producing different densities.

3.7 Preparation and Curing of Test Specimens

Preparation of test specimens was performed in accordance with ASTM C192, "Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory" (ASTM 2018), as well as the associated ASTM and AASHTO standards for each test.

Form release was applied prior to casing of specimens to ease the demolding process of the hardened concrete samples. Numerous people assisted in the batching and sample preparation process, so continuity of sample type made by each individual was ensured where possible. Upon demolding of samples, they were placed in a moist curing room meeting ASTM C511, “Standard Specification for Mixing Rooms, Moist Cabinets, Moist Rooms, and Water Storage Tanks Used in the Testing of Hydraulic Cements and Concretes” where a continuous mist was applied until the maturity specified for testing of each sample was reached (ASTM 2013).

3.8 Testing of Hardened Concrete

Hardened properties of concrete tested as part of this research can be found in lower portion of Table 3.2 (Section 3.4). Mechanical property tests performed include compressive strength, MOR, MOE and Poisson’s ratio, and shrinkage. Properties related to durability that were tested in the hardened state included surface resistivity, bulk resistivity, rapid chloride permeability, and the Bucket Test.

3.8.1 Mechanical Properties

Mechanical properties are historically the ones used for specification and acceptance, and therefore are particularly important. As previously mentioned, tests to determine the mechanical properties were performed for all mixtures produced.

3.8.1.1 Compressive Strength

Compressive strength testing was performed in accordance with ASTM C39, “Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens” (ASTM 2018). The cylinders used for testing were 4in x 8in, and testing was performed at 3, 7, 28, 56, and 90 days after the mixing date. A minimum of 4,500 psi average compressive strength is required in section 1000-3 of the NCDOT 2018 Roadway Standard Specifications (NCDOT 2018).

3.8.1.2 Modulus of Rupture (MOR)

Testing to determine the MOR for paving mixtures was performed according to ASTM C78, "Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)" (ASTM 2018). Specimens were cured for 28 days after the mixing date before testing. The MOR is used for evaluating the tensile strength of the concrete, as well as for conformance to NCDOT specifications. NCDOT Roadway Standard Specifications Section 1000-3 requires a minimum MOR average of 650 psi at 28 days for concretes used for paving purposes (NCDOT 2018). As previously stated, MOR prisms were cast for 12 of the mixture designs.

3.8.1.3 Modulus of Elasticity (MOE) and Poisson's Ratio

The MOE and Poisson's Ratio were found using the procedures outlined in ASTM C469, "Standard Test Method for Static MOE and Poisson's Ratio of Concrete Compression" (ASTM 2014). MOE testing was performed on day 28 following the mixing date, and two specimens were tested. A minimum of two team members were present for MOE testing to ensure proper loading of specimens and determination and recording of values.

3.8.1.4 Shrinkage

Shrinkage properties were determined using an unrestrained shrinkage testing method. Testing was performed according to the procedures outlined in ASTM C157 (ASTM 2017). Three specimens that were 4in x 4in x 11in were cast for shrinkage testing. Gauge studs were inserted into the molds used for casting so that when disassembled the studs would be embedded into the concrete. Following demolding of the specimens, they were wet cured for 28 days, at which point they were transferred to an environmental chamber. The chamber is a controlled environment with a temperature of 73 degrees Fahrenheit (ºF) (with a 3º tolerance), and a relative humidity of 50% (with a 4% tolerance). ASTM C157 correlates the results found from this unrestrained shrinkage testing to potential volumetric contraction related to factors other than outside forces and temperature changes. Although casting of specimens and testing was performed as part of the laboratory testing program, shrinkage results will be presented in the project report and other publications.

3.8.2 Durability Properties

Properties relating to good durability are necessary for concrete pavements and structures to have a long service life. Testing for durability was done by evaluating the surface resistivity, bulk resistivity, rapid chloride penetration, and formation factor.

3.8.2.1 Surface Resistivity

AASHTO T 358 was used to evaluate surface resistivity properties (AASHTO 2017). Three 4 in x 8 in cylinders were tested at days 3, 7, 28, 56, and 90 following mixing. These cylinders were also used to perform the compressive testing at the same ages. A Resipod surface resistivity meter manufactured by Proceq was used for testing as part of this research (Proceq 2011). As previously stated, surface resistivity testing serves as a non-destructive method of testing concrete permeability, and results can be correlated to other permeability tests, namely RCPT (Rupnow and Icenogle 2011).

3.8.2.2 Bulk Resistivity

Testing for bulk electrical resistivity is a means to obtain a rapid indication of concrete's ability to resist chloride ion penetration. The method of testing is similar to that of the surface resistivity test and uses the same apparatus, in this case the Proceq Resipod (Proceq 2011). The results of surface resistivity and bulk resistivity are directly related. ASTM C1760 was used for resistivity analysis (ASTM 2012). Bulk resistivity testing was performed on days 28 and 90 following the mixing date. Three cylinders were tested at each date, and were the same cylinders used for surface resistivity and compressive strength testing. Plate electrodes were used to pass 60 volts through the saturated cylinders via plate electrodes resting upon saturated sponges. Following the testing of the concrete samples, the conductivity of the sponges was tested and utilized to adjust the calculated bulk resisitivity of the cylinder specimen.

3.8.2.3 Chloride Permeability

Evaluation of chloride permeability is the third electrical test performed as part of the testing program for this project. RCPT was completed according to the procedures outlined in ASTM C1202 (ASTM 2018). Two 4 in x 8 in cylinders were cast for use in this test. Testing was performed at ages 28 and 90 following the mixing date. One cylinder was used to create two RCPT samples for each testing date, as the thickness of the samples must be approximately 2 in.

Prior to testing, the samples were saw cut and conditioned. Conditioning consisted of allowing the cylinders to air dry for at least an hour after saw cutting. During this drying period, water was boiled and allowed to cool for a later portion of the conditioning procedure. Following the drying period, the samples were placed in a vacuum desiccator for three hours. A second attached desiccator was filled with the cooled water previously mentioned at the end of the three hour vacuum, and the water was transferred into the desiccator containing the specimens. The vacuum was continued for another hour, at which point the pump was turned off and the vacuum released. The samples were left to soak in the water filled desiccator for approximately 18 hours (with a 2 hour tolerance).

At the conclusion of the conditioning procedures, the specimens were placed into the testing cells. The cells position the specimens so that the two round faces are exposed to solutions, one containing a sodium chloride solution, and the other containing a sodium hydroxide solution. Rubber gaskets ensured that the solution was not able to escape the cells and provide a good seal. The cells were then connected via lead wires with banana clip ends to the monitoring device. 60 volts were applied to the cells for a period of 6 hours, at which the test was concluded and results recorded. The cells were periodically monitored to ensure the voltage and solution levels in the cells did not fluctuate. The result was reported in Coulombs, which has been correlated to the concrete's resistance chloride ion penetration (Whiting 1981). Lower results indicate better resistance to penetration, and higher coulomb readings are indicative of the concrete being more susceptible to chloride ion penetration.

3.8.2.4 Bucket Test

The Bucket Test is an experimental method for evaluating the formation factor of concrete. The formation factor is beneficial for evaluating characteristics of the pore system of hardened concrete (Weiss 2014, AASHTO 2017). This method involves performing resistivity testing on concrete cylinders conditioned using a standardized pore solution. This method is believed to be advantageous to other testing methods due to pore solution resistivity having an effect on results obtained in other electrical resistivity tests (Weiss et al. 2018). This procedure was developed by Oregon State University, and involves submerging two 4in x 8in concrete cylinders in a 5 gallon bucket containing a solution that mimics the pore solution of concrete. The solution consists of 13.2535 kilograms (kg) water, 102.60 grams (g) sodium hydroxide (NaOH), 143.90 g potassium hydroxide (KOH), and 27.0 g calcium hydroxide [$Ca(OH)_2$]. At set intervals ranging from two hours to 91 days, the cylinders are removed from the solution, towel dried, and the mass and resistivity of the concrete are measured.

CHAPTER 4: TEST RESULTS

This chapter serves as a summary of data collected through the testing discussed in Chapter 3 of this thesis. As previously stated, mixture designations were coded in order to easily identify differentiations between key mix characteristics. The first part of the mixture designation is a letter indicative of the w/cm: "H" for a 0.47 w/cm, "M" for a 0.42 w/cm, and "L" for a 0.37 w/cm. The second portion of the mixture designation relays the cement content of the mixture in pcy, ranging between 420 and 700. The final portion of the mixture designation is the fly ash content in pcy, ranging from 0 to 180.

4.1 Testing of Fresh Concrete

This section of this thesis provides the results obtained from the fresh concrete property tests discussed in Section 3.6. Tests for fresh properties included slump, fresh air content, unit weight, and SAM number. These tests were performed on every mixture to ensure the acceptance criteria outlined for this project was met. The target air content for each mixture was between 5.0% - 6.0%, which was held to a strict requirement to ensure consistency with other mixtures from this project, as well as previous projects. Table 4.1 below provides a summary of fresh test results for each mixture, which will be discussed in further detail in the subsequent sections. Dosage amounts of both the AEA and WRA ranged due to mixture characteristics and environmental influences, and were allowed to be able to meet the acceptance criteria.

The target slump for this project was set at 3.5 in. A number of batches had slumps deviating from 3.5 in. Mixtures with slumps less than 3.5 in were dosed with a WRA to increase the slump. Mixtures with slumps greater than 3.5 in were still utilized since a major focal point of the laboratory portion of this project was the influence of w/cm on hardened concrete performance, and w/cm were maintained.

Table 4.1 Fresh concrete test results

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	4.3	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	2.3	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

4.1.1 Slump

The results of the slump test for each mixture are shown in Table 4.1. As previously mentioned, the different characteristics of each mixture required different dosages of WRA to achieve the desired slump. Mixtures with the low w/cm (0.37) and those with the low cementitious material contents (600 pcy) required the greatest amount of WRA, and as can be observed in Table 4.1, many were significantly lower than the target slump. All mixtures retained for testing, however, were judged to have adequate workability for proper specimen consolidation. It was important for mixtures to conform to the target w/cm ratios outlined in the mixture matrix, but also to achieve adequate workability for specimen consolidation. If the consistency of the concrete is too stiff, specimens could produce test results not representative of the mixture's performance ability due to poor placement, compaction, and possible separation of larger aggregates within the mixture (ACI 2008, Kosmatka et al. 2011). Mixtures with the high w/cm (0.47) and higher cementitious contents (650 - 700 pcy) required far less WRA, as workability was provided from components of the mixture design. The WRA dosages for these lower 33% of mixtures were less than 1.0 fluid oz/cwt. Among this lower third of dosages, six mixtures (H-700-0, H-560-140, H-650-0, H-520-130, H-600-0, and M-700-0) required no WRA.

4.1.2 Air Content

The results of the Type B air meter tests for fresh air content are provided in Table 4.1. As could be assumed, the varying characteristics of each mixture resulted in variations to the amounts of AEA needed to meet the target air content of 5.0% to 6.0%. The characteristics of cement and fly ash composition, material temperatures, and WRA dosages have also been shown to influence AEA performance. Of the mixtures with the highest 50% of required AEA dosages, 15 out of 18 of the mixtures contained either fly ash or portland limestone cement. The same mixtures requiring the upper half of dosage rates included 15 of 18 of the highest WRA dosages. The range of AEA dosage for all mixtures was 0.42 – 2.99 fluid oz/cwt. 15 of the 36 mixtures required less than 1.0 fluid oz/cwt, with 12 requiring between 1.0 and 2.0 fluid oz/cwt and 9 requiring more than 2.0 fluid oz/cwt.

4.1.3 Unit Weight

The results of the unit weight tests can be found in Table 4.1. Unit weights ranged between 136.1 pounds per cubic foot (pcf) and 143.9 pcf. This range was expected due to the variations in mixture constituents and proportions. A correlation can be seen between the unit weight and w/cm, as well as total cementitious content. Mixtures with the high w/cm, low cementitious content, and those with a fly ash replacement typically had lower unit weights. Of the 15 mixtures with the lowest unit weights, 14 of them were either mixtures with the high w/cm ratio (0.47) or the low cementitious content (600 pcy). The 5 heaviest unit weights were associated with mixtures containing no fly ash, with all but three of the straight cement mixtures (7 of 10) falling in the upper 50% of the range. These three that were not in the upper 50% also happened to be the high w/cm, which provides explanation for why they differed. A graphical representation of unit weights separated by non-fly ash and fly ash mixtures, and color coded by total cementitious material content can be found in Figure 4.1

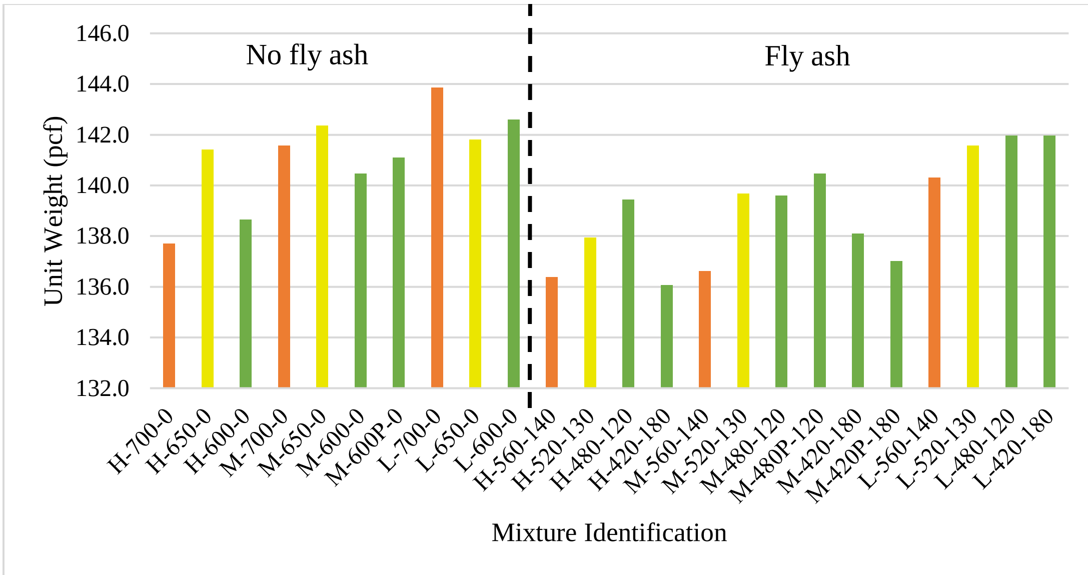

Figure 4.1 Fresh unit weight

4.2 Testing of Hardened Concrete

This portion of the thesis provides results and discussion of the tests performed on hardened concrete specimens. It is divided into two sections, one of which discusses mechanical properties, while the other is for properties related to durability. The test results and discussion of mechanical properties in Section 4.2.1 include compressive strength, MOE, Poisson's ratio, and MOR. The durability properties discussed in Section 4.2.2 include surface resistivity, bulk resistivity, chloride permeability, the Bucket Test, and formation factor.

4.2.1 Mechanical Properties

The subsequent sections provide results and discussion of mechanical property testing, including compressive strength, MOE, Poisson's ratio, and MOR, for which the results of each are provided below in Table 4.2. It should be noted that MOR was not tested for all mixtures, only for those with characteristics typical of a NCDOT paving mixture.

Table 4.2 Mechanical property test results

Designation	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	744.6
H-480-120	2,784	3,150	3,982	4,418	5,148	2,530,000	0.20	808.3
H-420-180	2,446	3,417	4,328	4,869	5,521	2,460,000	0.22	724.4
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	821.8
M-480-120	4,167	4,895	5,390	5,832	6,483	3,350,000	0.19	726.3
M-420-180	3,991	4,260	5,007	5,590	6,216	3,080,000	0.20	726.5
M-600P-0	4,661	5,212	6,284	6,841	7,098	3,450,000	0.23	809.0
M-480P-120	4,249	5,314	6,415	6,967	7,215	3,130,000	0.19	719.9
M-420P-180	3,852	4,288	5,091	5,418	6,004	3,000,000	0.20	680.6
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	816.9
L-480-120	5,510	6,184	6,814	7,107	7,650	3,090,000	0.22	718.1
L-420-180	5,264	5,716	6,228	6,693	7,063	3,240,000	0.20	815.4

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

4.2.1.1 Compressive Strength

Compressive strength testing was performed for each mixture at ages of 3, 7, 28, 56, and 90 days. Three cylinders were tested per mixture at each age, and the results were averaged. 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 this requirement. Hydration of fly ash also occurs at later ages than cement, H-420-180 met the minimum requirement at 56 days, and H-480-120 met the requirement by 90 days. Compressive strength test results can be found in Table 4.2, and are graphically displayed in Figures 4.2 and 4.3.

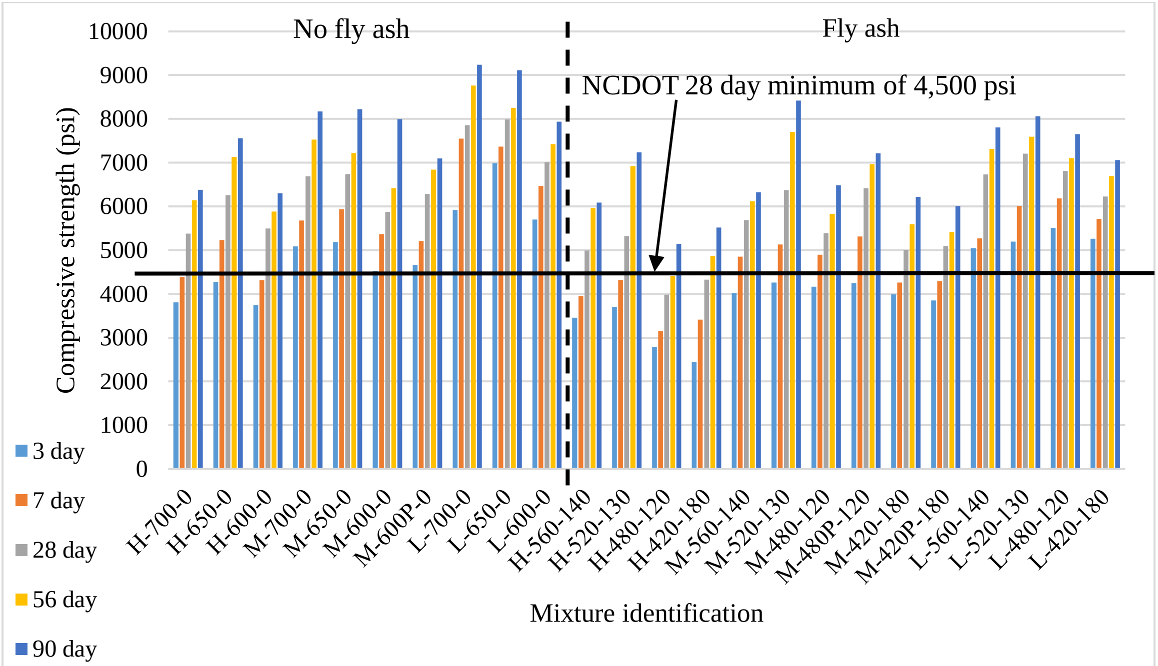

Figure 4.2 Compressive strengths with mixtures sorted by non-fly ash and fly ash mixtures

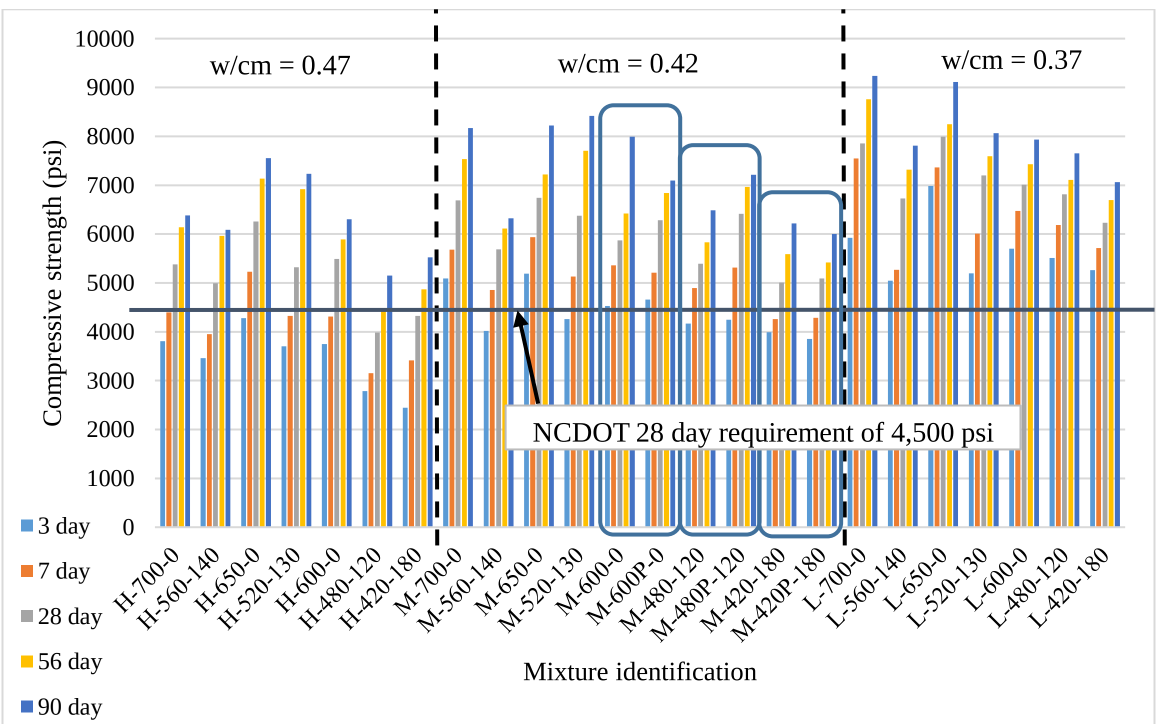

Figure 4.3 Compressive strength test results with mixtures sorted by w/cm

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 increasing water contents in these higher w/cm mixtures result in a reduction in compressive strength. 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 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. This could be indicative of NCDOT mixtures having excessive amounts of cement, which 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.

In regards to total cementitious content, mixtures with the higher cementitious material contents 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. This is a second factor that could account for H-480-120 and H-420-180 not meeting the 28 day minimum. 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 performed the best in the compressive strength test. Of the ten groups of mixtures (as grouped by cementitious material content and w/cm), all but one (M-480P-120) mixture showed straight cement mixtures outperforming their companions with a fly ash replacement. These nine non-fly ash mixtures had superior compressive strengths across all five test dates. As previously mentioned, fly ash hydrates more slowly than portland cement. This could mean that had compressive strength testing been performed at even later ages, these compressive strengths could have increased to be 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. 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 were higher than their companions. 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. However, additional testing would be necessary to confirm the cause of the increased compressive strength obtained by most PLC mixtures.

4.2.1.2 Modulus of Rupture

Modulus of Rupture testing was performed at 28 days for pavement mixtures only, with three beams being tested for each and the results averaged. MOR testing results can be found above in Table 4.2. Mixtures for which MOR testing was performed are colored green in Figure 3.1. A graphical depiction of MOR results can be found in Figures 4.4 and 4.5.

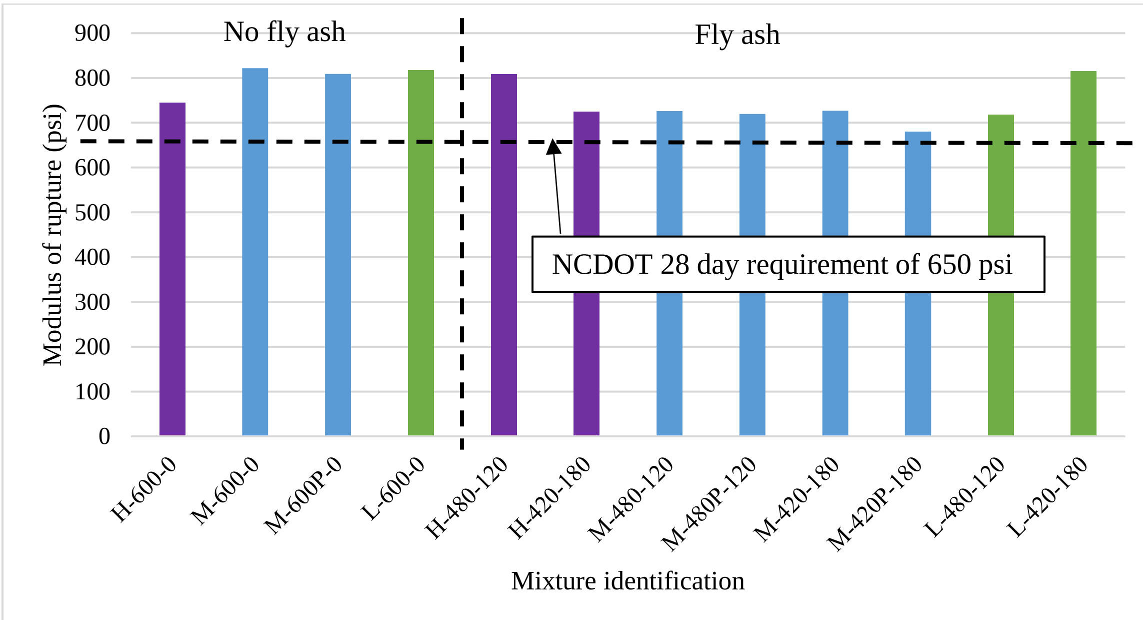

Figure 4.4 MOR results with mixtures sorted by non-fly ash and fly ash mixtures

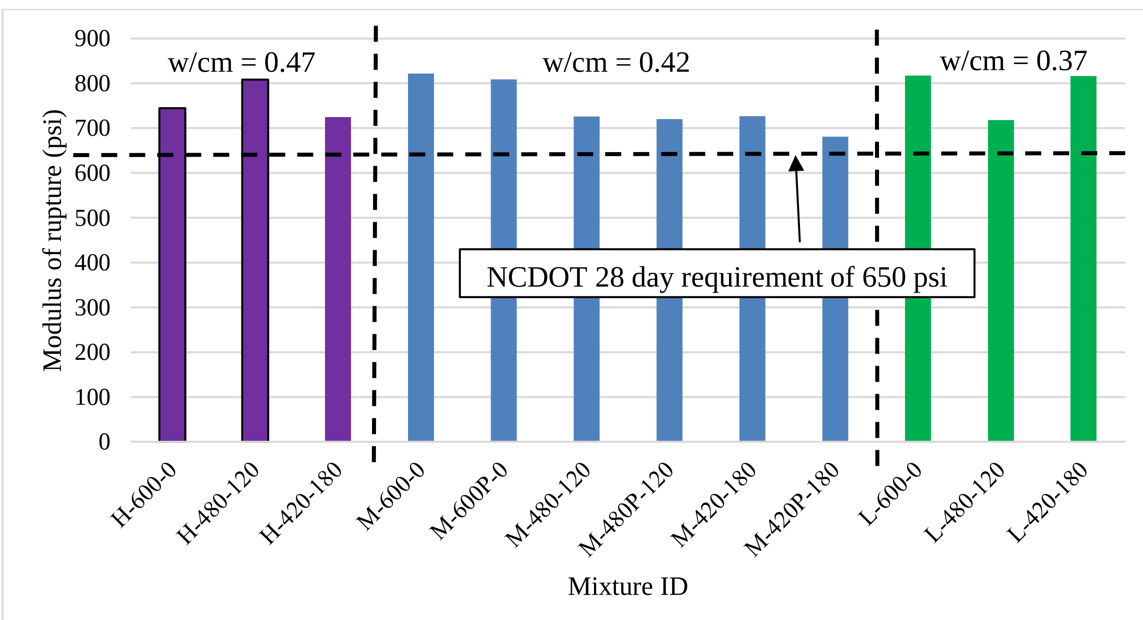

Figure 4.5 MOR results sorted by w/cm

NCDOT's 2018 Standard 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 performed the best. The four mixtures with no 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.

4.2.1.3 Modulus of Elasticity and Poisson's Ratio

A graphical representation of the calculated and measured MOE's can be found in Figure 4.8. Non-fly ash mixtures showed better MOE performance than their fly ash companion mixtures, which is visible in Figure 4.6, color coded by total cementitious material, and separated by fly ash and non-fly ash mixtures. Boxes are also used in Figure 4.6 and subsequent graphs to group OPC and PLC companion mixtures. The MOE values for lower w/cm mixtures performed better than the higher w/cm mixtures. Mixtures with higher cementitious material contents also typically performed better 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.

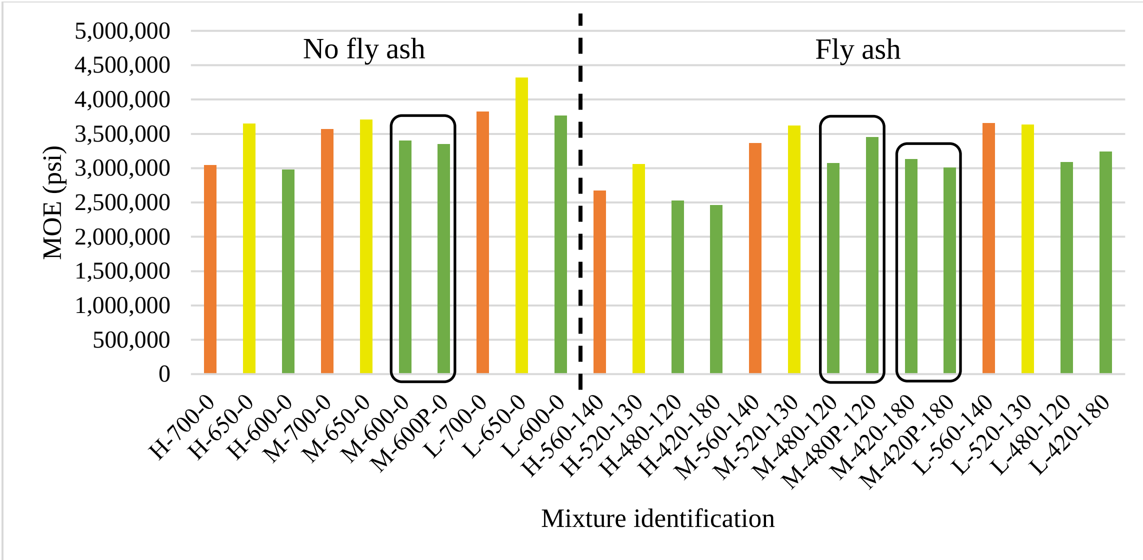

Figure 4.6 MOE results sorted by non-fly ash and fly ash mixtures

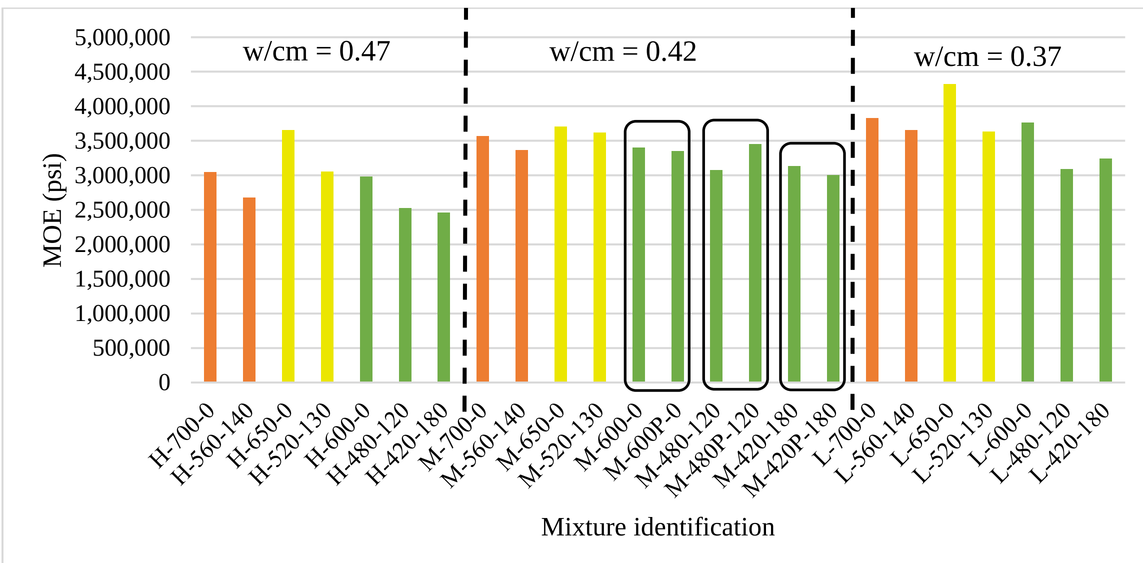

Figure 4.7 MOE results sorted by w/cm

Test results for MOE and Poisson's Ratio testing can be found in Table 4.2. 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 Equation 5.1 from 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 than the predicted counterparts, roughly 13% - 33% lower. This trend of lower-than-predicted MOE values measured from concrete with North Carolina 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.

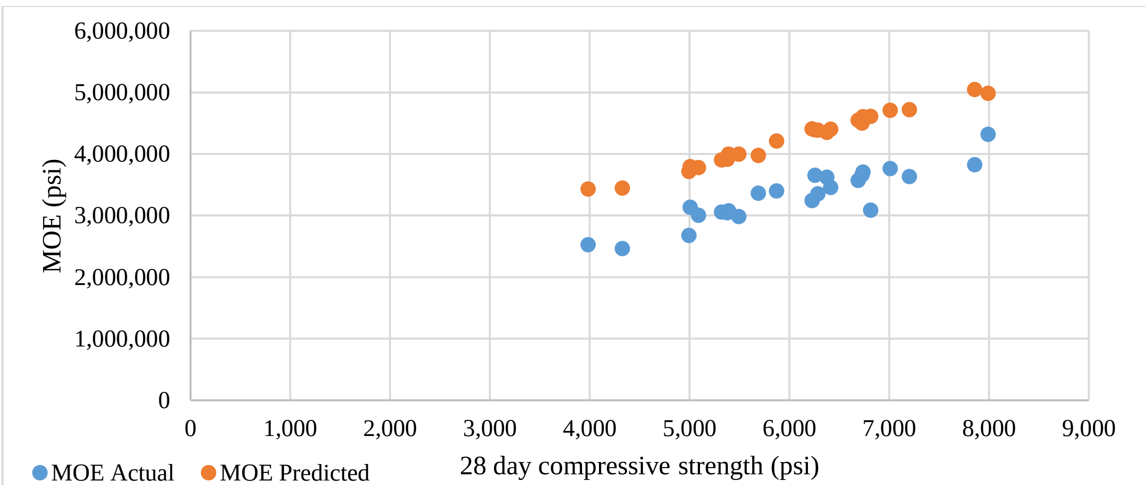

Figure 4.8 Predicted and measured MOE results

No trend was evident when comparing OPC and PLC companion mixtures. These mixtures are represented in Figure 4.9.

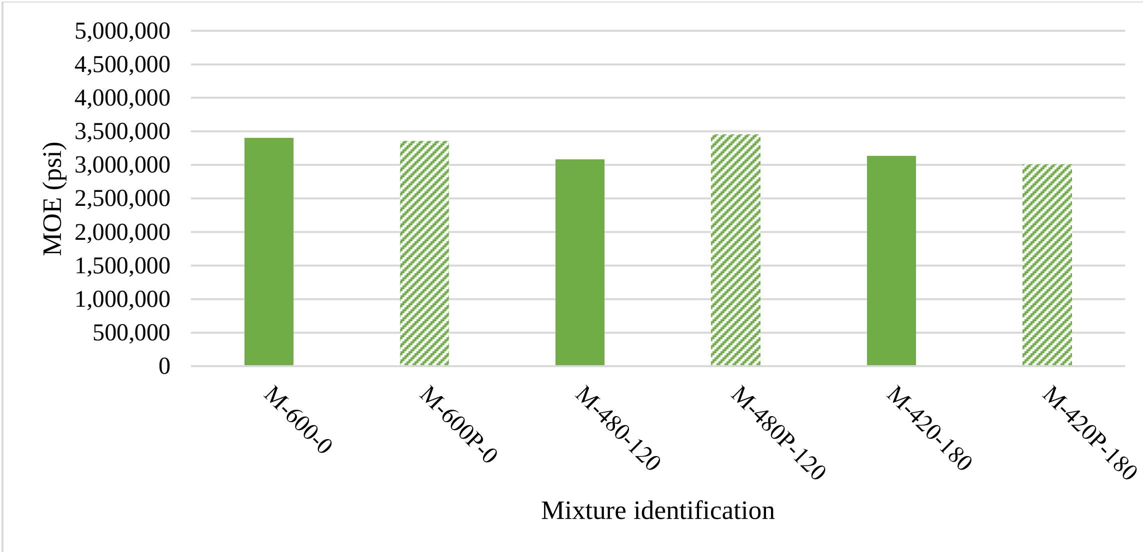

Figure 4.9 MOE results for OPC and PLC companion mixtures

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 graphed. Figures 4.10 and 4.11 show this data color coded by total cementitious material content and separated by fly ash vs. non-fly ash mixtures and w/cm.

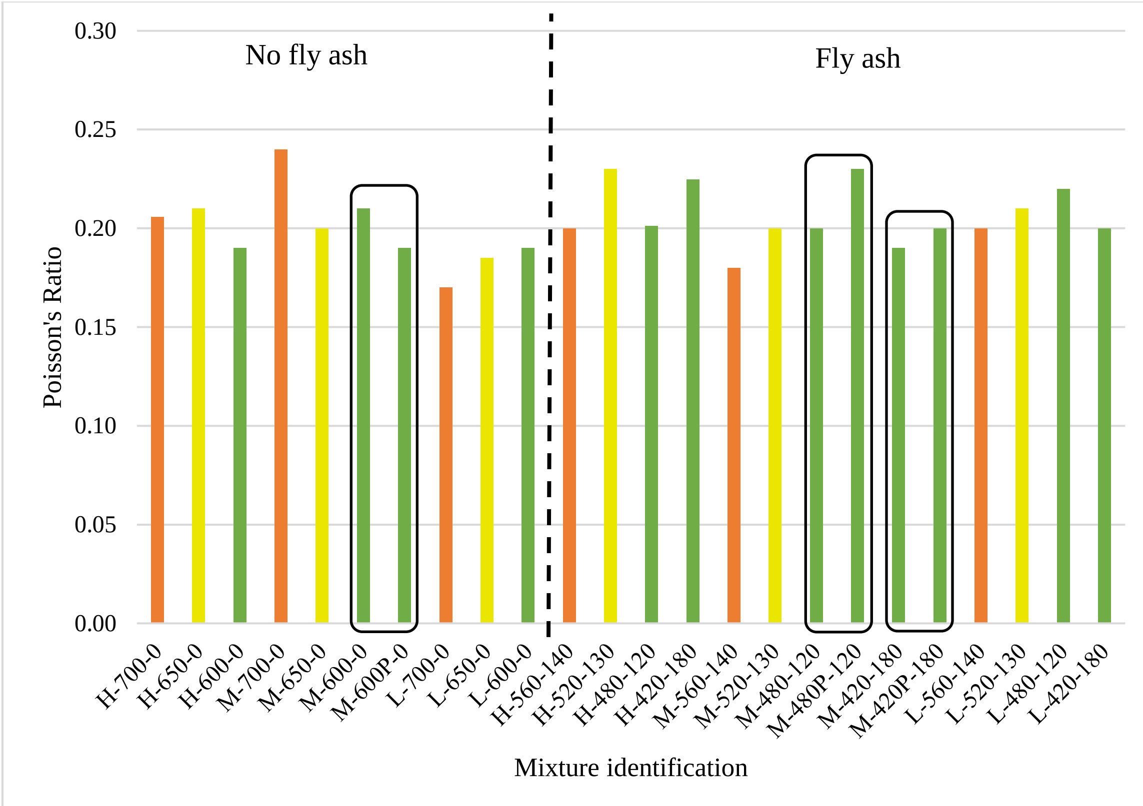

Figure 4.10 Poisson’s Ratio sorted by fly ash and non-fly ash mixtures

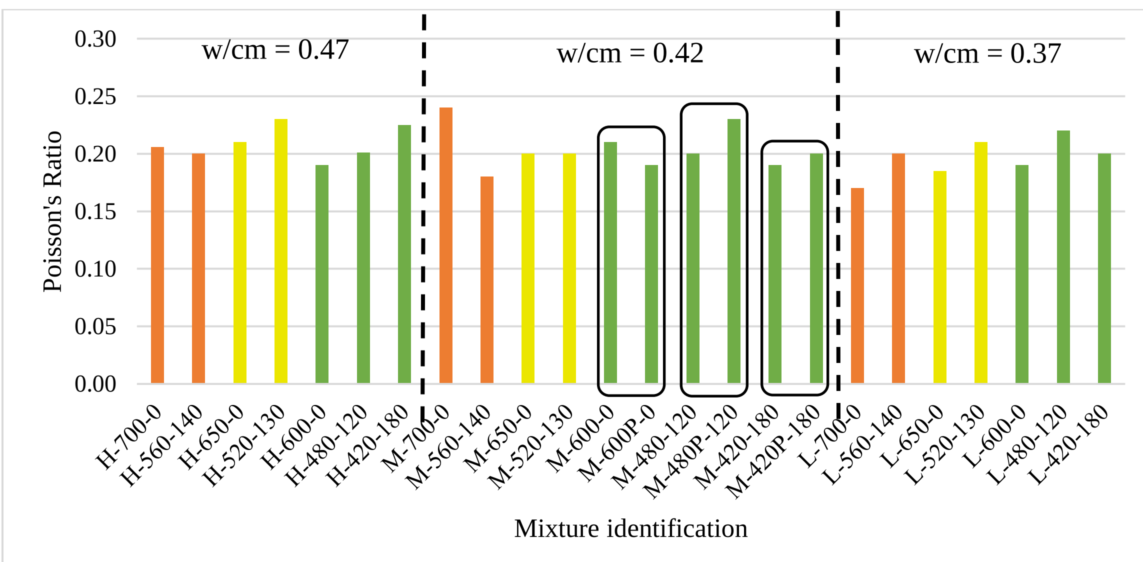

Figure 4.11 Poisson’s Ratio sorted by w/cm

4.2.2 Durability Performance

The purpose of the following sections is to provide test results and discussion of several durability performance tests. These properties include surface resistivity, bulk resistivity, chloride permeability, and formation factor. The durability test results for each mixture are shown in Table 4.3.

Table 4.3 Durability performance

Designation	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

4.2.2.1 Surface Resistivity

Surface resistivity testing was the primary focus of this research, as North Carolina does not currently specify the test for QA or QC, but desires to do so in the future. Testing was performed at ages 3, 7, 28, 56, and 90 days on three cylinders, with the average of the three reported for each mixture in Table 4.3. These results are shown graphically in Figure 4.12 sorted by fly ash and non-fly ash mixtures, and Figure 4.13 sorted by w/cm, with AASHTO T 358 performance ranges listed in Table 4.4. It should be noted that higher resistivity results are indicative of lower permeability (AASHTO 2017).

Table 4.4 AASHTO T 358 performance targets

Chloride ion penetration	Resistivity (kΩ-cm)
High	< 12
Moderate	12 – 21
Low	21 – 37
Very low	37 – 254
Negligible	> 254

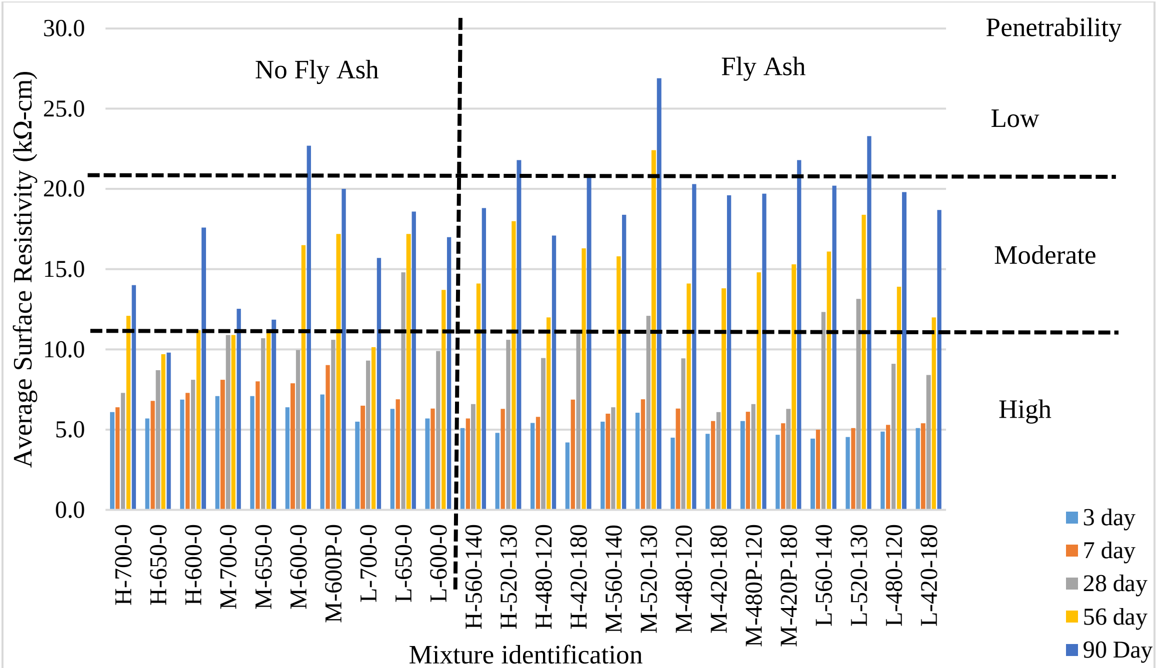

Figure 4.12 Surface resistivity averages with mixtures sorted by non-fly ash and fly ash mixtures

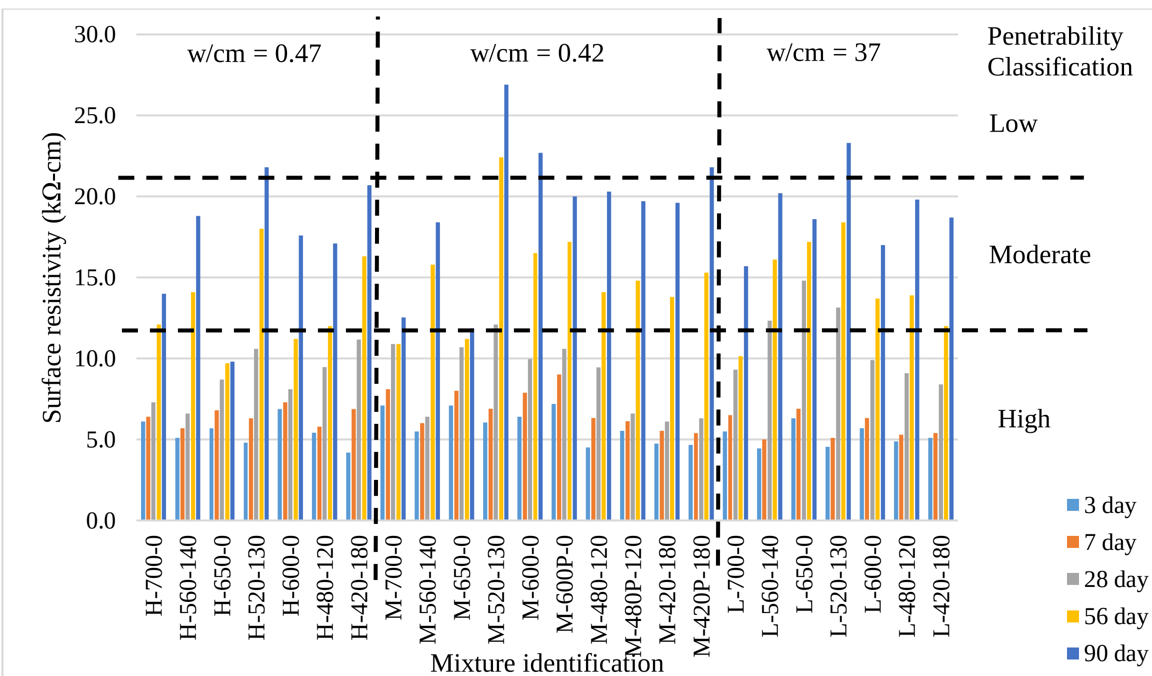

Figure 4.13 Surface resistivity averages with mixtures sorted by w/cm

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 4.14, which is color coded by total cementitoius content in a manner similar to Figure 3.1. 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 4.14 – 4.16 (w/cm vs. surface resistivity graphs) in sequence, the separation of the values can be clearly observed.

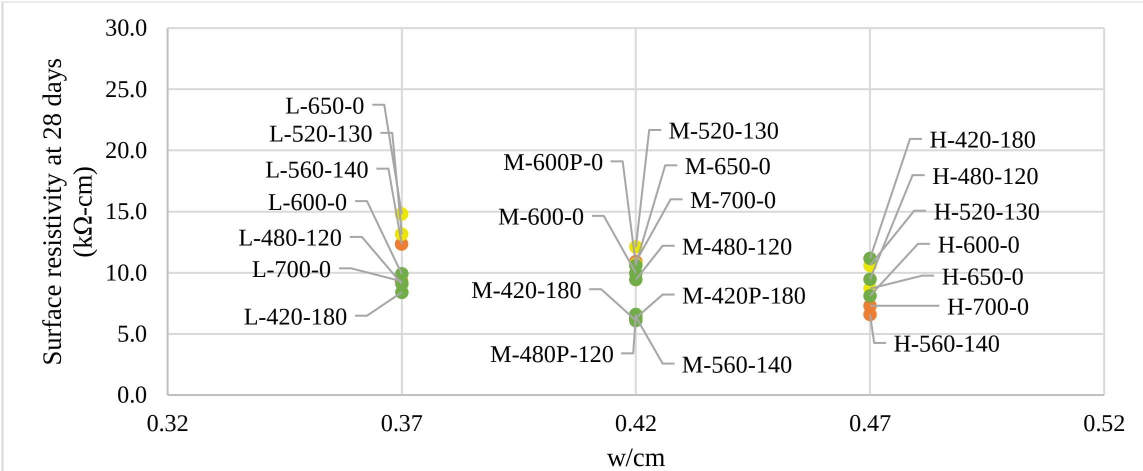

Figure 4.14 w/cm vs. surface resistivity averages at 28 days

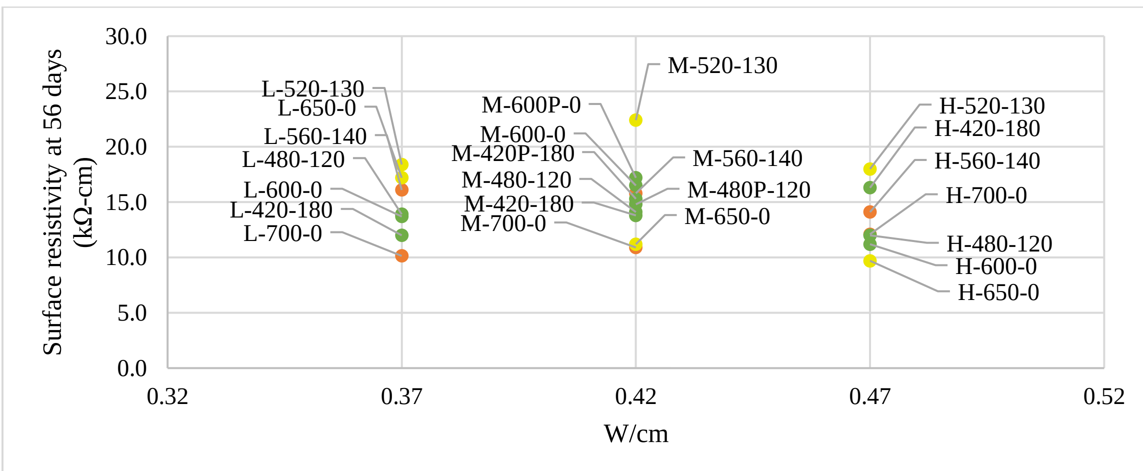

Figure 4.15 w/cm vs. surface resistivity averages at 56 days

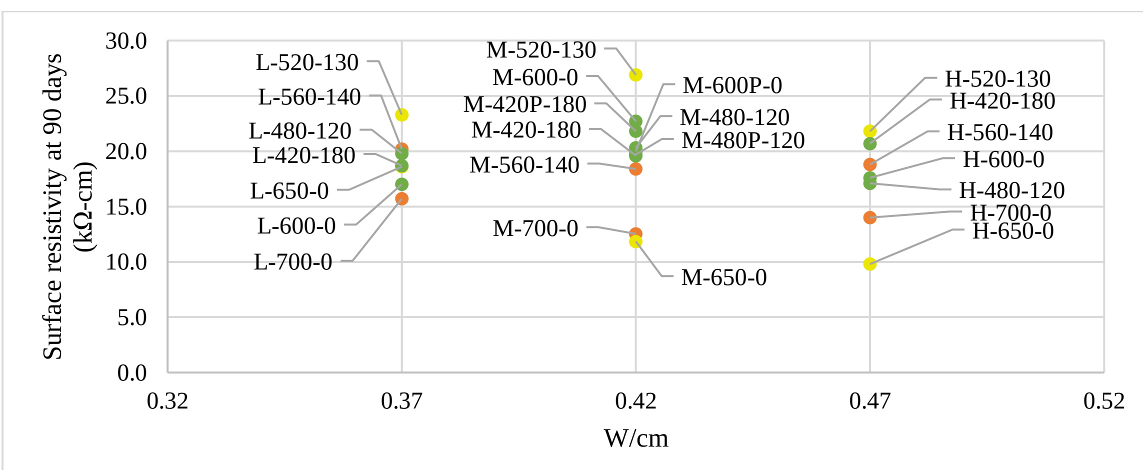

Figure 4.16 w/cm vs. surface resistivity averages at 90 days

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.

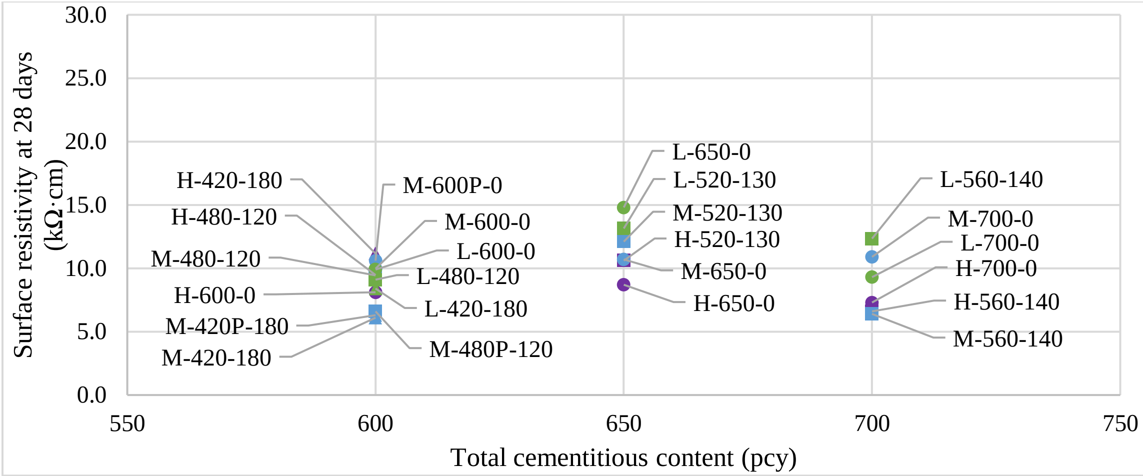

Figure 4.17 Total cementitious content (pcy) vs. surface resistivity averages at 28 days

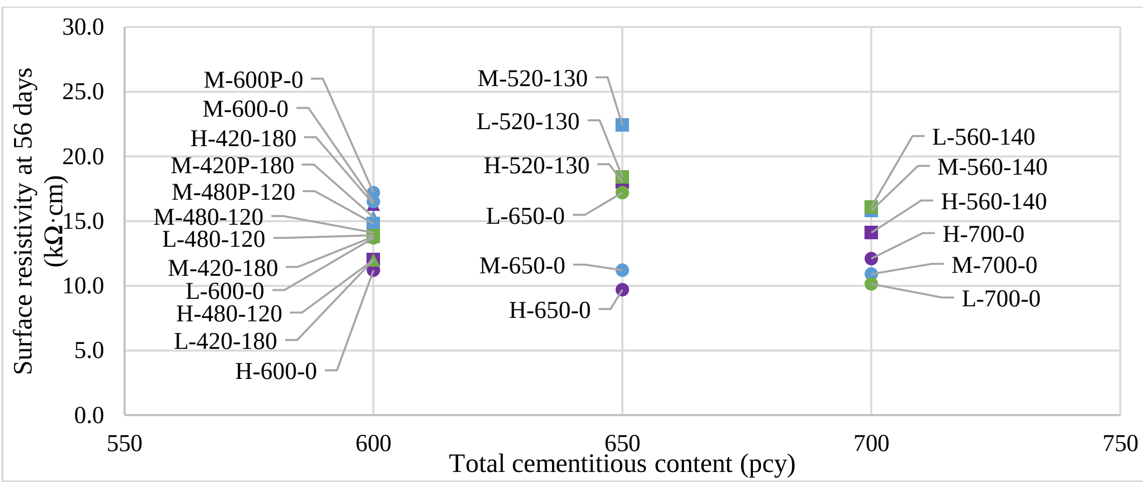

Figure 4.18 Total cementitious content (pcy) vs. surface resistivity averages at 56 days

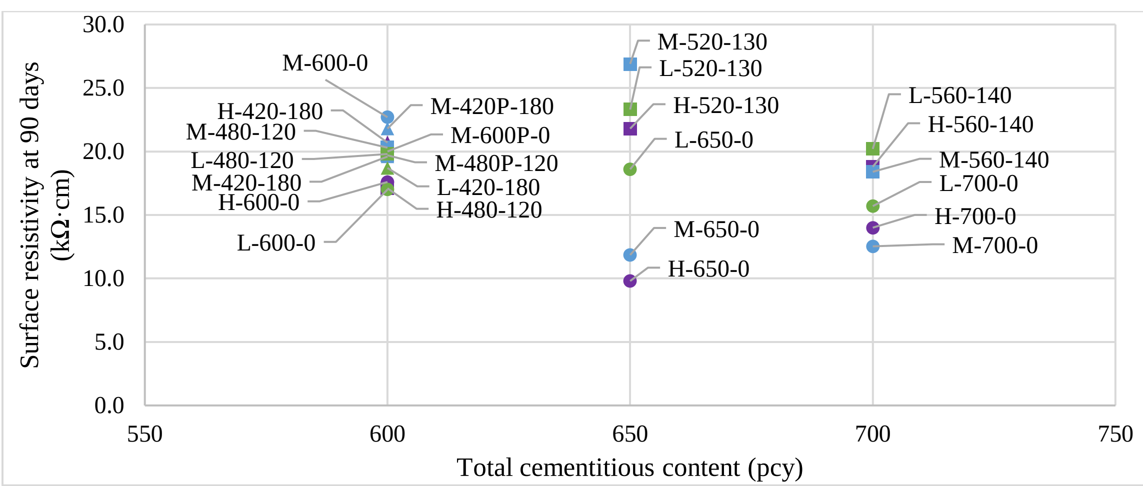

Figure 4.19 Total cementitious content (pcy) vs. surface resistivity averages at 90 days

When straight cement mixtures are graphed by cement content vs. surface resitivity at 28 days, the superior performance of lower w/cm is readily observable. The 0.47 w/cm has the lowest surface resistivity averages for each of the three cementitious contents. The 650 pcy mixtures for the 0.47 and 0.42 slightly outperformed their 600 pcy and 700 pcy companion mixtures. The 0.37 w/cm 650 pcy mixtures significantly outperformed its companion mixtures. This can be seen in Figure 4.20.

At 56 days, the 600 pcy mixtures improve their performance when compared to 650 pcy and 700 pcy mixtures. A trend can begin to be seen in Figure 4.21 showing the improved performance for surface resistivity testing of mixtures with lower cement contents.

The trend observable at 56 day testing comes more prevelant at 90 days. It can be seen that in Figure 4.22 that in most cases, the mixtures with lower cement contents perform better than those with higher contents.

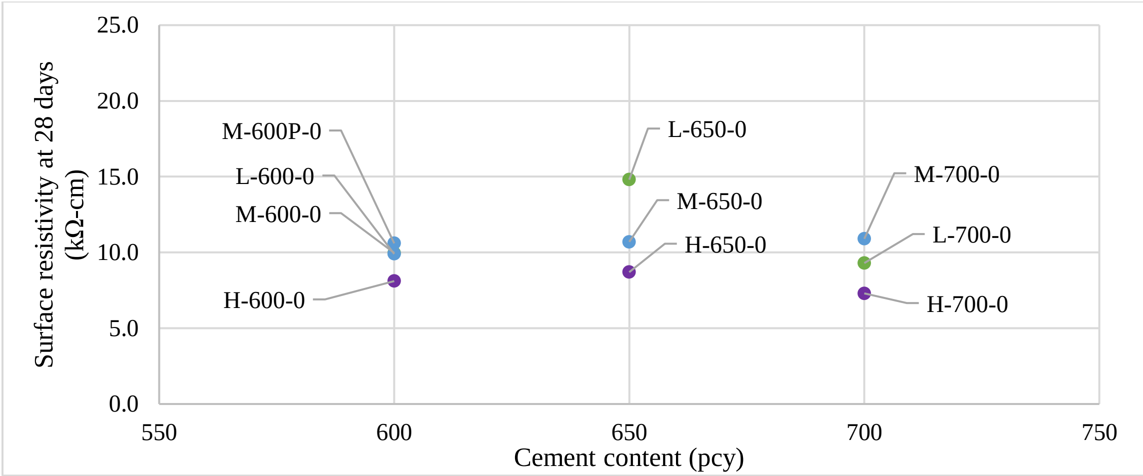

Figure 4.20 Straight cement mixture cement content vs. surface resitivity at 28 days

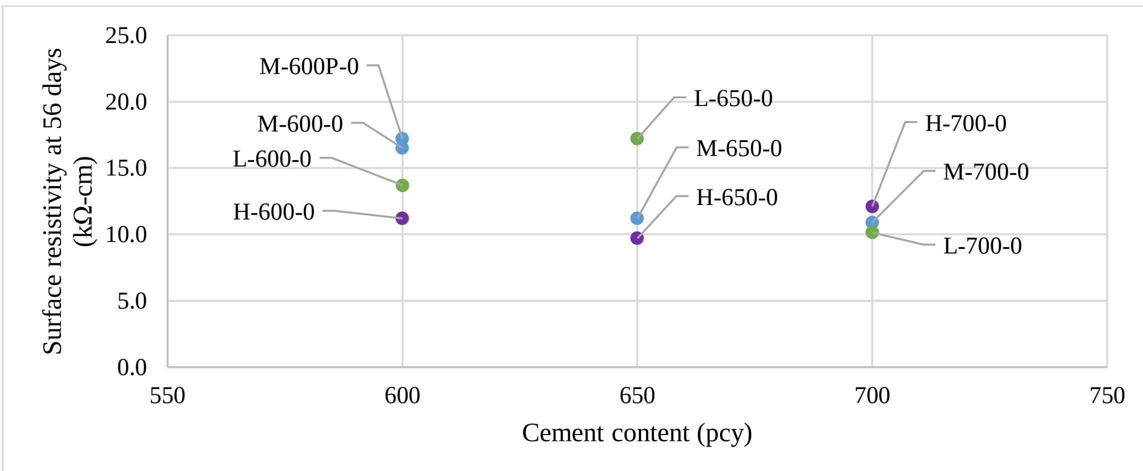

Figure 4.21 Straight cement mixture cement content vs. surface resitivity at 56 days

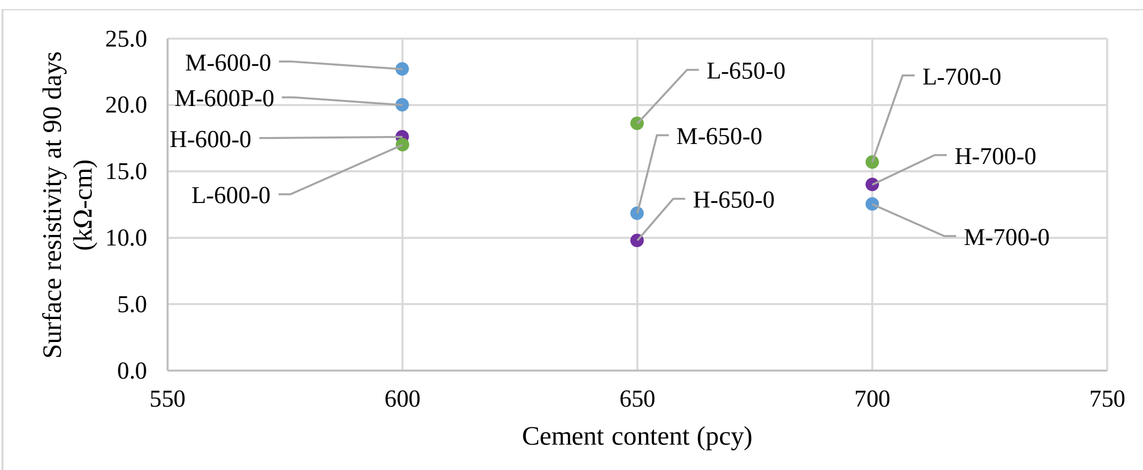

Figure 4.22 Straight cement mixture cement content vs. surface resitivity at 90 days

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. 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. These mixtures can be seen in Figure 4.23.

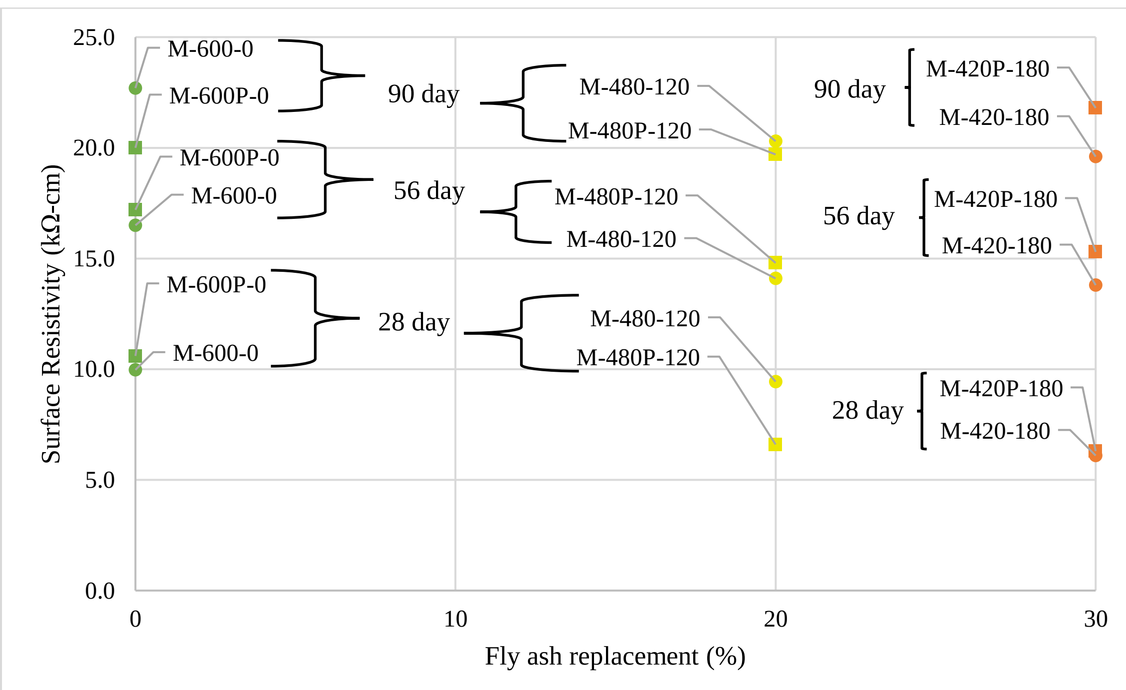

Figure 4.23 Fly ash replacement % vs. surface resistivity for OPC and PLC companion mixtures at 28, 56, and 90 days

4.2.2.2 Bulk Resistivity

The bulk resistivity testing results can be found in Table 4.3. 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 4.24. Mixtures with a fly ash replacement also typically performed better than their companion non-fly ash mixtures, particularly when comparing 90 day results.

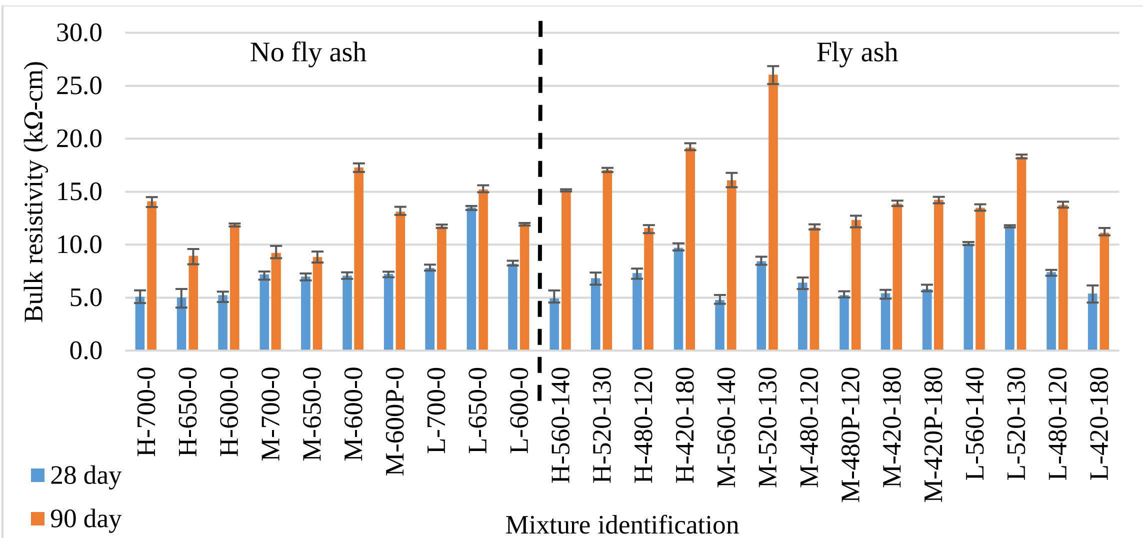

Figure 4.24 Bulk resistivity with mixtures sorted by non-fly ash and fly ash mixtures

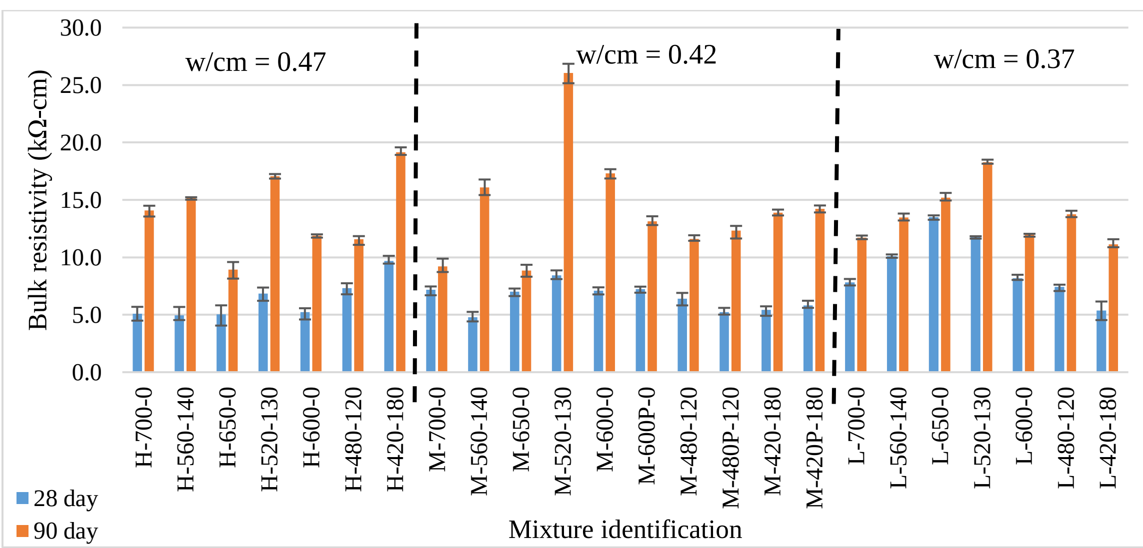

Figure 4.25 Bulk resistivity results with mixtures sorted by w/cm

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 can be seen in Figures 4.26 and 4.27, 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 4.26 with an R^2 value of 0.86. However, as can be seen in Figure 4.27, a power model provided a slightly better fit to this data with an R^2 value of 0.89. The cause of this stronger non-linear relationship is not readily evident at this time, and it is acknowledged that the linear fit may be stronger if a more robust dataset is utilized.

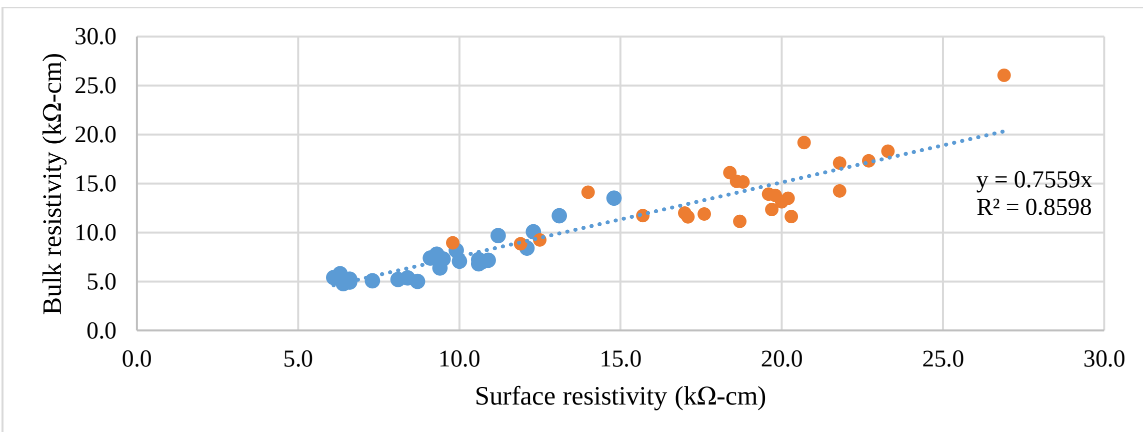

Figure 4.26 Bulk resistivity vs. surface resistivity linear model

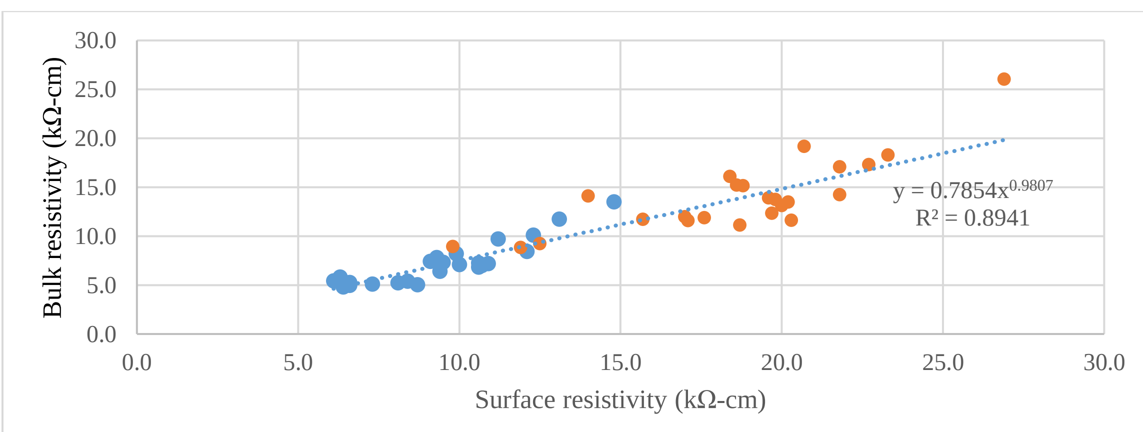

Figure 4.27 Bulk resistivity vs. surface resistivity power model

4.2.2.3 Chloride Permeability

The results of the RCPT testing for each mixture are presented above in Table 4.3, and are graphically shown in Figures 4.28 and 4.29 below. Two samples were tested for each of the 24 mixtures at ages 28 and 90 days, with the average of the two samples at each date reported. ASTM C1202 performance criteria, linking charge passed to chloride ion penetrability are shown in Table 4.5 (ASTM 2018). Figure 4.28 shows mixtures grouped by the presence or absence of fly ash, and Figure 4.29 shows mixtures grouped by w/cm.

Table 4.5 Chloride permeability performance criteria

Charged passed (coulombs)	Chloride ion penetrability
> 4,000	High
2,000 – 4,000	Moderate
1,000 – 2,000	Low
100 – 1,000	Very low
< 100	Negligible

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 performances. The two mixes at 28 days without fly ash 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, which could provide reasoning. These observations can be seen in Figure 4.28. A graphical comparison of 28 and 90 day RCPT sorted by total cementitious content and w/cm is shown in Figure 4.29. 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 mixes 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.

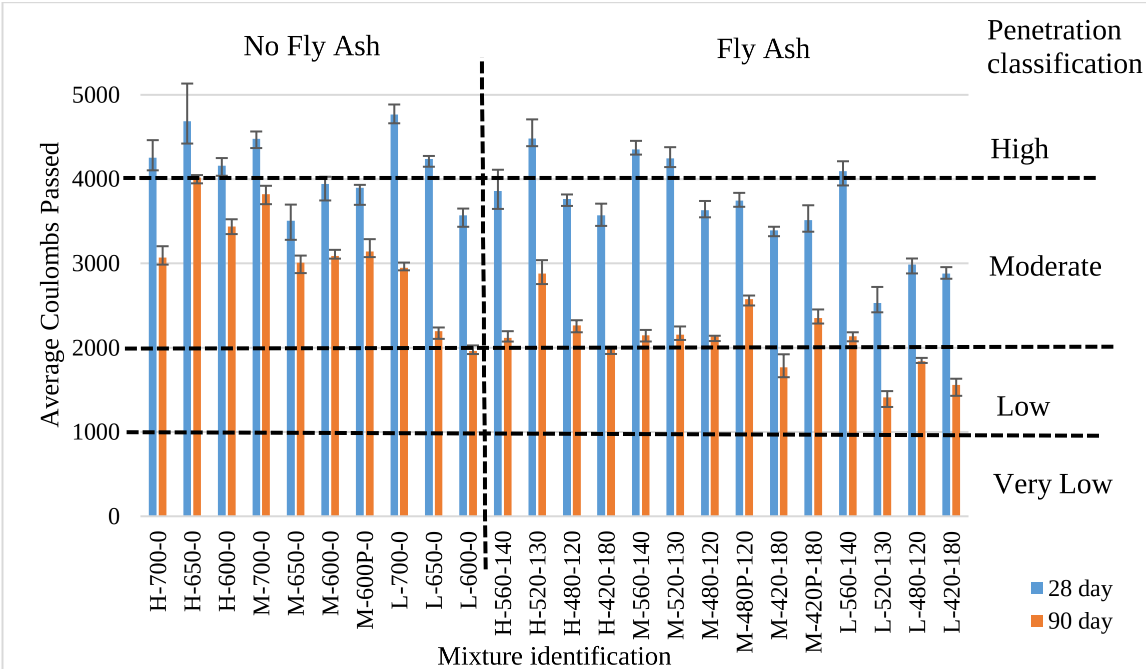

Figure 4.28: RCPT results with mixtures sorted by non-fly ash and fly ash mixtures

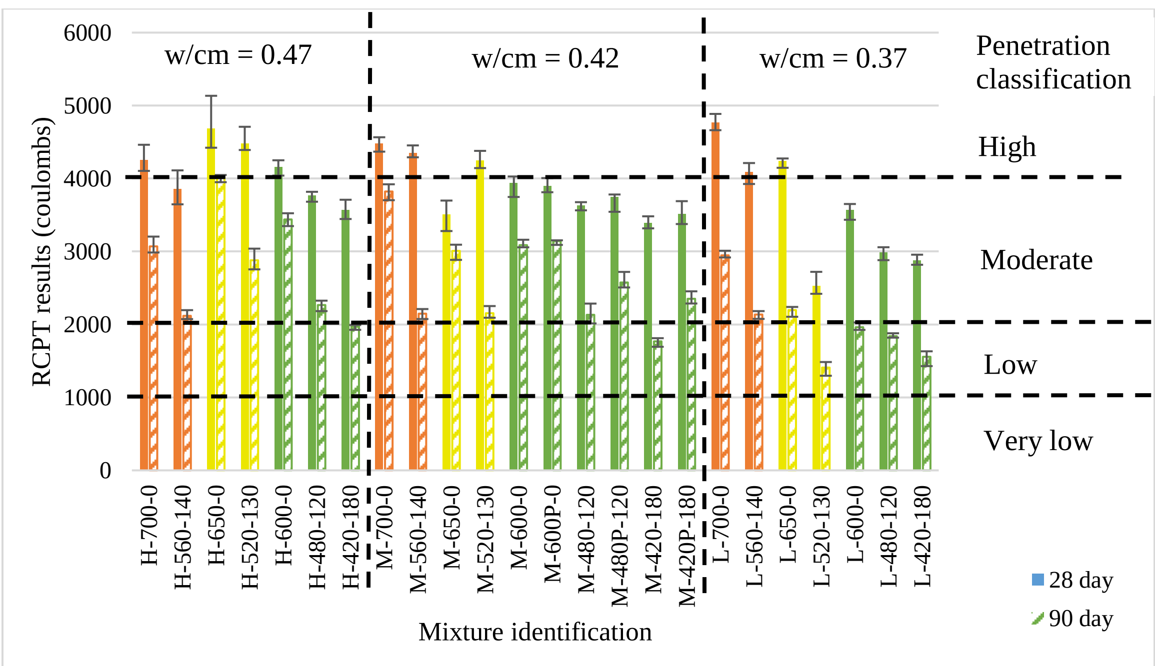

Figure 4.29: RCPT results with mixtures sorted by w/cm

4.2.2.4 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 writing this thesis, the Bucket Test procedure had recently been released by Weiss et al., and this work should be considered preliminary at this time.**

Two samples per mixture were tested using the procedure developed by Dr. Weiss (described in Section 3.8.2.4) 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. A table showing selected sample calculations and conversions for Bucket Test and formation factor values can be seen in Table 4.6.

Table 4.6 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

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). This pore solution is used to adjust measured resistivity values using a standardized value for typical pore solution resistivity.

The measured formation factor averages can be found in Table 4.3. 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 electric 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 had nearly identical formation factor values. These trends can be seen in Figures 4.30 and 4.31. Figures 4.32 and 4.33 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 4.7 shows the formation factors associated with various levels of chloride ion penetrability, as found in AASHTO PP 84.

Table 4.7 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

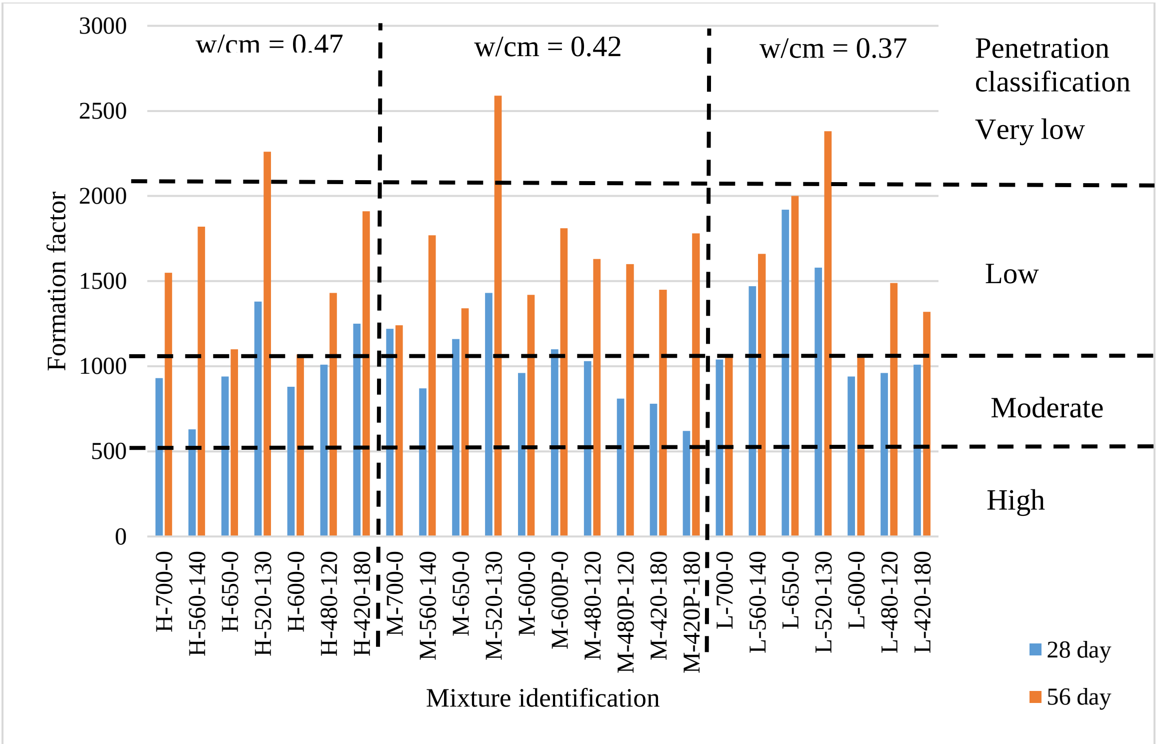

Figure 4.30 Formation factor values

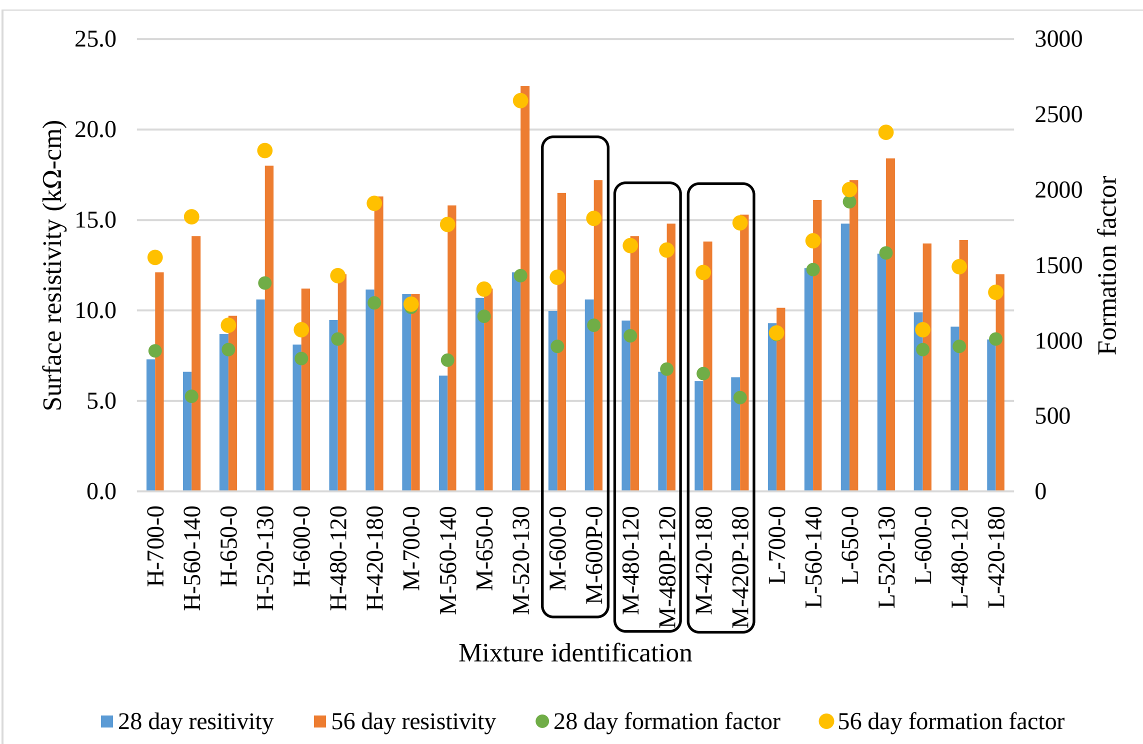

Figure 4.31 Formation factor and surface resistivity at 28 and 56 days

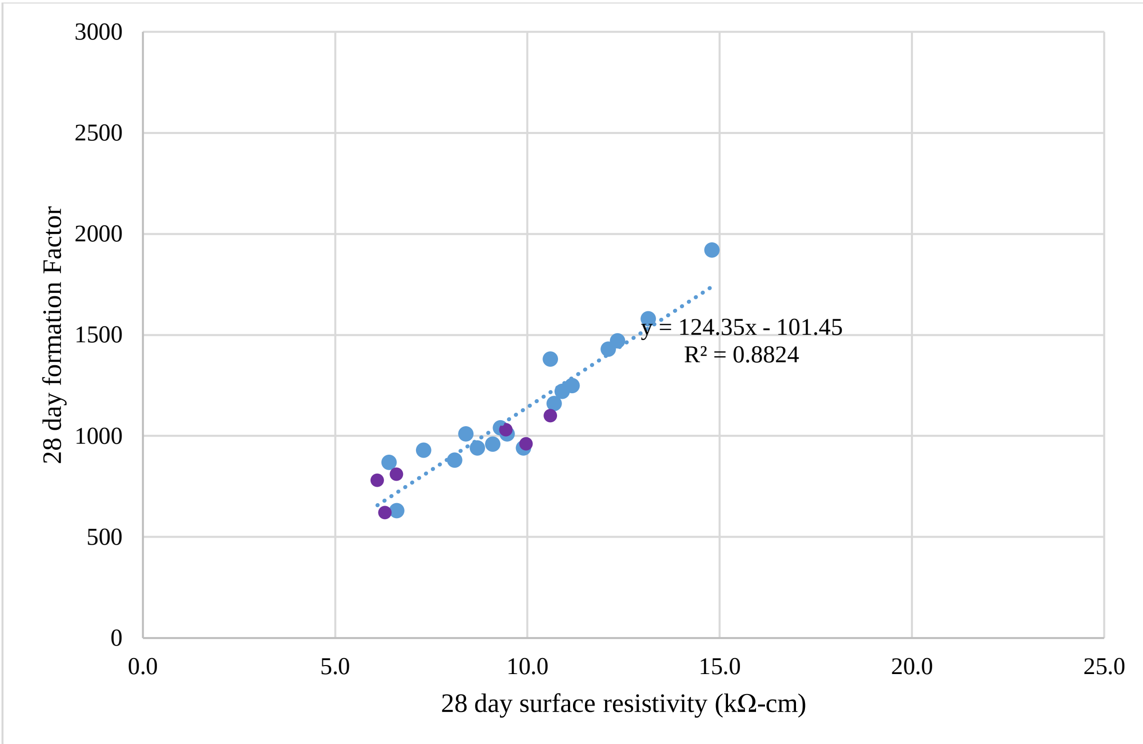

Figure 4.32 Formation factor vs. surface resistivity as 28 days

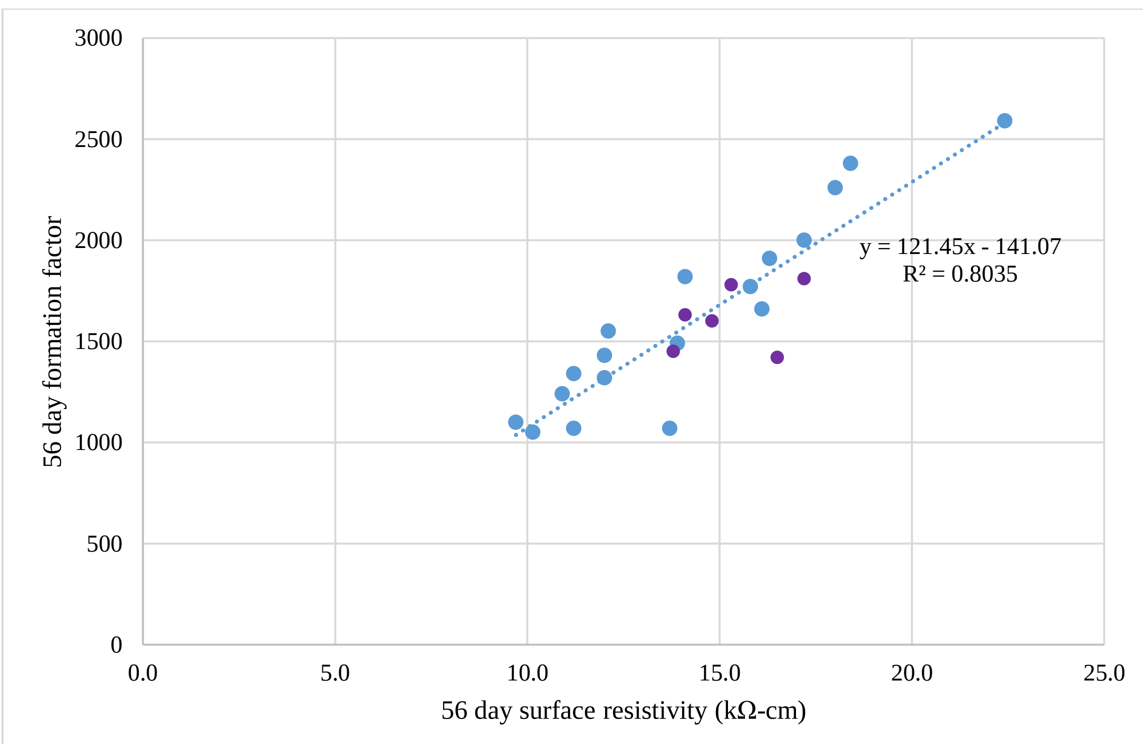

Figure 4.33 Formation factor vs. surface resistivity at 56 days

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.

4.3 Summary of Findings

The experimental program utilized for the 24 mixtures included in the laboratory portion of this research provided a large array of data for analysis of the influence of mixture inputs and proportions on mechanical properties and durability performance tests. Some of the key findings of the research are as follows:

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 ratio 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% replacement by weight of cement) should have minimal impact on concrete's long-term strength and other mechanical properties. 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 test values for 3 day compressive strength, MOR, and MOE, although the average difference for each of these tests 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 is similar to that of other NCDOT concrete studies performed by this research team, 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.

<u>Durability Performance</u>

- 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 can be seen with greater improvements between 28 and 90 day tests for lower total cementitious material contents, especially for those with a fly ash replacement.

    - 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 improved with 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 was observed 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. Twenty-eight 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.

CHAPTER 5: DEVELOPMENT OF RECOMMENDED SURFACE RESISTIVITY SPECIFICATION

5.1 Introduction

This chapter provides a summary of current specifications utilized by various state DOTs for RCPT and surface resistivity, as well as an in-depth analysis of surface resistivity and chloride permeability data measured using North Carolina concrete mixtures. To provide a more robust data set to support development of the specification recommendations for NCDOT, portions of the analysis were expanded past the 24 mixtures produced for this project to include an additional 23 mixtures from 3 previous research projects on North Carolina bridge and pavement concrete batched and tested by the research team. Although there have been many more than 23 mixtures produced as part of these 3 previous projects, these 23 were included due to the strong similarity in mixture materials and proportions characteristics. Mixtures for these other research projects that did not utilize conventional materials, e.g. utilized prewetted lightweight sand for internal curing (Leach 2017) or beneficiated fly ash (Ojo 2018) were not utilized.

The characteristics of these additional mixtures can be found below in Table 5.1. It should be noted that these mixtures were completed prior to the NCDOT Standard Specification for fly ash replacement changing from 1.2 lbs of fly ash per 1.0 lb of cement replacement to a 1:1 ratio. These ash replacement rates are noted by a "*". Color coding, consistent with that used previously in this thesis, has been added to the table to show pavement mixtures (green) and structural mixtures (orange), as well as to highlight the w/cm similarities to the mixtures batched and tested as part of this project (higher w/cm of 0.48 in purple, lower w/cm of 0.35 in green).

Table 5.1 Additional North Carolina concrete mixtures included in expanded dataset

Mixture ID	Mixture Characteristics			Mixture Proportions, pcy				
	Mixture type (project publication)	**w/cm**	**Fly ash replacement level (%)**	**Cement**	**Fly Ash**	**Coarse Aggregate**	**Fine Aggregate**	**Water**
P.A.N.M	Paving (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	Paving (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 (Leach 2017)	0.35	0	715	0	1720	1113	266
CF		0.35	20*	572	172	1720	1113	266

5.2 Analysis of Relevant Requirements

Efforts to develop the results found from this and previous research projects into a surface resistivity specification, included a review of existing state highway agency specifications for chloride-resistant concrete using 1) RCPT and 2) surface resistivity. Two AASHTO standards were also included in the review.

5.2.1 Applicable Standards

The two AASHTO standards evaluated for this portion included AASHTO PP 84 and AASHTO T 358. These are the standards for performance engineered concrete mixtures and surface resistivity, respectively (AASHTO 2017, 2017). The standard testing methods for chloride permeability, ASTM C1202 and AASHTO T 277, have the same permeability classifications as those listed below for AASHTO PP 84 (AASHTO 2015, ASTM 2018). Table 5.2 below shows the requirements set forth in each standard, with all values presented being applicable for testing on 4 in by 8 in cylinder specimens, or samples created from drilled core samples of the same size.

Table 5.2 Relevant AASHTO standards for development of surface resistivity specification

Standard	RCPT Specification			Surface Resistivity Specification		
	Concrete Type	Requirement (coulombs)	Age	Concrete Type	Requirement (kΩ-cm)	Age
AASHTO T 358	-	-	-	High chloride risk	< 12	28 Days
	-	-	-	Moderate chloride risk	12 - 21	28 Days
	-	-	-	Low chloride risk	21 - 37	28 Days
	-	-	-	Very low chloride risk	37 - 254	28 Days
	-	-	-	Negligible chloride risk	> 254	28 Days
AASHTO PP 84	High chloride risk	> 4,000	28 Days	High chloride risk	< 5	91 Days
	Moderate chloride risk	2,000 – 4,000	28 Days	Moderate chloride risk	5 - 10	91 Days
	Low chloride risk	1,000 – 2,000	28 Days	Low chloride risk	10 - 20	91 Days
	Very low chloride risk	100 – 1,000	28 Days	Very low chloride risk	20 - 200	91 Days
	Negligible chloride risk	0 - 100	28 Days	Negligible chloride risk	> 200	91 Days

Both of the above standards specify the required values based on specimens undergoing a standard moist cure period, and specify values based on the risk of chloride ion penetration. As shown, the surface resistivity requirements set forth in AASHTO T 358 are slightly more aggressive than those set forth in AASHTO PP 84, in regard to both target values and age at which values must be achieved. The values presented for surface resistivity in AASHTO PP 84 can also be applied to a 28 day test if accelerated curing conditions are used. Accelerated curing conditions are presented in the standard as a standard 3 day moist cure, followed by 25 days of curing at 122 ºF.

5.2.2 Applicable State Specifications

Standards 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. 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 5.3. A description of the type of requirement for each specific state, concrete type, and targeted testing values at specific ages are required. A table summarizing all state RCPT and surface resistivity requirements can be found in Appendix C, Table C.1.

Table 5.3 Relevant state specifications for development of a surface resistivity specification

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	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	> 22	28 days

				P1, P2, P3; CIP Structural: Class S		
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	-	-	-	Superstructures and substructures	> 24	28 days

proposed thresholds for design mix performance 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
	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,	Must have "low to negligible risk" according to AASHTO T 358	

				AA (AE) = other structural elements)		
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

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, New York, Rhode Island, Texas, Virginia, and West Virginia (KDOT 2015, VDOT 2016, WVDOT 2016 NYDOT 2018, RIDOT 2018, TDT 2004). Of these states, 4 of them (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' 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, Louisiana (LADOTD 2016), Montana (MDOT 2014), New Hampshire (NHDOT 2016), New Jersey (Nassif et al. 2015), New York, Rhode Island, 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). 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).

To assist in the development of a surface resistivity specification for NCDOT, it was decided to focus upon specifications of Virginia's DOT (VDOT). VDOT has shown improved permeability characteristics in RCPT results through the use of SCMs. These results were shown through a study on seven typical VDOT concrete mixtures, with the six containing an SCM meeting 28 day RCPT requirements. The one control mixture containing no SCM had significantly higher RCPT values, and did not meet 28 day requirements (Sharp et al. 2014). When reviewing the states with RCPT and/or surface resistivity specifications included in Table 5.3, Virginia was determined to be the most similar to North Carolina for the following reasons:

- proximal geographical location and similar climate
- similar mountain, piedmont, and coastal regions
- similar population distribution (major urban corridors and rural lands) and highway network conditions

It was also determined that provisions or targets of a number of state specifications in Table 5.3 were likely not appropriate for NCDOT mixtures due to various aspects of the specifications. These include those by 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 do 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 North Carolina concrete mixtures. One example of a provision viewed as too aggressive for current North Carolina 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.

5.3 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 previously in thesis, North Carolina concrete mixtures show a strong correlation between RCPT and surface resistivity (Figures 5.1 and 5.2). For these and a number of subsequent RCPT vs. surface resistivity figures, 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. Previous research studies performed by the research team both had R^2 values of 0.94 (RP 2015-03 and 2016-06). The expanded dataset, including mixtures produced for this projected and the ones shown in Table 5.1 had an R^2 value of 0.77 as shown in Figure 5.2.

Due to the confidence in the correlation between RCPT and surface resistivity data, as well as the confidence in field performance of VDOT mixtures utilizing their RCPT specifications (Sharp et al. 2014), RCPT targets based on VDOT's current specifications were utilized to identify corresponding surface resistivity targets using North Carolina 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, VDOT's RCPT requirements were plotted against the RCPT and surface resistivity data presented in Sections 4.2.2.1 and 4.2.2.3 to determine the surface resistivity measurements associated with those RCPT values for NCDOT mixtures. These values were determined to be 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.

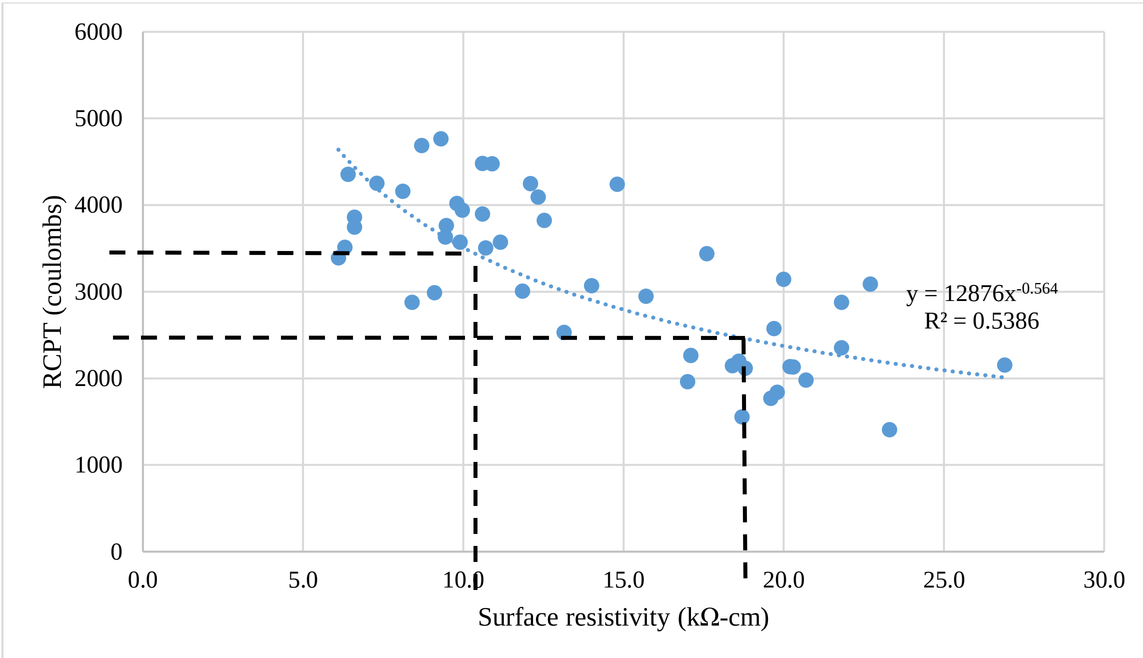

Figure 5.1 28 and 90 day RCPT vs. surface resistivity with target RCPT values

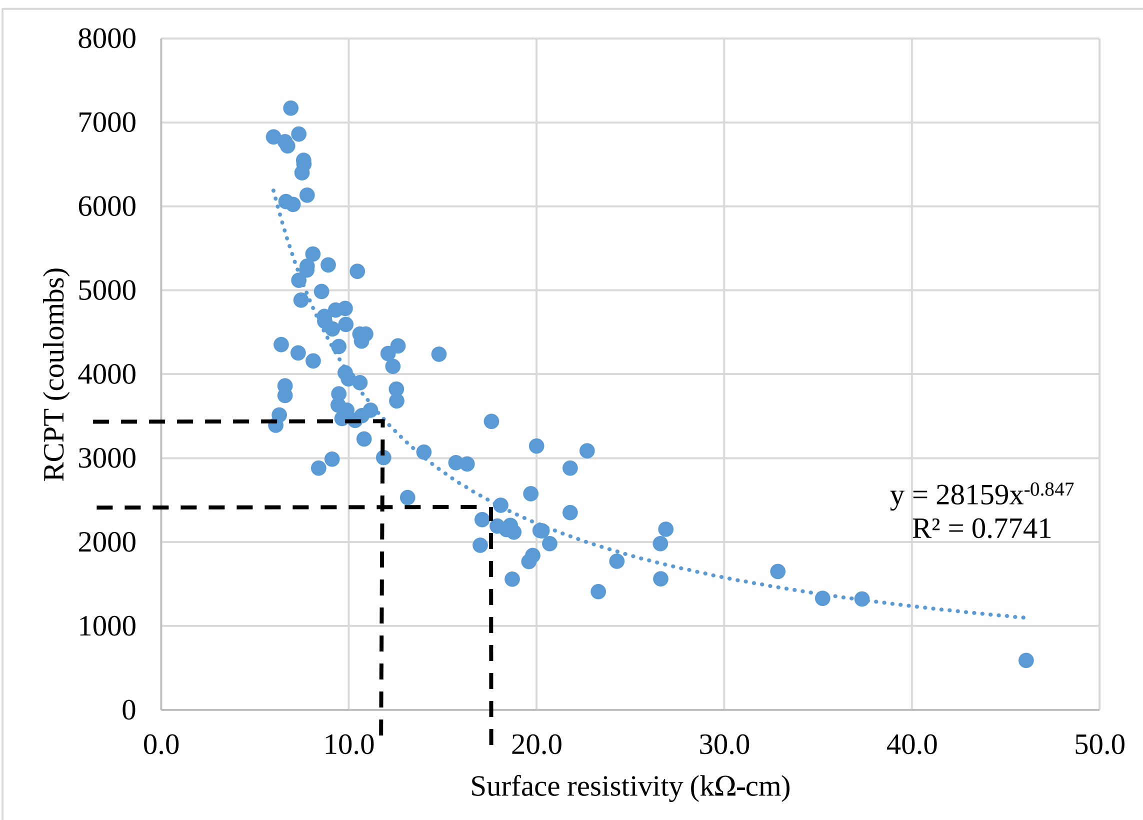

Figure 5.2 Expanded dataset RCPT vs. surface resistivity at 28 and 90 days with target RCPT values identified

Virginia DOT's values are associated with 28 day RCPT test results. Many North Carolina 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 was determined that the target values would be applied to 56 day surface resistivity testing to encourage use of fly ash mixtures, lower w/cm ratios, and other SCMs in North Carolina infrastructure. New Jersey 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.

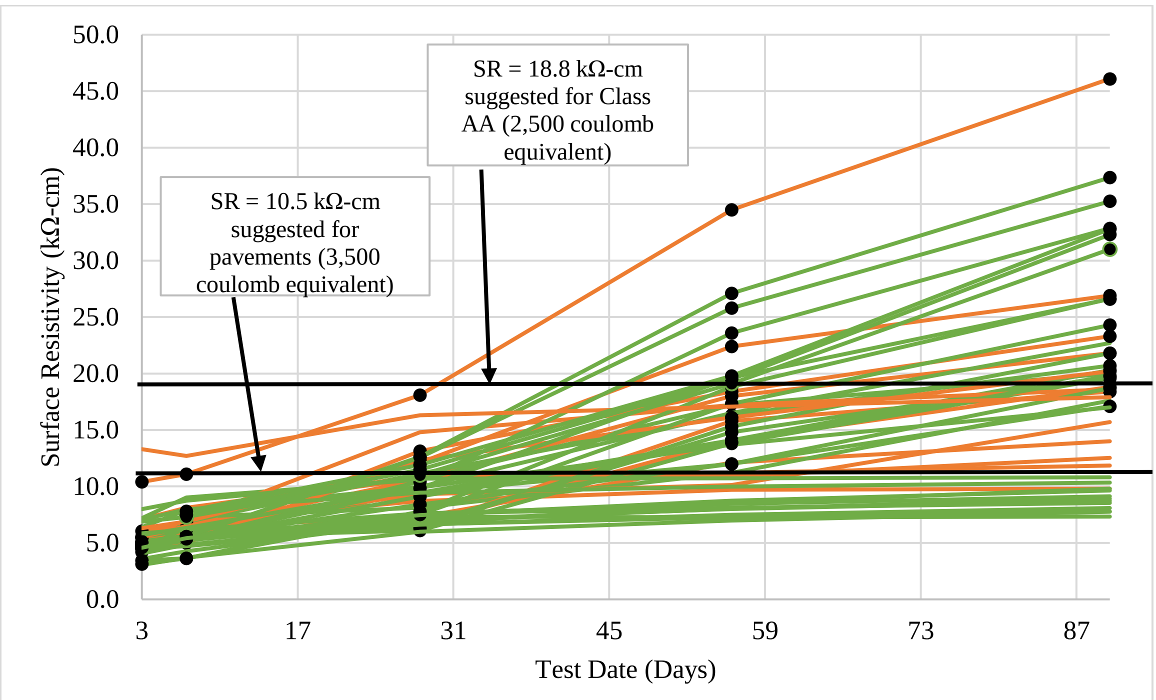

Figure 5.3 Surface resistivity data with target resistivity goals identified

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. This can be seen in Tables 5.4 and 5.5. Upon finding the percent of mixtures passing at various target values, evaluation of the mixture characteristics of those passing and failing was performed. This was done to ensure the mixtures designed to have better durability properties fell within the passing mixtures at 56 days. For bridge mixtures, this evaluation can be seen in Tables 5.6 through 5.10.

Table 5.4 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	7.14%	28.57%	7.14%	42.86%	14.29%	50.00%	14.29%	57.14%

Table 5.5 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	21.43%	64.29%	28.57%	64.29%	42.86%	78.57%	42.86%	78.57%

Table 5.6 Bridge mixtures passing and 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 target		Not meeting 18.0 kΩ-cm target		Meeting 17.0 kΩ-cm target		Not meeting 17.0 kΩ-cm target	
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	

Table 5.7 Bridge mixtures passing and failing at 28 and 56 days for performance targets 16.0 kΩ-cm and 15.0 kΩ-cm

Target value	Meeting 16.0 kΩ-cm target		Not meeting 16.0 kΩ-cm target		Meeting 15.0 kΩ-cm target		Not meeting 15.0 kΩ-cm target	
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	

Table 5.8 Bridge mixtures passing and 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 target		Not meeting 14.0 kΩ-cm target		Meeting 13.0 kΩ-cm target		Not meeting 13.0 kΩ-cm target	
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					

Table 5.9 Bridge mixtures passing and 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 target		Not meeting 13.0 kΩ-cm target		Meeting 12.0 kΩ-cm target		Not meeting 12.0 kΩ-cm target	
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		

Table 5.10 Bridge mixtures passing and failing at 28 and 56 days for performance target 11.0 kΩ-cm

Target value	Meeting 11.0 kΩ-cm target		Not meeting 11.0 kΩ-cm target	
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		

Evaluation of the above tables 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 surface resistivity target. 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 with no 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.

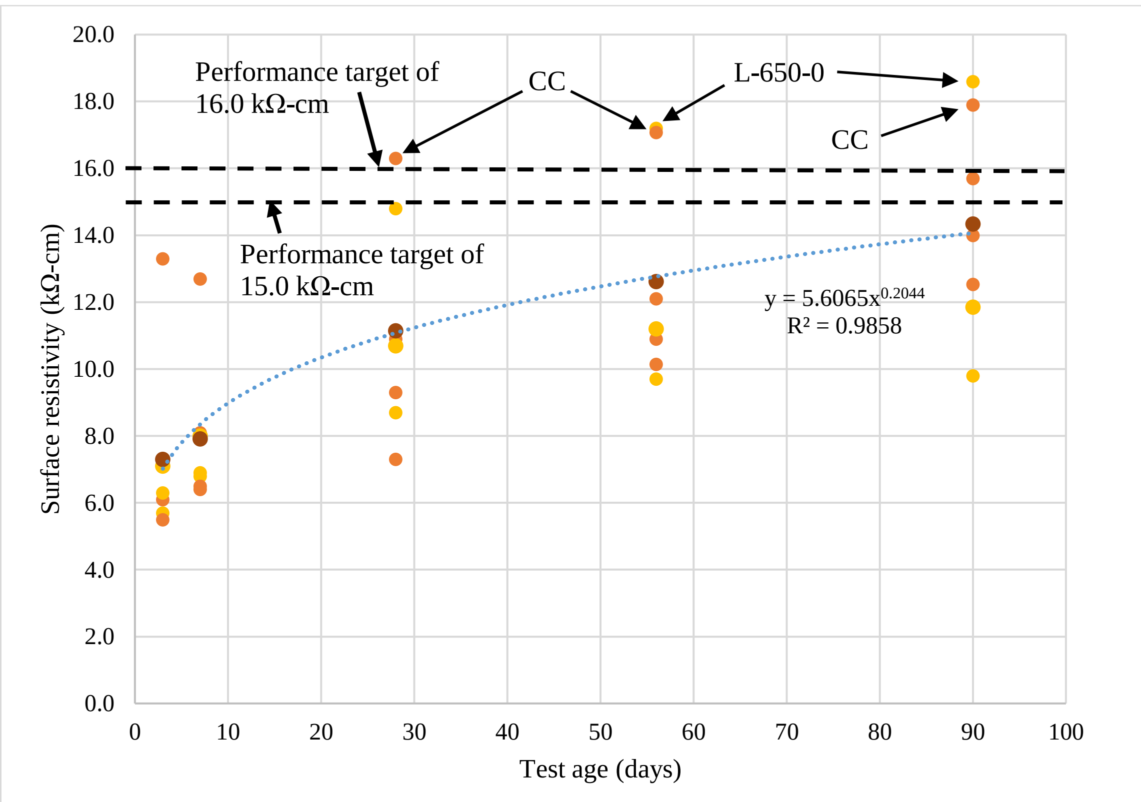

Figure 5.4 Surface resistivity averages for straight cement bridge mixtures with 15.0 and 16.0 kΩ-cm targets

In regards to paving mixtures, Tables 5.11 to 5.12 were used similarly to identify a target surface resistivity value of 11.0 kΩ-cm meeting the same criteria discussed for the bridge mixtures for both passing and failing mixtures at 56 days. This value roughly corresponds to an RCPT of approximately 3,300 coulombs.

Table 5.11 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
Percent passing	18.18%	63.64%	30.30%	69.70%

Table 5.12 Paving mixtures passing and failing at 28 and 56 days for various performance targets

Target value	Meeting 11.0 kΩ-cm target		Not meeting 11.0 kΩ-cm target		Meeting 10.0 kΩ-cm target		Not meeting 10.0 kΩ-cm target	
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 North Carolina 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 color coded 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. These mixtures are identified in Figure 5.4.

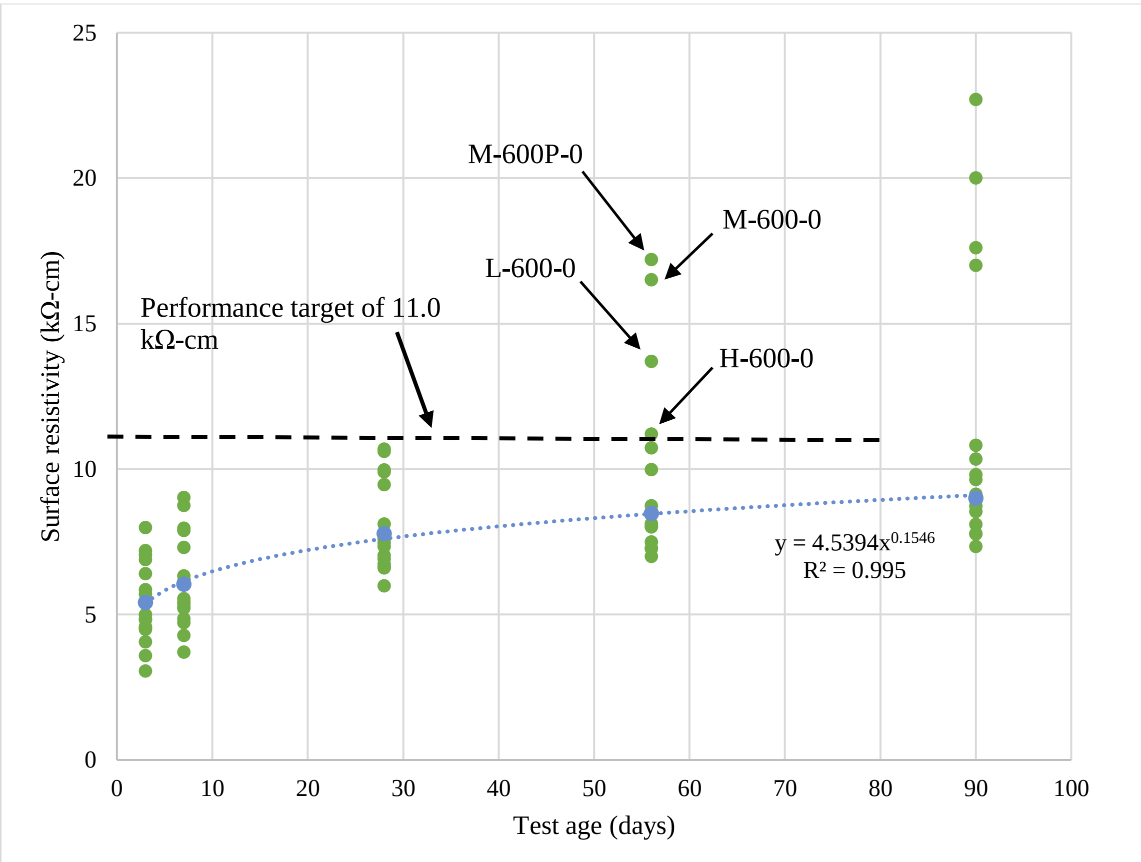

Figure 5.5 Surface resistivity averages for straight cement paving mixtures with 11.0 kΩ-cm target

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.

5.4 Summary of Findings

The following findings were used in development of a surface resistivity specification for NCDOT paving and bridge mixtures:

- Virginia was selected as a state of key interest to support specification development for North Carolina due to similarities in climate, geography, population distribution (urban corridors along with much rural land area) and highway network. VDOT's RCPT target values at 28 days are 3,500 coulombs for paving and 2,500 coulombs for bridges, and these target values have been reported to correlate well with satisfactory field performance (Sharp et al. 2014).
- To promote use of SCMs for improved durability, an age of 56 days was selected for the resistivity targets. Use of test results at 56 days rather tan 28

days should allow for adequate hydration of SCMs such as fly ash, and is consistent with other state resistivity specifications (Nassif et al. 2015).

- When compared to the expanded dataset, it was determined that NCDOT concrete mixtures had associated surface resistivity values at 56 days of approximately 10.5 kΩ-cm for paving applications and 18.8 kΩ-cm for bridge applications.
- Evaluation of the mixtures meeting the target value for bridges determined 18.8 kΩ-cm at 56 days may be slightly aggressive for application by the NCDOT. Various target values near 18.8 kΩ-cm were used to determine the target value associated with a sufficient number of quality mixtures passing.
- Target values of 11.0 kΩ-cm for paving applications and either 15.0 kΩ-cm or 16.0 kΩ-cm for bridge applications at 56 days were selected. For bridge applications, 15.0 kΩ-cm should be considered a realistic and achievable target value at 56 days. A target value of 16.0 kΩ-cm could be considered a more aggressive target value at 56 days for mixtures with a lower w/cm or utilizing a fly ash replacement.
- These initially suggested target values could be made more aggressive (reduced) if expanded data analysis and field performance suggests that new targets could be readily met by producers and contractors. This would support further improvements in the durability performance of North Carolina highway infrastructure.

5.5 NCDOT 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. As it stands, Section 1000-4C is presented as follows:

(C) Strength of Concrete

> The compressive strength of the concrete will be considered the average compressive strength 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. When the average compressive strength of the concrete test cylinders is less than the minimum strength specified in Table 1000-1 and the Engineer determines it is within reasonable close conformity with strength requirements, concrete strength will be considered acceptable. When the Engineer determines average cylinder strength 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 suggested revision to include application of a surface resistivity specification by the NCDOT is as follows:

(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 would be added or incorporated into Table 1000-1 with the associated footnote:

Table 5.13 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 higher.

CHAPTER 6: CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE WORK

This thesis presents the results found from laboratory batching and testing of concrete representative of NCDOT paving and bridge deck mixtures, along with analysis of the test results to support development of a proposed surface resistivity specification for use by NCDOT to improve the durability performance of concrete pavements and structures. A review of other state specifications focused on surface resistivity, paired with an analysis of laboratory data from North Carolina concrete mixtures, was used to support identification of target surface resistivity values for NCDOT's structural and pavement mixtures that could be readily met by well-proportioned mixtures exhibiting adequate mechanical strength and durability performance characteristics.

Work performed as part of this effort leveraged knowledge gained in a series of previous project for NCDOT that support evaluation of the durability performance of North Carolina concrete mixtures. In previous projects, the organization of the concrete mixture matrix was created to explore an array of different materials held at consistent proportions. This was done in order to evaluate the influence of the various materials on a wide assortment of tests, many of which were mentioned in this thesis. However, for this project, materials were the constant (aside from the PLC mixtures) in order to explore the effect of different proportions of the same materials on test results, particularly key mechanical properties (strength, MOE, MOR) and the durability performance tests of RCPT and surface resistivity. Additionally, preliminary data on an emerging test utilized in AASHTO PP 84, the Bucket Test, and corresponding formation factor values were presented for informational purposes.

Overall, this work supports NCDOT's larger effort aligned with FHWA's PEM initiative, and conclusions and recommendations presented in this thesis should be considered a preliminary step in NCDOT's movement towards PEM specifications for more durable, sustainable concrete infrastructure.

This chapter will serve to summarize the conclusions found from these test results and research, as well as provide recommendations for future and ongoing projects related to durable concrete.

6.1 Conclusions

Laboratory testing of the 24 mixtures, along with the evaluation of the results of these and previous research projects provided valuable information regarding various aspects influencing concrete durability, namely the resistance to chloride ion penetration. The key findings from the laboratory testing are as follows:

- Portland limestone cement does not have a detrimental effect on the mechanical or durability performance of NCDOT concrete mixtures, and performance of companion OPC and PLC mixtures was similar.
- The well-established fact that lower w/cm mixtures generally have superior mechanical properties and durability performance characteristics to those with a higher w/cm was confirmed, indicating that NCDOT may wish to explore use of a prescriptive specification provision reducing w/cm to encourage use of WRAs, optimized aggregate gradations, and fly ash. This should result in lower paste contents and improved durability performance.
- Although straight cement mixtures typically have better mechanical properties fly ash mixtures, particularly at early ages, there is little to no impact to later-

age mechanical properties in regards to increasing fly ash replacement rates from 20% to 30%.

- For a given cementitious content, fly ash mixtures exhibit improved durability test performance than straight cement mixtures, particularly at later ages. Current NCDOT specifications requiring 4500 psi compressive strength at 28 days may preclude many of these mixtures, which exhibit superior durability, from being utilized. NCDOT should revisit the 28-day compressive strength requirements as part of their effort to move towards specifications for PEM.

The results of the laboratory testing indicate that NCDOT concrete mixtures may not meet some of the more aggressive RCPT and surface resistivity specifications set forth by other states at early ages. At later ages, such as 56 days, some other state specification targets are achievable, particularly by fly ash mixtures. To produce a preliminary specification for possible implementation by the NCDOT, values from a state with similarities to North Carolina were evaluated against the laboratory results. These results were used to determine achievable target values that could be used as provisional specification targets on future NCDOT paving or bridge deck projects.

It is understood that the implementation of new or proprietary testing methods, such as the PEM test methods (surface resistivity, the Bucket Test, SAM, etc.), can be met with hesitation from contractors, as they are burdened with the task of becoming familiar with implementing a new test and meeting new specifications. However, the ease of performance of the surface resistivity test, as well as its low variation and ability to be performed within minutes on the same test specimens as the required compressive strength cylinders, should be met with less hesitation from contractors than could typically be expected. In order to allow for further ease of implementation, the suggested method of specification for surface resistivity was integrated into NCDOT's current compressive strength testing specifications. This should streamline the implementation process, as well as provide minimal modifications to existing (and currently well understood) NCDOT specifications.

6.2 Recommendations for Future Work

This section provides recommendations for future work in two particular areas: to explore the benefits of surface resistivity testing, and improved test results through performance engineered mixture designs.

Although there was a large sample size of concrete mixtures used for the development of target values for a 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. This leads to the need for test results from field-produced concrete to be used in conjunction with additional laboratory test results and field performance data for further exploration of surface resistivity values typically achieved by NCDOT mixture designs and refinement of specification targets. This also opens the door to exploration into ways to improve durability performance for their mixture designs through mixture proportioning, use of SCMs, and enhanced testing and specification methods per AASHTO PP 84.

Future research projects to support NCDOT's PEM initiatives can further explore the effects of mixture proportioning on test results. Suggested work includes exploration of optimized aggregate gradations to reduce paste content, means to reduce w/cm to support revised prescriptive specification provisions, and increased use of emerging technologies to support durable, sustainable infrastructure including internal curing and increased use of interground limestone (PLCs). Ultimately, target resistivity values should be compared with field performance of North Carolina structures and pavements to facilitate assessment of the targets and further refinement of the specification.

Use of emerging rapid test methods set forth in the PEM initiative and AASHTO PP 84 can greatly improve the long-term durability of NCDOT concrete. Use of state-of-the-art testing methods, such as formation factor testing, workability tests (such as the VKelly and box test) and air void system testing (using the SAM) should allow concrete structures and pavements to be more constructible and offer improved performance over longer lifespans. These alternative design and testing methods should also ultimately encourage producer and contractor innovation, providing additional benefits to both concrete performance and costs for constructing and maintaining NCDOT infrastructure.

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APPENDIX A: SUPPLEMENTAL INFORMATION FOR CHAPTER 3

Version 6.42

Holcim

Material Certification Report

Material:	Portland Cement	Test Period:	14-Sep-2015
Type:	I-II(MH)	To:	15-Sep-2015

Certification

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

General Information

Supplier:	Holcim (US) Inc.	Source Location:	Holly Hill Plant
Address:	2173 Gardner Boulevard Holly Hill, SC 29059		2173 Gardner Boulevard Holly Hill, SC 29059
Telephone:	803-496-2995	Contact:	Scott Poaps
Date Issued:	15-Dec-2015		

The following information is based on average test data during the test period.
The data is typical of cement shipped by Holcim; individual shipments may vary.

Tests Data on ASTM Standard Requirements

Chemical

Item	Limit[A]	Result
SiO_2 (%)	-	20.4
Al_2O_3 (%)	6.0 max	4.8
Fe_2O_3 (%)	6.0 max	3.3
CaO (%)	-	63.8
MgO (%)	6.0 max	1.6
SO_3 (%)	3.0 max[B]	3.1
Loss on Ignition (%)	3.0 max	1.6
Insoluble Residue (%)	0.75 max	0.23
CO_2 (%)	-	1.1
Limestone (%)	5.0 max	2.6
$CaCO_3$ in Limestone (%)	70 min	92
Inorganic Processing Addition (%)	5.0 max	0.0
Potential Phase Compositions[C]:		
C_3S (%)	-	54
C_2S (%)	-	17
C_3A (%)	8 max	7
C_4AF (%)	-	10
$C_3S + 4.75C_3A$ (%)	100 max	87.3

Physical

Item	Limit[A]	Result
Air Content (%)	12 max	6
Blaine Fineness (m^2/kg)	260-430	393
Autoclave Expansion (%) (C151)	0.80 max	0.05
Compressive Strength MPa (psi):		
3 days	10.0 (1450) min	29.0 (4210)
7 days	17.0 (2470) min	34.8 (5040)
Initial Vicat (minutes)	45-375	117
Mortar Bar Expansion (%) (C1038)		0.006
Heat of Hydration: kJ/kg (cal/g)[D] 7 Days (for informational purposes)	-	305 (73)

Tests Data on ASTM Optional Requirements

Chemical

Item	Limit[A]	Result
Equivalent Alkalies (%)	0.60 max	0.53

Physical

Item	Limit[A]	Result

Notes

[A] Dashes in the limit / result columns mean Not Applicable.
[B] It is permissible to exceed the specification limit provided that ASTM C1038 Mortar Bar Expansion does not exceed 0.020 % at 14 days.
[C] Adjusted per Annex A1.6 of ASTM C150 and AASHTO M85.
[D] Test result represents most recent value and is provided for information only. Analysis of Heat of Hydration has been carried out by CTLGroup, Skokie, IL. This data may have been reported on previous mill certificates.

Silo 18
9/14/2015
Grind 257-259

Additional Data

Inorganic Processing Addition Data

Item	Result[A]
Type	-
Amount (%)	-
SiO_2 (%)	-
Al_2O_3 (%)	-
Fe_2O_3 (%)	-
CaO (%)	-
SO_3 (%)	-

Base Cement Phase Composition

Item	Result
C_3S (%)	56
C_2S (%)	16
C_3A (%)	7
C_4AF (%)	10

By Scott Poaps, Quality Manager

Figure A.1: OPC mill report

TEC Services
Testing • Engineering • Consulting

Client: Mr. Jim Simon
Ash Venture LLC
188 Summerfield Court, Suite 201
Roanoke, VA 24019

Date: January 30, 2015
TEC Services Project No: TEC 14-1097
TEC Laboratory No: 14-1090

REPORT OF FLY ASH TESTS

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

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

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

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

The results of our testing indicate that this sample complies with ASTM C618-12a and AASHTO M295-11 specifications for Class F pozzolans.

Respectfully Submitted,
Testing, Engineering & Consulting Services, Inc.

Dean T. Roosa
Senior Laboratory Technician

COMMONWEALTH OF VIRGINIA
BRIAN J. WOLFE
Lic. No. 402053518
1/31/15
PROFESSIONAL ENGINEER

Shawn McCormick
Laboratory Principal

Testing, Engineering & Consulting Services, Inc.
235 Buford Drive | Lawrenceville, GA 30046
770-995-8000 | 770-995-8550 (F) | www.tecservices.com

Figure A.2 Fly ash report

A.S.T.M. C 127 Standard Test Method for Specific Gravity and Absorption of Coarse Aggregate

Date Sampled	Material	Control Number	Bulk - Dry	Bulk - Saturated Surface Dry	Apparent	Absorption	Laboratory	Technician	Comments
		Triangle							
April 4, 2018	# 67	T-1	2.632	2.644	2.663	0.4%	WSC Central	C. Gastiger	Dark Gray Material

Figure A.3 Coarse aggregate specific gravity and absorption report

A.S.T.M. C - 131 Standard Test Method for Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine

Date Sampled	Material	Control Number	Grading	Percent Loss	Laboratory	Technician	Comments
		Triangle					
April 4, 2018	# 67	T-1	B	47	WSC Central	C. Gastiger	Dark Gray Material

Figure A.4 Coarse aggregate LA Abrasion Test report

A.S.T.M. C - 136 Standard Test Method for Sieve Analysis of Fine and Coarse Aggregate

Date Sampled	Material	Control Number	Percent Passing															Soil Mortar				
			2"	1 1/2"	1"	3/4"	1/2"	3/8"	#4	#8	#10	#16	#30	#40	#50	#100	#200	LL	PI	#30	#40	#200
		Triangle																				
April 4, 2018	# 67	T-1			100	98	59	36	5	3												

Figure A.5 Coarse aggregate sieve analysis report

Sample Status: Complete

NORTH CAROLINA DEPARTMENT OF TRANSPORTATION
MATERIALS AND TESTS UNIT
1801 BLUE RIDGE RD. RALEIGH, N.C. 27607
01/31/2018
Fine Aggregate Test Yearly

Hicams No.: 889572
Contract No.:
County: Harnett
Date Sampled: 12/01/2017
Sampled By: Christian, Guy C
Sampled From: Stockpile - 1
Contractor:
Prod./Suppl.: G.S. Materials
Facility: Hall Pit - Lemon Springs
Material: Yearly Quality Check for Fine Aggregate (Strength, Soundness, etc.)

T.I.P. No.:
Field ID: FA122-1
Engineer:
Received: 12/13/2017

Work Order No.:
P.O./Other No.:
Reported: 01/29/2018
Test Category: Verification
Represented Qty.: 10000.000 TON

Test No.: AASHTO T112, T19, T27, T104, T71, T84
QA Indicator: N
Location of Source: Stockpile
Property Owner:

SIEVE ANALYSIS

Sieve Size	Percent Passing
3/8"	100
# 4	100
# 8	98
#16	86
# 30	41
# 40	
# 50	9
# 80	
# 100	2
# 200	1.0
Fineness Modulus	2.65
Deleterious Substance	0.4

STRUCTURAL STRENGTH

Compression Test on 2 inch Cubes

Strength Ratio: 3-Day: 112.7 %
7 Day: 118.7 %

Color: 2
Sp.Grav.: 2.61
Absorp.: 0.40 %
Soundness: 3.1 % Loss

Unit Weight

Solid 163.1 lbs/ft3
Dry & Rodded 101.6 lbs/ft3

Comments:

Chun Kun Su P.E.
GeoMaterials Workgroup Supervisor

cc: G.S. Materials

Figure A.6 Fine aggregate report

Table A.1 Calculated coarse aggregate properties

Property	Sample 1	Sample 2	Sample 3	Average
Bulk specific gravity (bulk SG)	2.57	2.57	2.49	2.54
Bulk specific gravity (saturated surface dry)	2.61	2.62	2.56	2.60
Apparent specific gravity (apparent SG)	2.67	2.69	2.69	2.68
Absorption (%)	1.49	1.63	2.95	2.02

Table A.2 Calculated fine aggregate properties

Property	Sample 1	Sample 2	Sample 3	Average
Bulk specific gravity (bulk SG)	2.58	2.54	2.58	2.56
Bulk specific gravity (saturated surface dry)	2.61	2.59	2.61	2.60
Apparent specific gravity (apparent SG)	2.66	2.67	2.68	2.67
Absorption (%)	1.26	1.91	1.52	1.56

APPENDIX B: SUPPLEMENTAL INFORMATION FOR CHAPTER 4

Table B.1 Compiled 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

Table B.2 Compiled 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.3 Compiled 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.4 Compiled 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.5 Compiled 28 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.6 Compiled 28 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.7 Compiled 28 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

APPENDIX C: SUPPLEMENTAL INFORMATION FOR 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 unreinforce d	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 .	-	-	-	Ternary blend - extremely aggressive environment	> 29	28 days

Implemented AASHTO T 358 in January 2017	-	-	-	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 criteria where specified	-	-	-	Superstructures and substructures	> 24	28 days
	-	-	-	Footings, piles, drilled shafts, underground applications, sign bases, etc.	> 14	28 days
	-	-	-	Pavements, sidewalks, gutters, curbs, barriers, headwalls, drainage	> 16.5	28 days

				elements, pipe inverts, maintenance repair		
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 MC²	≤ 3000	28 days	Mass Concrete: Class MC²	≥ 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 MC²	≤ 1500	28 day accelerated cure	Mass concrete: Class MC²	≥ 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) =	Must have "low to negligible risk" according to AASHTO T 358	

				other structural elements)		
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