OPTIMIZED AGGREGATE GRADATION CONCRETE MIXTURES WITH CEMENTITOUS MATERIAL REDUCTION

by

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ABSTRACT

PETER THEILGARD. Optimized Aggregate Gradation Concrete Mixtures with Cementitious Material Reduction. (Under the direction of DR. BRETT TEMPEST)

Optimized aggregate gradation concrete mixtures with reduced cementitious materials are emerging in concrete mixture design in an effort to reduce costs and CO_2 emissions while improving the concrete's durability characteristics. Optimizing the aggregates enables lowered cementitious content and results in reduced permeability and shrinkage while also reducing costs. The Tarantula Curve, named by Oklahoma State University, describes a distribution of particle sizes that maximizes the packing of the aggregates while maintaining workability. The goal of this research was to evaluate the Tarantula Curve's optimized aggregate gradation in concrete mixtures with a ten percent reduction in cementitious materials, using materials typical for use in concrete bridge and pavement construction specified by the North Carolina Department of Transportation (NCDOT).

Twenty-one different optimized aggregate gradation concrete mixtures representative of structural and pavement type mixtures used by the NCDOT were produced and compared to twenty-one similar companion mixtures with non-optimized aggregate gradations. Using materials sourced from the same quarries and suppliers, the gradations were optimized using the Tarantula Curve. The fresh properties and hardened mechanical and durability properties were evaluated by the research team. The test set included variations in the water-cement ratio, cementitious material content, and fly ash replacement percentage. The mixtures were proportioned to encompass a range of designs typical of concretes used in structural and pavement construction.

Both non-optimized and optimized aggregate gradation mixtures demonstrated consistent trends with increased mechanical performance as the w/cm ratio decreased. Optimized aggregate gradation mixtures did not show a significant decrease in mechanical when compared to the non-optimized aggregate gradation companion mixtures. Preliminary measurements of surface resistivity of the optimized aggregate gradation mixtures indicated a potential decrease in expected durability. However, additional experimentation is required to determine if the results of the electrical resistivity tests can be interpreted the same for optimized and non-optimized aggregate gradation mixtures and whether lower resistance of optimized mixtures is linked to lesser durability.

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I would like to thank my family and friends that have supported my decision to go back to school to earn my degrees in Civil Engineering. Your endless support, advice, and perspective on life have shaped who I am today more than you may know.

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LIST OF ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
AEA	Air Entraining Admixture
ACI	American Concrete Institute
ASCE	American Society for Civil Engineers
ASTM	American Society for Testing and Materials
cf	cubic feet
cwt	hundred weight of cement
cy	cubic yard
FHWA	Federal Highway Administration
ft	foot
g	gram
gal	gallon
hr	hour
in	inch
ITZ	interfacial transition zone
lb	pound
m	meter

mL	milliliter
mm	millimeter
MOE	modulus of elasticity
MOR	modulus of rupture
OPC	ordinary portland cement
oz	ounce
pcy	pounds per cubic yard
pcf	pounds per cubic foot
psi	pounds per square inch
RCPT	Rapid Chloride Permeability Test
SAM	Super Air Meter
SCM	supplementary cementitious material
w/cm	water-cement
WRA	Water Reducing Admixture
yd	yard

CHAPTER 1: INTRODUCTION

1.1 Background

The American Society of Civil Engineers (ASCE) 2021 infrastructure report card reported that 42 percent of bridges in the United States are older than 50 years old, 7.5 percent of bridges are structurally deficient, and 43 percent of roadways are considered to be in poor or mediocre condition (ASCE 2021). Structurally deficient bridges require a substantial investment in the form of replacement or rehabilitation and may disrupt surface transportation through closure for repair or weight restrictions. Estimates of required annual spending to meet bridge rehabilitation needs in the US range from $14.4 billion to $22.7 billion (ASCE 2021). ASCE recommends improving both bridges and roadways by selecting methods or materials that contribute to climate resilience (ASCE 2021). Climate resilience is used to describe the ability of infrastructure, such as bridges or roads, to withstand, respond to, and recover from disruptions caused by climate conditions (Mullan 2018).

Recent legislation, such as Moving Ahead for Progress in the 21st Century (MAP-21) and Fixing America's Surface Transportation (FAST), place an emphasis on improved performance of highways and bridges and have led the FHWA to develop a Performance Engineered Mixture (PEM) initiative (FHWA 2019). The American Association of State Highway and Transportation Officials (AASHTO) developed the provisional document PP 84 "Standard Practice for Developing Performance Engineered Concrete Pavement Mixture" (AASHTO 2021). In contrast to traditional methods of designing and accepting concrete primarily on strength, this practice aims to address concrete durability through prescriptive and performance specifications for strength, cracking, freeze-thaw resistance, aggregate durability, and permeability (FHWA 2019). Prescriptive specifications are the traditional method for qualifying concrete mixture that are employed by most highway departments. They strictly outline details of materials to be used in concrete mixtures, such as the quantity of cement, regardless of the impacts to characteristics beyond compressive strength and slump. Performance specifications target the delivery of desirable qualities of a concrete mixture and include standard test methods to meet acceptance criteria (NRMCA 2005). AASHTO PP 84 was initially intended to provide guidance for concrete pavement specifications, but it can be applied to other concrete applications as well.

Durability is a key characteristic of concrete mixtures that defines its susceptibility to deterioration during service. Concrete durability is often determined by the mixture's permeability properties, resistance to freeze-thaw cracking, and resistance to slab warping and cracking due to shrinkage. AASHTO PP 84 adds a robust QA/QC program to concrete construction in order to reduce the risks posed by these durability challenges, and in turn, reduce the lifecycle costs associated with maintaining the structure or roadway. One prescriptive method in AASHTO PP 84, limits the paste volume in the concrete mixtures to a maximum of 25 percent to prevent slab warping and shrinkage cracking in place of a volumetric shrinkage test. Through the PEM initiative, the NCDOT can benefit from the potential economic advantages of paste reduction, as well as potential construction efficiencies realized from preventing plastic shrinkage, as it will prevent costs and time associated with replacing sections developing plastic shrinkage.

1.2 Optimized Aggregate Gradations

The goal of optimized aggregate gradation concrete mixtures is to reduce the volume of paste required to fill aggregate voids by using additional aggregate to fill the voids instead (Shilstone 1990). Until the 1990s, the total gradation of the aggregate used in a concrete mixture was rarely considered. Shilstone developed one of the first methods in controlling a mixture aggregate gradation through the Coarseness Factor Chart, which focused on reducing segregation in mixtures (Shilstone 1990). Additional methods, such as the Power 45 chart and the Band 8-18 gradation methods, were developed to help control concrete mixture aggregate gradation and improve the concrete mixture workability. These methods are further discussed in Chapter 2 of this thesis.

The Tarantula Curve was developed at Oklahoma State University for the Oklahoma Department of Transportation to produce concrete mixtures that are more sustainable, more durable, and more economic through paste reduction without sacrificing workability and finishability (Cook et al. 2013). Limits were created after studying the effects that varying the amounts of aggregates retained on each sieve had on the concrete mixture's workability, this process is discussed in more detail in Chapter 2 (Cook et al. 2013). Figure 1.1 shows the recommended percent retained on each sieve for the Tarantula Curve, and the general impact of the relative aggregate sizes on placement and finishability.

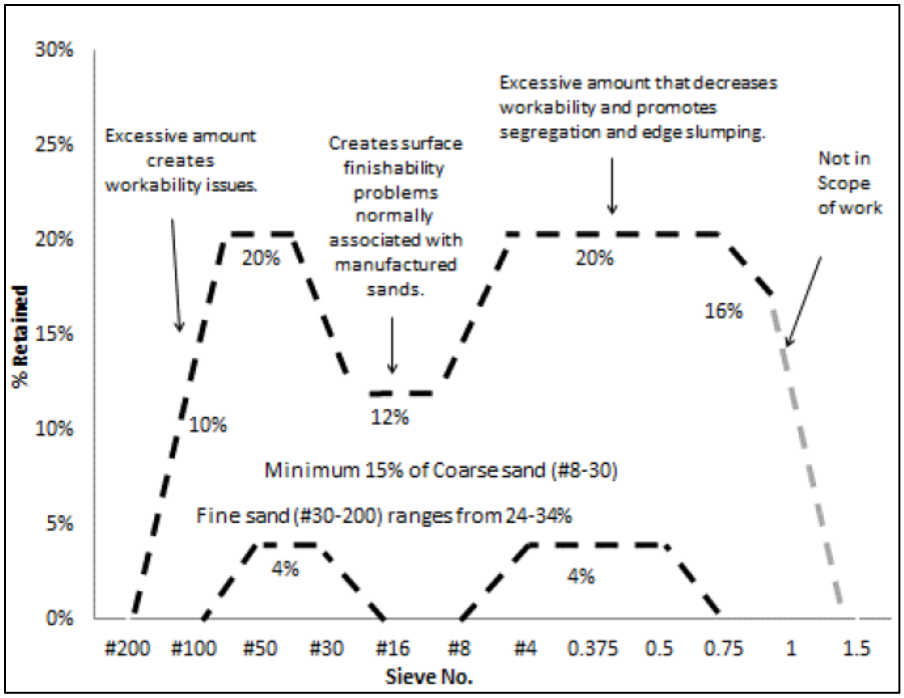

Figure 1.1: Tarantula Curve recommended percent retained

The Tarantula Curve has been used by independent contractors that had no prior knowledge of the Tarantula Curve. These contractors were successfully able to refine mixtures to meet the limits defined in the Tarantula Curve in Minnesota, Iowa, North Dakota, and South Africa (FHWA 2015). The NCDOT has recognized the potential economic and environmental benefits of utilizing optimized aggregate gradations, such as the Tarantula Curve, in an effort to meet prescriptive specifications in AASHTO PP 84 that limit the paste in a concrete mixture to 25 percent.

1.3 Objective and Scope

The objective of this research was to gain a better understanding of the implementation of the Tarantula Curve for use in optimized aggregate gradation concrete mixtures with reduced paste in bridges and pavements using typical materials and proportions used in North Carolina. Using 21 bridge and pavement mixtures previously designed, batched, and tested by the research team as a preliminary data set, "twin" concrete mixtures were created using the Tarantula Curve to optimize the aggregate gradation of concrete mixtures with a uniform paste reduction.

Tests were performed on fresh and hardened concrete to determine mechanical and durability properties of both sets of mixtures. Slump, air content, and Super Air Meter (SAM) testing was done on fresh concrete to establish typical workability and air content characteristics. Compressive strength, modulus of elasticity and Poisson's ratio, and modulus of rupture tests were performed on hardened concrete to determine mechanical properties of the optimized and non-optimized aggregate gradation mixtures. Surface resistivity, chloride permeability, volumetric shrinkage, and the formation factor determined using the "bucket test" method to determine the properties associated with the durability of optimized and non-optimized aggregate gradation mixtures. Data analysis was then performed to understand the influence of optimized aggregate gradation concrete mixtures and non-optimized aggregate gradation concrete mixtures on several mixture design and proportioning characteristics (water to cement ratio (w/cm), cementitious content, and fly ash replacement rate).

1.4 Contents of Thesis

This thesis was written to investigate the potential benefits of concrete mixtures that have an aggregate gradation optimized by using the Tarantula Curve. Chapter 2 of this thesis contains a Literature Review that details different aspects of concrete mixtures for highway applications, introduces AASHTO PP 84, discusses optimized aggregate gradation methods, and summarizes the Tarantula Curve. Chapter 3 of this thesis contains the methodology, including details on how the concrete mixtures aggregate gradations were optimized with the Tarantula Curve, the materials used in this research study, and the tests that were conducted. Chapter 4 of this thesis contains all collected test results and provides an analysis of mechanical and durability properties of optimized aggregate gradation concrete mixtures when compared to companion non-optimized aggregate gradation concrete mixtures. Chapter 5 of this thesis contains conclusions on how optimized aggregate gradation concrete mixtures impacted mechanical and durability properties and provides recommendations for future work.

CHAPTER 2: LITERATURE REVIEW

2.1 Concrete Mixtures for Highway Applications

Concrete is used for highway pavements and bridges across the United States and all over the world. Concrete is stronger and stiffer than asphalt, and therefore concrete pavements create a more durable surface that can withstand loads over a longer life cycle with less maintenance (Kosmatka et al. 2011). Due to the lower lifecycle costs and durability associated with concrete pavements, it is not only used on highway roadways, but is often selected for use on residential streets, city streets, parking lots, intersections, and airstrips as well.

More than 70 percent of bridges in the United States have one or more elements constructed from concrete. These bridges are located in many different environments and geographical regions (Kosmatka et al. 2011). Because of the wide range of exposure conditions and climates experienced by bridges, as well as the loads they carry and the critical role they play in our roadway infrastructure, concrete mixtures exhibiting higher strengths and enhanced durability characteristics are often used. These mixtures typically ensure public safety and reduced maintenance and rehabilitation needs over the designed lifespan of the bridge.

The quality of a concrete mixture's cementitious materials, aggregates, and the bond created between the materials directly impacts the quality of the concrete (Kosmatka et al. 2011). Specifications from multiple agencies have been written to help mitigate risks of choosing inadequate materials such as ASTM, AASHTO, and ACI. State agencies choose from these specifications according to their needs for particular applications. Mixture proportioning is the process of selecting the materials correct proportions to be used in a concrete mixture to meet the requirements (air content, workability, compressive strength, and other characteristics) established by the concrete design (Mehta and Monteiro 2014). Concrete mixture design and proportioning is typically performed with the intent of simultaneously meeting several goals, including strength, durability, economy, and sustainability, described in the following sections.

2.1.1 Strength

The strength of a material is defined as “the ability to resist stress without failure” (Mehta and Monteiro 2014). Historically, concrete mixtures have been proportioned based upon a stakeholder’s previous experience, using available materials that were subject to extensive variability (Dolen 2008). Development of mixture design protocols and material quality control procedures have resulted in great improvements in concrete quality over the past century. In 1918, Duff Abrams established a relationship between the water-cement ratio (w/cm) and compressive strength, a characteristic of concrete mixtures that is still critical to the control of quality today (Abrams 1918). Current mixture designs are more technical and practical than in the early 1900s and incorporate specifications to maximize the strength and durability of the mixture. The most widely used mixture proportioning standard is ACI 211.1, “Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete” (ACI 2002).

The quality of a concrete mixture has traditionally been evaluated through its compressive strength. Itis imperative that a concrete mixture used for a structure or pavement have adequate strength to meet the design criteria (Ley et al. 2017). However, it is recognized that in aggressive environments, concrete strength is not as reliable an indicator of durability as permeability and other durability performance characteristics (Armaghani et al. 1992). Most DOT’s have specifications that require more cementitious material than would be required to meet design strength. Excess cement quantities can result in adverse durability effects (Taylor et al. 2019). Although responsible for strength gain, the cementitious content of a concrete mixture is also responsible for other undesirable characteristics, including heat generation during curing and shrinkage. Additionally, higher cement paste renders the concrete more permeable to water carrying aggressive agents such as chlorides and sulfates (Mehta and Montiero 2014).

To reduce the cementitious material required in concrete mixtures, some state agencies are moving towards using optimized aggregate gradations that reduce the paste content of a mixture by improving the particle packing. These mixtures have been shown to provide adequate strength, workability, and can, in some cases, improve the mixture's durability performance. There are many different computational mixture proportioning methods currently available, all aimed at allowing the designer to determine the best combination of materials to maximize the particle packing density and reducing the voids (Jones 2002). The most commonly used methods to optimize aggregate gradations in concrete mixtures will be discussed in depth in Section 2.3.

Water is required in concrete mixtures to hydrate the cement. A challenge facing optimized mixture designs is how to reduce the cementitious material while holding the w/cm ratio constant without adversely impacting the workability of the mixture. To be able to achieve the low w/cm ratios desired for strength and durability properties, the use of water reducing admixtures (WRA) is often required. Control of the water content of concrete mixtures is critical to achieving both the desired mechanical properties and durability performance (Taylor et al. 2019).

2.1.2 Durability

While concrete mixture designs must meet their prescriptive requirements for characteristics such as required compressive strength, the concrete's durability performance ultimately determines the potential life span of the concrete structure or pavement. The Transportation Research Board's (TRB) Committee on the Durability of Concrete recognizes that durability is not a singular material characteristic of concrete, but rather “a series of properties required for the particular environment to which concrete will be exposed to during its service life” (Taylor et al 2013). In its circular, the committee states that the primary concrete characteristics that define durability are: 1) resistance to freeze thaw cycles, 2) compressive strength of at least 4,000 psi, 3) resistance to effects of alkali-silica reaction (ASR) and/or sulfate attack, 4) resistance from damage by abrasion, and 5) resistance from damage from steel corrosion (Taylor et al. 2013). These criteria are similar to the ones identified by the American Concrete Institute (ACI) in 201.2R-16 Guide to Durable Concrete, which identifies mass transport, freeze thaw cycles, alkali-aggregate reaction, sulfate attack, chemical attack, corrosion of metals or materials embedded in the concrete, and abrasion as major contributing factors that can limit a concrete’s lifespan by impacting it’s durability (ACI 2016). Many of these factors can be attributed to the climate in which the concrete is in service, exposure conditions such as moisture and aggressive agents, and how the environment will interact with the material.

ACI 318’s Chapter 19 is designated to address durability concerns for structural concrete, and how the risks can be mitigated with required maximum w/cm ratios, compressive strength, and air content (ACI 2019). Similarly, AASHTO’s PP 84 focuses on concrete pavements, allowing for state highway departments to select criteria that they know from professional experience to have an impact on the durability of concrete in their state. Although initially developed to support use in concrete pavement application, the approach prescribed by AASHTO PP 84 and many provisions can be adapted to structural concrete mixtures as well (Cavalline et al. 2020).

Producing durable concrete that provides adequate performance in a given service environment requires consideration of a variety of material and mixture characteristics to minimize the risk of chemical attacks by deleterious substances in the materials used. Aggregates typically comprise up to 60-75 percent by volume of a concrete mixture and must possess the qualities required to withstand the loading and environmental exposure the concrete mixture will face in its service life (Kosmatka et al., 2011). Studies have shown that using an optimized aggregate gradation can allow for an increase in aggregate content while reducing the paste content, minimizing chemical reactions, and increasing dimensional stability (Taylor et al., 2013). Aggregates selected for a concrete mixture should be non-reactive, meaning they are resistant to alkali-silica reaction (ASR). Typically, aggregates are prequalified for use, using testing standards to determine the potential reactivity through ASTM and AASHTO standardized methods, such as ASTM C1293, "Standard Test Method for Determination of Length Change of Concrete due to Alkali-Silica Reaction (Concrete Prism Test)," and ASTM C1260 (AASHTO T 303), "Potential Alkali-Reactivity of Aggregates (Mortar-Bar Method)." ASTM C1293 is a more accurate but longer (one year) test, while ASTM C1260 is an accelerated (16 day test) but less accurate method (Kosmatka et al. 2011).

Since most aggregates used in concrete mixtures are dense and virtually impermeable, water and other liquids move through the hardened concrete's paste. The area within a concrete mixture where the aggregates and cementitious materials form a bond is called the interfacial transition zone (ITZ). The ITZ is generally considered as the strength limiting factor in the concrete and has the greatest influence on the elastic modulus of the concrete. It is a direct contributing cause to damage due to permeability-related attacks because of the presence of microcracks within the ITZ's structure (Mehta and Monteiro 2014). In addition to the type and amount of cementitious materials used in a mixture, several factors have an influence on the ITZ's properties. For example, the thickness of the ITZ increases with increasing w/c ratio and aggregate/cement ratio (Crumbie 1994) and longer mixing times may create more a pronounced ITZ (Katz et al. 1998, Leeman et al., 2005).

It is well known and accepted fact that concrete mixtures with lower w/cm ratios are less permeable and more durable than those with higher w/cm ratios. However, the cement type used affects concrete durability, as well as the type and amount of supplementary cementitious (SCM) materials used. To improve a mixture's durability, the cement type used should be resistant to sulfate attack while also having an adequate strength to resist damage due to abrasion (Taylor et al., 2013). SCMs such as fly ash, slag, metakaolin, or silica fume, may be used in a concrete mixture as a replacement for a certain percentage of cement. Some SCMs are used to increase economy, but most SCMs can also enhance a concrete mixture's durability properties if proportioned with the other materials in the mixture. One drawback of some commonly used SCMs, such as fly ash and slag, are that they may reduce the early strength of the concrete. Longer-term strength is often similar to or improved over mixtures without SCMs, and these SCMs provide the benefits of lower heat of hydration, a denser paste microstructure resistant to moisture and aggressive chemicals, and reduced shrinkage (Taylor et al., 2013).

Chemical admixtures, such as WRAs and air entraining admixtures (AEA), can also help to achieve desired properties in fresh and hardened concrete (Kosmatka et al. 2011). AEAs are used to entrain a matrix of air bubbles in the concrete mixture, helping to resist risks of damage due to freeze-thaw cycles. WRAs are used to reduce the w/cm ratio in a concrete mixture while maintaining the workability. Additionally, they can help to influence the rates of cement hydration and early strength development of concrete mixtures (Mehta and Monteiro, 2006). By reducing the w/cm ratio of a mixture through the use of WRAs, there is typically an increase in compressive strength development, and a decrease in susceptibility to chloride ion and sulfate penetration, although there is a possible increase in drying shrinkage (Kosmatka et al. 2011). WRAs function by influencing the electrostatic and steric repulsive forces of the cement by giving the particles a slight negative charge to repel one another and releasing the water reducing the viscosity of the concrete (Kosmatka et al. 2011).

2.1.3 Economy

The American Society of Civil Engineering's (ASCE) 2021 report on American infrastructure, reports that 7.5 percent of highway bridges were designated as structurally deficient, and the total percentage of bridge deck area that is designated as structurally deficient is 5.5 percent (ASCE 2021). While these numbers are an improvement relative to previous report cards, which designated 12.1 percent of highway bridges as structurally deficient in 2009 and 6.3 percent of total percentage of bridge deck area as structurally deficient in 2016, the annual rate of reduction of structurally deficient bridges has reduced to 0.1 percent with an increasing number of bridges moving from good-to-fair condition to "poor" condition (ASCE 2021). Repair, rehabilitation, maintenance, and replacement of these bridges, as well as our highway pavements, will require a significant investment in materials.

Our infrastructure is aging, and resources to repair, rehabilitate, and maintain its components are limited. A 2010 report to the Federal Highway Administration (FHWA), estimated that one 12-inch-thick lane-mile of concrete can require about 4,800 tons of material with 10 – 14 percent by volume being cementitious materials (Tayabji et al. 2010). Economic considerations have, and will continue to play, a role in development and use of concrete mixture designs for structures and pavements.

Typically, aggregates are the lowest cost component of a concrete mixture. The cost of cement, cementitious materials, and admixtures is often significantly higher. Using an estimate for cement and fly ash costs obtained from a paving contractor performing work for NCDOT, an assumed concrete depth of 11 inches, lane width of 11 feet, and an additional 15 percent for waste, reducing the cementitious materials in a concrete mixture by 10 percent would reduce the cost per lane mile by $8,980 - $10,567 depending on quantities of cement and fly ash used. These savings are displayed in Table 2.1.

Table 2.1: Projected cost savings with 10% reduction in cementitious material

Original Cement Content (pcy)	Reduced 10% Cement Content (pcy)	Fly Ash Replacement (%)	Cost Savings ($/CY)	Cost Savings per Lane Mile
700	630	0	$ 4.66	$ 10,567
560	504	20	$ 4.19	$ 9,501
650	585	0	$ 4.32	$ 9,796
520	468	20	$ 3.89	$ 8,821
600	540	0	$ 3.99	$ 9,048
480	432	20	$ 3.59	$ 8,141
420	378	30	$ 3.96	$ 8,980

Producing concrete that is economic is also sustainable, since using optimized concrete mixtures reduces the amount of Portland cement which reduces the amount of CO_2 released. On average, the cement industry produces 0.92 tons of CO_2 for every ton of cement produced (Kosmatka et al. 2011). Using 0.92 pounds of CO_2 emitted per pound of cement produced, an assumed concrete depth of 11 inches, lane width of 11 feet, and an additional 15 percent for waste, CO_2 emissions associated with concrete mixtures could be reduced by 87,621 – 146,035 pounds per lane mile by reducing the cement used by 10 percent, depending on the mixture characteristics, shown in Table 2.2.

Table 2.2: CO_2 emission reduction per lane mile

Original Cement Content (pcy)	Reduced 10% Cement Content (pcy)	Fly Ash Replacement (%)	CO_2 Emission Reduction (lbs/cy)	CO_2 Emission Reduction per Lane Mile (lbs)
700	630	0	64.4	146,035
560	504	20	51.5	116,828
650	585	0	59.8	135,604
520	468	20	47.8	108,483
600	540	0	55.2	125,173
480	432	20	44.2	100,139
420	378	30	38.6	87,621

2.1.4 Sustainability

ASCE defines sustainability as "a set of economic, environmental, and social conditions in which all of society has the capacity and opportunity to maintain and improve its quality of life without degrading the quantity, quality, or the availability of economic, environmental, and social resources" (ASCE 2021). Concrete mixtures have a public perception that they are major contributor to the amount of carbon dioxide in the atmosphere. It was estimated in 2007, 1.5 percent of carbon dioxide (CO_2) generated in the United States resulted from the manufacture of portland cement, and that portland cement is responsible for 90 – 95 percent of CO_2 emission associated with concrete (Taylor and Van Dam 2009). However, advancements in cement production have greatly decreased these impacts (Van Dam et al. 2012).

Using SCMs such as fly ash or silica fume while utilizing optimized aggregate gradations can reduce the traditionally required amount of Portland cement in concrete mixtures, potentially improving fresh concrete performance and improving durability characteristics. Additionally, these materials are industrial byproducts, and beneficially reusing them in concrete saves landfill space and can provide other sustainability benefits associated with energy use, hauling, and water quality (Van Dam et al. 2012). Use of optimized aggregate gradations can also improve the sustainability of a mixture. Studies have shown the volume of paste in a concrete mixture is correlated with plastic shrinkage and cracking (Shaeles and Hover 1988, Darwin et al. 2004). By implementing the use of high quality SCMs in optimized aggregate gradation mixtures, the amount of portland cement used can be reduced, potentially improving permeability of the concrete while reducing plastic shrinkage (Van dam et al. 2012).

There are emerging technologies in the concrete industry aimed at improving the sustainability of concrete mixtures. One of these technologies is the use of high-volume SCM/portland limestone cement paving mixtures to potentially reduce the greenhouse gas emissions significantly (Van Dam et al. 2012). Portland limestone cement (PLC) is a cementitious material that allows up to 15 percent limestone replacement of portland cement clinker per AASHTO M240 (AASHTO 2020). Using a PLC cement at that replacement level can reduce the CO_2 emissions by roughly 10 percent (Van Dam et al. 2012). Photocatalytic cements can be used to degrade pollutants like nitrogen oxides by as much as 60 percent (Van Dam et al. 2012). Low carbon and carbon sequestering cementitious systems, which sequester carbon dioxide as they harden, lowering the carbon footprint (Van Dam et al. 2012). Photocatalytic cement and carbon sequestering cementitious systems have promising results but require additional research to become viable options for large scale operations (Van Dam et al. 2012).

2.2 AASHTO PP84

AASHTO PP 84, “Standard Practice for Developing Performance Engineered Concrete Pavement Mixtures” is a guidance document that allows DOT’s and other state transportation agencies to adopt tests and quality control measures described in the provision to best fit their needs. AASHTO PP 84 focuses on six primary characteristics that influence concrete performance. These characteristics are strength, susceptibility to slab warping and shrinkage cracking, freeze-thaw durability, transport properties, requirements for aggregate stability, and workability (AASHTO 2020). Depending on the exposure conditions and climate the concrete is likely to be subjected to, AASHTO PP 84 has recommended tests and performance targets for each characteristic that should be considered throughout the concrete's lifespan, from mixture qualification testing to acceptance testing during and after construction.

2.2.1 Overview

AASHTO PP 84-20, outlines existing, alternative, and emerging methods to evaluate concrete performance for concrete pavement life in the range of 30 years, providing states DOTs with the ability to evaluate and choose which methods will work best for their climates and uses (AASHTO 2020). The aim of the document is to assist state agencies in moving from prescriptive specifications for concrete mixtures to performance specifications by implementing tests and quality assurance control measures to better understand the quality of a mixture for the specific climate and/or failure mechanisms it will be exposed to throughout its service life. Performance specification provisions should help ensure satisfactory performance of concrete in the fresh and hardened state, and therefore support implementation of Performance Engineered Mixtures (PEMs) (Cackler et al. 2017).The movement towards PEMs does not mean that state highway departments will abandon the specification provisions that they have been using. In fact, many of the requirements of performance engineered concrete outlined in AASHTO PP 84 are similar to requirements already in place with the NCDOT and are outlined in Table 2.3 (AASHTO 2020, NCDOT 2018).

Table 2.3: Prescriptive testing requirements

Test	AASHTO PP 84 Requirement	NCDOT Requirement
Flexural Strength	600 psi at 28 days	650 psi at 28 days
Compressive Strength	4,000 psi at 28 days	4,500 psi at 28 days
Air Content	5 – 8%	5.0% ± 1.5%

The goal of moving towards performance specifications is to support agencies in amending and/or revising their specifications to help meet their design goals, as well as their durability and sustainability performance targets.

2.2.2 Strength Provisions

AASHTO PP 84 section 6.3.1 recommends that concrete mixtures have a flexural design strength of 600 psi at 28 days using AASHTO T 97 "Standard Method of Test for Flexural Strength of Concrete" (AASHTO 2020). Section 6.3.2 states that concrete mixtures should achieve compressive strength of 4,000 psi at 28 days using AASHTO T 22 "Standard Method of Test for Compressive Strength of Cylindrical Concrete Specimens" (AASHTO 2020). Both sections 6.3.1 and 6.3.2 state that agencies may consider flexural strength and compressive strength either alone or in combination, and acknowledges that it is not uncommon for agencies to have different target values at different ages (AASHTO 2020).

2.2.3 Durability Provisions

Section 6.4 of PP 84 "Susceptibility to Slab Warping and Shrinkage Cracking," identifies the volume of paste in a mixture, unrestrained length change, and cracking potential as factors to be controlled to prevent damage from shrinkage or slab warping, recommending that only one be selected for project QC purposes (AASHTO 2020). One prescriptive approach is recommended in section 6.4. Section 6.4.1 states that if slab warping or drying shrinkage cracking is a concern, a maximum paste content of 25 percent should be allowed in a mixture. Alternatively, performance tests include measurement of the unrestrained length change (AASHTO 2020). AASHTO T 160 (harmonized with ASTM C157) is the test standard to be used for testing the unrestrained length change. In this test method, three 3-in x 3-in x 11-in specimans are tested and averaged, with 420 microstrain at 28 days as a target (AASHTO 2020).

Section 6.4.2 provides performance specifications for shrinkage cracking caused by water-related volume change if cracking is a concern (AASHTO 2020). Section 6.4.2.1 states that estimated cracking potential can be determined using a Restrained Ring Test, and that cracking tendency of restrained concrete can be estimated using 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.” Section 6.4.2.1.1 states cracking tendency estimated using AASHTO T 334 is to be tested without cracking before 180 days (AASHTO 2020). Section 6.4.2.1.2 states cracking tendency estimated using T 363 should have an average stress less than 60 percent of the splitting tensile strength when tested in the dual ring at the standard relative humidity and temperature in T 363 for 7 days (AASHTO 2020). Both methods are based on limits set for bridge decks and are considered conservative for pavement applications (AASHTO 2020). Cracking potential may also be estimated using numerical models as detailed in section 6.4.2.2, which states that the unrestrained volume change (determined using T 160) at 91 days should result in a probability of cracking of less than 5, 20, or 50 percent depending on the mixture design and application (AASHTO 2020).

The influence of freeze-thaw cycles on a concrete mixture’s durability is impacted by w/cm ratio, fresh air content, entrained air void system characteristics, time and duration of critical saturation, and presence of deicing solutions in joints (Li et al. 2012). Section 6.5.2 of AASHTO PP 84 provides recommended prescriptive specifications for w/cm ratio, air content, and Super Air Meter (SAM) to qualify concrete mixtures as freeze-thaw resistant (AASHTO 2020). Section 6.5.1.1 states the w/cm ratio of a mixture should not exceed 0.45 (AASHTO 2020). Section 6.5.1.2 states the air content should be within 5 to 8 percent, as determined using T 152 "Standard Method of Test for Air Content of Freshly Mixed Concrete by the Pressure Method," T 196, "Standard Method of Test for Air Content of Freshly Mixed Concrete by the Volumetric Method," or TP 118, "Standard Method of Test for Characterization of the Air-Void System of Freshly Mixed Concrete by the Sequential Pressure Method," (AASHTO 2020). Section 6.5.1.3 states air content that is 4 percent or greater as determined in accordance with specifications listed in section 6.5.1.2, must have a SAM number less than or equal to 0.20 determined using TP 118 (AASHTO 2020). Section 7.1.2 provides construction acceptance requirements, stating a SAM number of 0.25 or lower may be accepted, a SAM number between 0.25 and 0.30 will require the concrete mixture to be modified, and a SAM number 0.30 and above will result in rejection (AASHTO 2020). Performance specifications for freeze-thaw durability are found in section 6.5.2.1, detailing "the properties of a mixture required to reach a critical saturation at 30 years," (AASHTO 2020). Section 6.5.3 details two prescriptive specifications for reducing joint damage due to deicing chemicals when $CaCl_2$ or $MgCl_2$ is used; either using a SCM to replace the cement with a mass of at least 30 percent or applying a topical sealer in accordance with M 224, "Standard Specification for Use of Protective Sealers for Portland Cement Concrete," (AASHTO 2020).

Transport properties refer to the ability of ions and fluids to move through the material, potentially damaging the concrete and/or steel embedded in it. AASHTO PP 84 recommends that the w/cm ratio, formation factor, and the penetration of iconic species in concrete be used as key indicators of transport properties influencing a concrete mixtures durability (AASHTO 2020). Prescriptive specifications for transport properties are in section 6.6.1. Section 6.1.1 states that the w/cm ratio shall be less than 0.50 if the concrete mixture is not subjected to freeze-thaw cycles or deicing applications, and less than 0.45 if the concrete mixture is subjected to freeze-thaw cycles or deicing applications (AASHTO 2020). The performance specification for a transport related property states that the formation factor will be determined and used in the determination of a service life (AASHTO 2020). The formation factor is determined by dividing the resistivity at 91 days according to TP 119-15, "Standard Method of Test for Electrical Resistivity of a Concrete Cylinder Tested in a Uniaxial Resistance Test," with conditioning option A by a pore solution resistivity of 0.127 k **Ω**. Section 6.6.1.2 states that the formation factor (F factor) must be greater than 500 if the concrete mixture is not subjected to freeze-thaw cycles or deicing applications and must be greater than 1000 if the concrete mixture is subjected to freeze-thaw cycles or deicing applications (AASHTO 2020).

These specifications have been provided in Table 2.4 and show both prescriptive and performance specifications of AASHTO PP 84, with performance specifications highlighted in green.

Table 2.4: AASHTO PP 84 prescriptive and performance requirements

AASHTO PP 84 Section	Provision	Specification
6.3	Flexural Strength	600 psi at 28 days
	Compressive Strength	4,000 psi at 28 days
6.4	Susceptibility to Slab Warping and Shrinkage Cracking (choose one)	Volume of paste shall not exceed 25%
		Unrestrained length change less than 420 microstrain at 28 days
		Estimated cracking tendency estimated using T 334
		Estimated cracking tendency estimated using T 363
6.5	Freeze-Thaw Durability	Maximum w/cm ratio of 0.45
		Air content between 5 - 8%
		Air content greater than 4% and a SAM number less than 0.20
		Model calculations that show a mixture will reach critical saturation at 30 years
6.6	Transport Properties (choose one)	w/cm less than 0.50 if concrete is not subjected to freezing and thawing or deicer application
		w/cm less than 0.45 if concrete is not subjected to freezing and thawing or deicer application
		Formation Factor greater than 500 if concrete is not subjected to freezing and thawing or deicer application
		Formation Factor greater than 1000 if concrete is not subjected to freezing and thawing or deicer application

2.3 Optimized Aggregate Gradations

Historically, concrete mixtures consist of two aggregate gradation types: a fine aggregate and coarse aggregate. Optimized aggregate gradations (also known as “optimized gradations”) generally require the presence of at least three or more aggregate gradation types (often a fine aggregate, a mid-size aggregate, and a coarse aggregate) to maximize the packing potential in the mixture. By maximizing the packing potential of aggregates in a mixture, the amount of cement paste used can be minimized to the amount needed to meet workability requirements of the mixture (Lindquist et al. 2015). Studies have shown that optimized aggregate gradation mixtures can have the same or improved workability and finishability with reduced particle segregation during vibration (Cramer et al. 1995).

2.3.1 Particle Packing

Optimized aggregate gradations seek to maximize the particle packing of the aggregates, in turn reducing the aggregate voids in a concrete mixture. To reduce voids, models have been developed in order attempt to maximize the packing density, or the ratio of the solid volume of the aggregates in a concrete mixture to the volume of the concrete mixture itself (Mangulkar and Jamkar, 2013). Packing density models assume that voids between larger aggregates will be filled by smaller aggregates (Mangulkar and Jamkar, 2013). There are different types of models that attempt to maximize the packing density, but these models can generally be grouped into discrete and continuous models.

Discrete particle packing models examine two or more unique particle sizes, where the voids from the largest particles are filled by smaller particles (Kumar and Santhanam 2003). Discrete models may be classified as binary, which assumes the ideal packing of two particle sizes, ternary, which assumes the ideal particle packing of three particle sizes, or multimodal. (Kumar and Santhanam 2003). Continuous models assume all possible sizes are in the particle distribution system with no gaps between particle sizes (Kumar and Santhanam 2003).

Most particle packing models assume that the particles are spherical. However, studies have shown shape factor and convexity of the aggregate are the most important geometric factors, while mean size, specific gravity, and the voids ratio of the aggregate are the most important size parameters influencing the packing of the aggregate (Mangulkar and Jamkar 2013). Historically, concrete mixtures have used two aggregate types (coarse and fine) in a blended mixture. This combination has been shown to produce voids that are larger than necessary, which require more cementitious material to fill than mixtures with less voids. Particle packing methods and other methods that seek to optimize aggregate gradations, are an attempt to minimize the voids remaining in aggregate gradations, and to reduce the cementitious material by introducing one or more intermediate size aggregates that will fill voids left by traditional aggregate combination gradations.

2.3.2 Aggregate Gradation Methods

The aggregate gradation(s) selected for use in a mixture depends on a variety of factors, including the element thickness, reinforcing details, required workability, and available aggregate types and gradations. A wide range of aggregate gradations have been used in different concrete applications. However, historically only one coarse and one fine aggregate, have typically been used in a mixture design, with the gradation of each considered separately (not combined and evaluated with the design process). Coarse aggregates consist of one aggregate type with particles predominantly larger than 0.2 in, and fine aggregates consist of natural sand or crushed stone with particles smaller than 0.2 in. ASTM C 33 is a standard written to ensure aggregates meet specified grading requirements, sourcing requirements, and limits on the amount of deleterious substances allowable in both fine and coarse aggregate (ASTM 2018). An aggregate gradation can be determined by performing a sieve analysis (ASTM C 136) on a representative sample. The results of the gradation analysis are used to understand the sizes of aggregates contained in a certain supply of material. Results are expressed as a percent retained on a sieve or percent passing a sieve, and then are compared to predetermined aggregate size numbers and their percentages passing each sieve in ASTM C 33 (ASTM 2018, ASTM 2020).

Gradations can be considered to be uniformly graded, well graded, or gap-graded. Essentially, concrete mixtures that historically use a single coarse aggregate and a single fine aggregate function as “gap-graded” combined aggregate gradations. Gap-gradations contain a relatively small percent retained on the mid-size sieves, theoretically allowing larger voids between the larger aggregates to be filled by the much smaller aggregates. Gap-graded mixtures could achieve desired workability levels, however, would require more cementitious material to achieve the workability due to the larger surface area of the smaller-sized aggregates. In addition, gap-graded combined aggregate gradations have proved to have problems with edge slump, segregation during vibration, and wear resistance (Richardson 2005).

Well-graded, or dense aggregate gradations, are typically desirable for concrete mixtures because they reduce the volume of voids between aggregates. Well-graded aggregate gradations aim at maximizing the density of the gradation by maintaining a similar percentage of aggregates being retained on each sieve to minimize voids between aggregates. By minimizing the voids between the aggregates in the concrete mixtures, there is also a decreased amount of paste (and therefore total cementitious content) required to fill gaps between the aggregate within the concrete mixture (Obla et al. 2007). Dense aggregate gradations were originally developed by Fuller and Thompson in the early 1900’s. However, studies suggested maximum aggregate methods may not always provide the maximum strength or density of a concrete mixture, and that they produced mixtures that contained too little paste and were difficult to place (Wig et al. 1916, Talbot and Richart 1923, Richardson 2005).

Several approaches were developed to help optimize aggregate gradations and improve concrete performance. The Power 0.45 Curve gradation approach aims to identify an aggregate gradation that will maximize the density of the mixture by plotting the cumulative percent passing of sieve sizes to the 0.45 power (Kennedy et al. 1994). A gradation that creates a straight line on the Power 0.45 Chart between the smallest particle size and the largest particle size will create a gradation with the highest density and minimize the voids in a mixture. Although this method has provided success for some, there have also been studies that show this is not possible for particles smaller than the #30 sieve, and this method can create workability issues (Ley and Cook 2014).

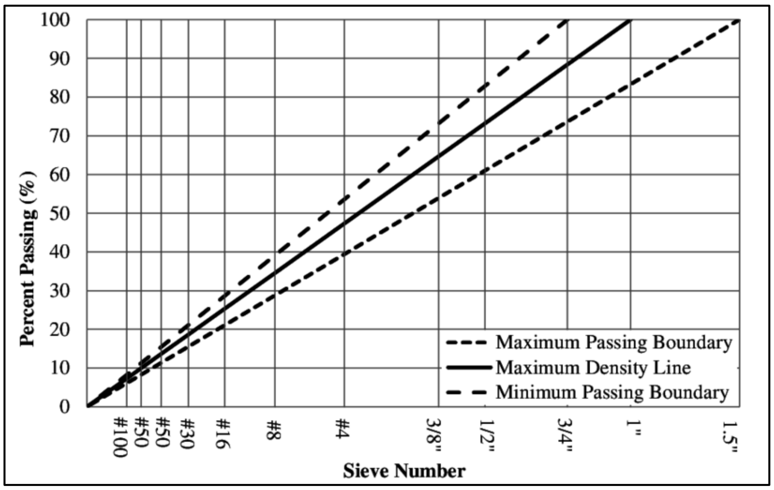

Figure 2.1: Power 0.45 curve (Ley and Cook 2014)

The Coarseness factor chart (also known as the Shilstone Chart, the Workability Chart) was developed by James Shilstone, and is an empirical approach on reducing segregation in mixtures (Shilstone 2002). This chart plots the workability factor, indicative of the amount of sand and cementitious material in a mixture, on the x-axis versus the coarseness factor, the ratio of large to intermediate aggregate, on the y-axis. The plot is segmented into 5 zones shown in Figure 2.2. The equations supporting the use of the Workability Factor chart are shown as Equation 2.1 and Equation 2.2.

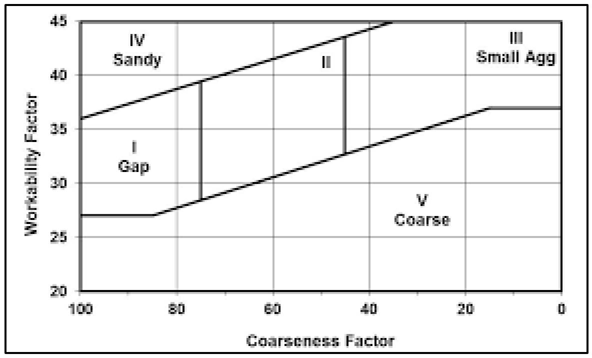

Figure 2.2: Coarseness factor chart

$$Workability\ factor = P + \frac{2.5 * (M - 564)}{94} \quad \text{Equation 2.1}$$

$$Coarseness\ factor = \frac{R}{S} * 100 \quad \text{Equation 2.2}$$

$$P = culula tive\ percent\ passing\ the\ number\ 8\ sieve \quad \text{Equation 2.3}$$

$$M = cementitious\ material\ content\ (\frac{lb}{yd^3}) \quad \text{Equation 2.4}$$

$$R = cumulative\ percent\ retained\ on\ the\ \frac{3}{8}"\ sieve \quad \text{Equation 2.5}$$

$$S = culative\ percent\ retained\ on\ the\ number\ 8\ sieve \quad \text{Equation 2.6}$$

Zone II is the desirable area on the chart, indicating a well-graded design for a concrete mixture. Zone I indicates aggregate blends that are gap-graded, and include little amounts of intermediate aggregate – these blends result in mixtures that tend to segregate during placement. Zone III also indicates aggregate blends that are gap-graded but have very little coarse aggregate. Zone IV indicates aggregate blends that contain a large amount of sand and can be expected to have low strength and segregate during vibration. Zone V is indicative of aggregate blends that are heavy in coarse aggregates and can be considered “very rocky” (Shilstone 2002).

The 8-18 Band Gradation, an aggregate gradation recommendation that is typically credited to Holland, requires the total percentage of fine and coarse aggregate retained on any sieve to be between 8 percent and 18 percent to prevent gap-gradations and prevent gradations from being too coarse or fine (Holland 1990). There are different variations of the 8-18 Band Gradation published in literature. For example in a subsequent revision, ACI 302.1R-96 "Guide for Concrete Floor and Slab Construction" section 5.4.3 recommended that smaller maximum size aggregate gradations (3/4 in or 1 in) have 8-22 percent retained on sieves ranging from the #100 – the maximum size aggregate (ACI 1996). In a subsequent revision, ACI 302.1R-15 has modified this in section 8.5.4, recommending a band gradation without setting particular limits requiring certain percentages to be retained on each sieve (ACI 2015).

2.3.3 Tarantula Curve

The Tarantula Curve was developed by researches at Oklahoma State University and is based on a modification to the 8-18 band gradation chart (Cook et al. 2013). It was developed by using five different aggregate types from different quarries in Oklahoma, designing mixtures with a constant w/cm ratio, and creating specific gradations to investigate the impacts of varying amounts of aggregates retained on each sieve. Proportions were identified based on three points of the Coarseness Factor chart (Shilstone chart), one point in the middle of zone II, one point on the border of zone II and V in between zones I and II, and one point on the border of zone I and II in between zones IV and V (Cook et al. 2013).

To determine the sieve limits required for coarse aggregates, the gradation of sand was held constant while varying the amounts retained on each sieve and charting the amount of WRA required to pass the Box Test, which is indicative of the workability of the mixture (Cook et al. 2013). It was determined that when intermediate or coarse aggregate retained on a single or multiple sieves became excessive, the amounts of WRA required to achieve the desired workability increased drastically, indicating the mixture had poor workability. Ultimately, it was determined that coarse and intermediate aggregates should be limited to 20 percent retained for sieves #4 – 3/8 inch. Additionally, the gradation with the lowest intermediate aggregate retained and highest coarse aggregate retained required the most WRA and had segregation and edge slumping issues (Cook et al. 2013).

The impact of gap-gradation was investigated using mixtures with varying amounts of intermediate and coarse aggregate, while maintaining a constant volume and gradation of sand (Cook et al 2013). A minor gap created on the 3/8 inch sieve and found to help the user achieve the desired workability with a typical amount of WRA. The gap moved from the 3/8 inch sieve to the #4 sieve. A gap on the #4 sieve achieved the desired workability with no added WRA, but was deemed to be on a borderline of acceptable due to the lack of required WRA. The aggregate size fractions were then redistributed to the 3/8 inch and 1/2 inch sieves, reducing the aggregate retained on the intermediate sieve sizes, causing a large increase in the required amount of WRA added to the mixture to achieve the desired workability (Cook et al. 2013).

Several aggregate gradations used in mixtures in the research study were designed to purposely contain low amounts of aggregate retained, or “valleys” which are thought to reduce workability. These were included in the study to explore their impact on a mixture’s workability (Cook et al. 2013). To understand the impact of valleys on the 3/8 inch sieve, gradations were created by varying the amount of aggregate retained on the 3/8 inch sieve between 0 – 15 percent while restricting the aggregate retained on all other sieves to less 20 percent, as previously discussed. The results of these mixtures showed a single valley did not negatively affect the workability of a mixture and did not require an excessive amount of WRA to attain the desired workability. Valleys were further studied by creating three aggregate gradations with troughs on the #4 and 3/4 inch sieves, and a valley on the 3/8 inch sieve. The mixture with the largest amount of aggregate retained on the #4 and 3/4 inch sieve and no aggregate retained on the 3/8 inch and 1/2 inch sieves required a jump in the required WRA to achieve desired workability. The results of studying valley gradations demonstrated that mixtures perform satisfactorily as long as a single sieve does not retain too much aggregate (Cook et al. 2013).

The impact of the maximum aggregate size on concrete workability was investigated by creating concrete using three gradations with a maximum aggregate size of 1/2 inch, 3/4 inch, and 1 inch, where no sieve retained more than 20 percent and had a similar volume of sand in each mixture (Cook et al. 2013). The mixture with the largest maximum aggregate size of 1 inch required the least amount of WRA to achieve desired workability. However, the difference in WRA required of all mixtures was not large enough to deem that the maximum aggregate size led to an improvement in workability. While the difference was not great enough to deem significant, it was noted that having a larger maximum aggregate size allows for an easier optimization of aggregate gradation, since it allows for a larger number of sieves to be used, effectively preventing any one sieve from retaining an excessive amount (Cook et al. 2013).

To determine the sieve limits for sand, coarse (#4 through #30 sieves) and fine (#30 through #200 sieves) sand were investigated separately, using the same method as with the coarse aggregates. The WRA required for concrete mixtures produced using different aggregate gradations to pass the Box Test were tracked (Cook et al. 2013).

To investigate the sand limits for each sieve for coarse sand, mixtures were designed to investigate the impacts of varying amounts of aggregate retained on each sieve individually (Cook et al. 2013). Amounts of aggregate retained on the sieve being investigated varied between 0 percent and 12 percent retained. All sieves smaller were held constant in their percent retained, and coarse aggregates held as constant as possible but requiring they be limited to the previously established maximum 20 percent retained per sieve. Using this method, a recommendation was developed that at least 15 percent of the aggregate should be retained on the #8 through #30 sieves to maximize workability (Cook et al. 2013).

To investigate the influence that fine sand has on the workability of a mixture, mixtures were designed to investigate the influence of varying amounts of aggregate retained on each individual sieve (Cook et al. 2013). Aggregate gradations were designed by holding 1 inch through #16 sieves consistent and investigating each sieve size individually by altering the percent retained on the sieve being investigated to determine how much WRA was required to produce a workable mixture. The recommendation for percent retained for fine sand, or sieves #30 - #200, is 24-34 percent retained, and the recommendation for percent retained for coarse sand, or sieves #8 - #30, is a minimum of 15 percent retained (Cook et al. 2013).

Ultimately, the research effort resulted in the development of a recommended framework for the combined gradation of aggregates. The recommended framework also includes guidance on what issues may occur if the gradation curve falls outside the framework. The Tarantula Curve does not provide a unique gradation aggregate gradation to optimize a concrete mixture. However, it provides range with multiple possible gradations to allow users to develop an optimized aggregate gradation. A figure of the framework (which resembles the shape of a Tarantula, spawning the name of the approach) can be found in Figure 2.3 (Cook et al. 2013).

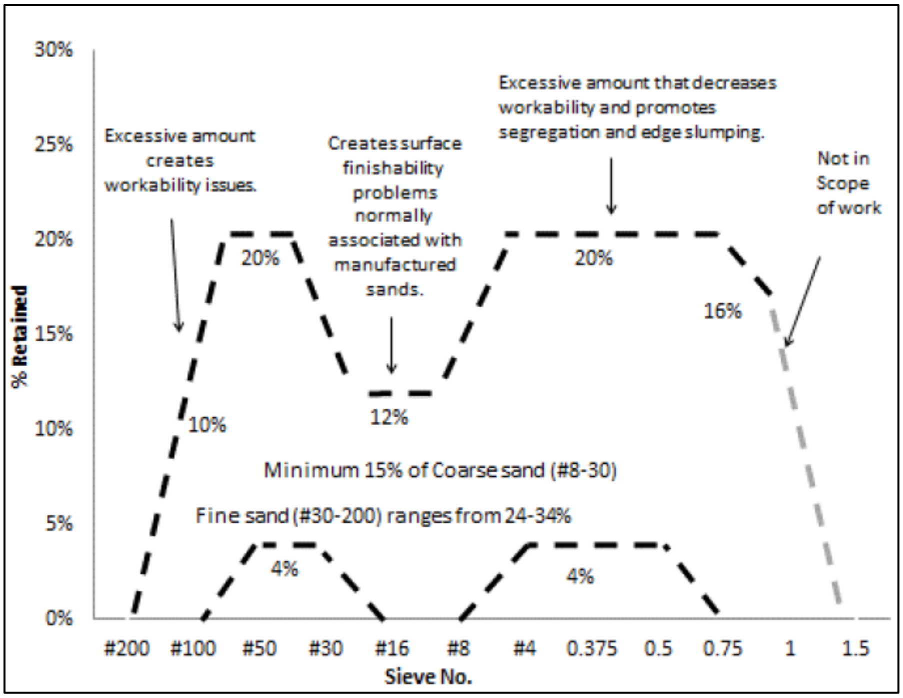

Figure 2.3: Tarantula Curve

CHAPTER 3: METHODOLOGY

3.1 Introduction

The potential benefits of optimized aggregate gradation, such as reduced costs and improved durability have been discussed in Chapter 2. To support implementation of performance specifications such as optimized gradations, research is needed. This chapter presents the methods used to optimize previously designed mixtures, materials used, and the testing schedule followed to compare to the original mixtures in an effort to support potential changes to NCDOT's specifications.

3.2 Development of Mix Design

The mixture matrix was developed by analyzing 24 previously designed and tested mixtures by the NCDOT and research team (Cavalline et al. 2020). Twelve mixtures were designed to meet Class AA bridge deck specifications, with 12 mixtures having relatively lower cement contents that could be considered for either structural or paving applications. Material types and sources were kept consistent for all mixtures, including aggregates, fly ash, admixtures, and water.

Three parameters were identified as key variables for the evaluation: w/cm ratio, cementitious material content, and fly ash replacement. Each of these variables was selected and controlled to replicate mixtures typical of NCDOT bridge (Class AA) and pavement construction. As shown in Figure 3.1, w/cm ratios of high (0.47), medium (0.42), and low (0.37) were used to divide the mixture matrix into three categories representative of typical, higher than typical, and lower than typical w/cm ratios. The w/cm ratios were selected to compare test results of non-optimized aggregate gradation concrete mixtures that are typical to NCDOT use, and poor concrete mixtures that are not typical to NCDOT use such as the high (0.47) w/cm mixtures.

These categories were further separated by cementitious material content per cubic yard; 700 pounds per cubic yard (pcy) to represent Class AA bridge mixtures often used on decks and other higher-strength applications, 650 pcy to represent Class AA structural mixtures with a slightly lower cementitious material content, and 600 pcy to represent Class AA structural mixtures with lower cementitious materials contents and paving mixtures.

Using the ACI 211.1 proportioning guide, mixture proportions were developed, and can be seen in Table 3.1. Twenty-one mixtures used OPC Type I/II cement, while 3 mixtures in the medium w/cm – 600 cementitious material zone of the mix matrix used PLC Type I/II cement sourced from the same location as mixtures previously designed and tested for the NCDOT (Cavalline et al. 2020).

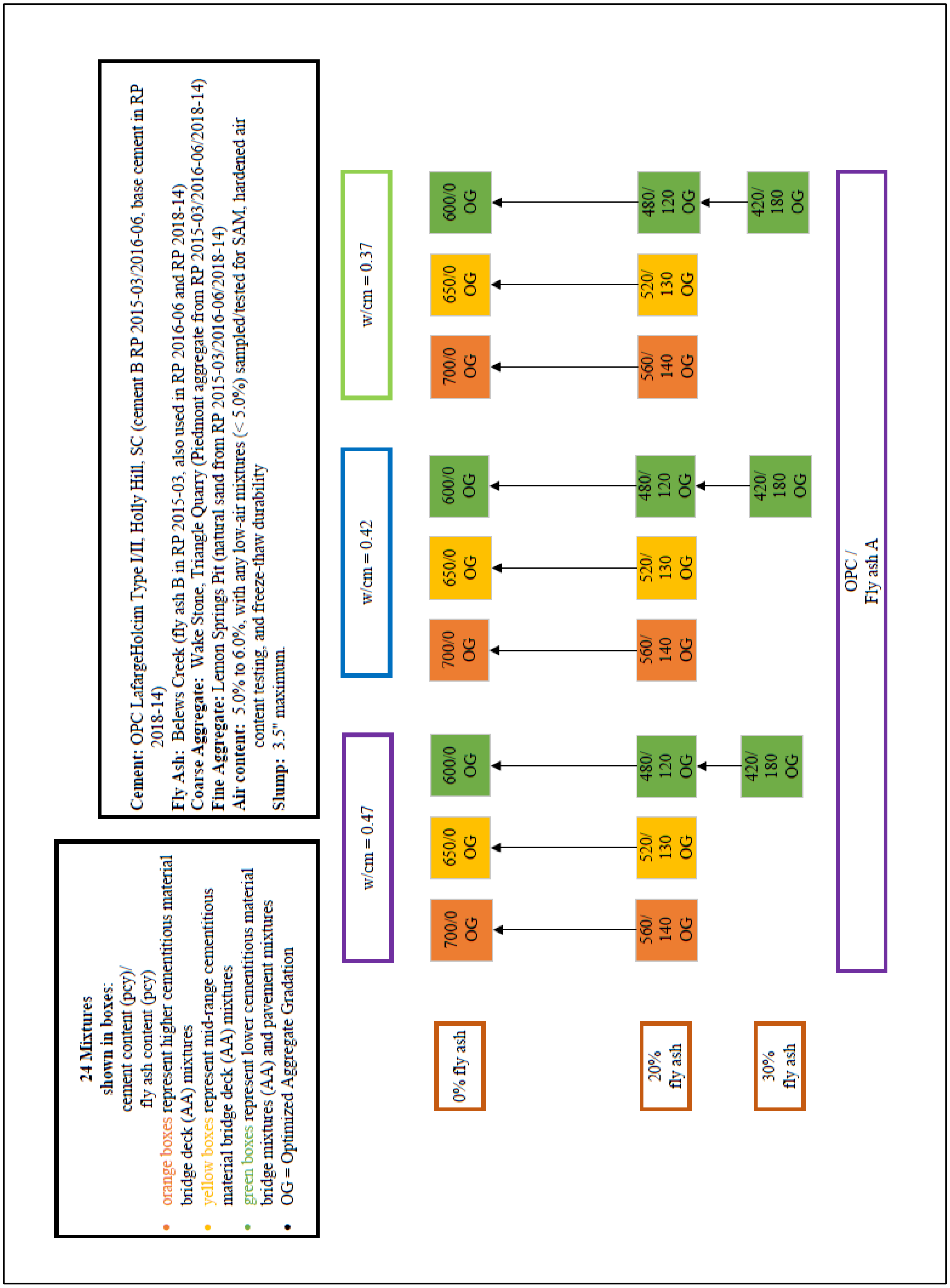

Figure 3.1 Concrete mixture matrix and supporting information

Table 3.1: Non-optimized aggregate gradation mixture proportions

Mixture ID **WW-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	1072	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

3.2.1 Implementation of the Tarantula Curve

AASHTO PP 84 suggests that mixtures should have a paste content less than 25 percent. With the twenty-four mixtures, material, and sources identified, non-optimized mixtures had paste contents ranging from 24.5 – 33.8 percent, and an average paste content of 28.5 percent. To support development of "optimized" mixtures, a target cementitious material reduction of 10 percent was selected (while adjusting water content to hold the w/cm ratio consistent) so that a uniform reduction could be applied to each non-optimized aggregate gradation concrete mixture.

This results in a fairly uniform comparison of non-optimized aggregate gradation mixtures with the optimized aggregate gradation mixtures. Larger paste reduction may have been feasible for some mixtures. However, this approach was not used since optimized versions of the non-optimized mixtures with paste contents already below the 25 percent recommendation by AASHTO PP 84 would have reduced paste contents close to 20 percent, causing concerns with the workability of these mixtures. The optimized mixtures with a 20 percent cementitious material reduction had a paste content ranging from 22.0 – 30.3 percent, with an average paste content of 25.6 percent. It should be noted, optimized gradation mixtures maintained the previous studies name structure of W-XXX-YYY, where W is the w/cm ratio, XXX is the cement content of the original non-optimized mixture and * denotes that the mixture has an optimized aggregate gradation with a reduced paste content, and YYY is the fly ash content of the mixture.

The quantities of each non-optimized mixture's materials were entered into a spreadsheet that projected each mixtures gradations across the Tarantula Curve with percent retained limits for each sieve discussed in Chapter 2. The limits set by the Tarantula curve are: the 1-inch sieve should retain less than 16 percent, sieves #4 – 3/4" should retain between 4 – 20 percent, sieves #8 and #16 should retain less than 12 percent, sieves #30 and #50 should retain between 4 – 20 percent, and sieve #100 retaining between 0 – 10 percent (Cook et al. 2013). A graph of the recommended Tarantula Curve boundaries can be seen in Figure 3.2.

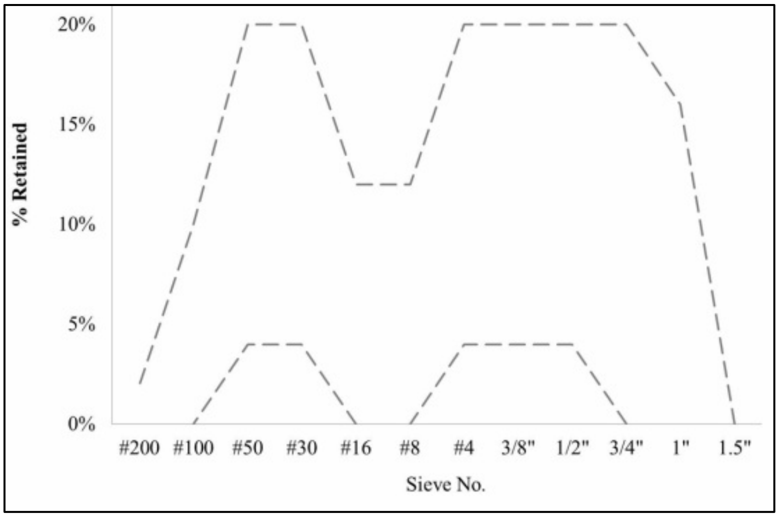

Figure 3.2: Tarantula Curve recommended gradation boundaries

The batch quantities of the non-optimized mixtures are presented in Biggers (2019) and Cavalline et al. (2020). Materials used in these mixtures were also used for the optimized aggregate gradation mixtures to support direct comparison. Aggregates used were the #67 stone from Wake Stone's Triangle Quarry, located in Raleigh, NC, and a natural sand from Lemon Spring's Quarry, located in Lemon Springs, NC. Once new supplies of these aggregates were obtained for this research study, sieve analyses were performed and were compared to the sieve analyses of the aggregates used for the non-optimized mixtures in the previous study to verify consistency. The gradations between the materials used for the initial study and this study were determined to be very similar, and can be seen in Figure A.1 in Appendix A.

The results from the sieve analysis of the aggregates obtained for production of the non-optimized mixtures quantities were then entered into a spreadsheet developed to assess the combined aggregates gradation with the guidance of the Tarantula Curve. In Figure 3.3, the combined aggregate gradation for the aggregates used in the non-optimized mixture H-700-0 is shown on the Tarantula curve. As can be seen, the combined gradation exceeds the recommended percent retained on the ½" sieve.

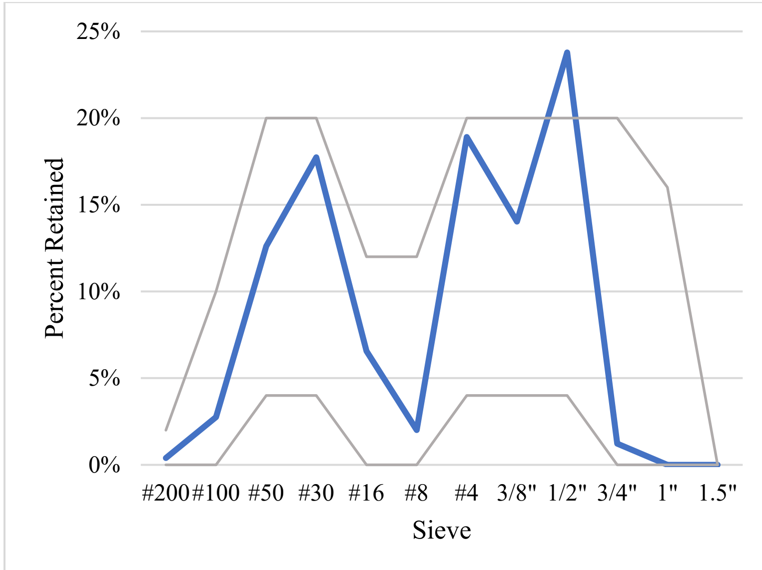

Figure 3.3: H-700-0 on Tarantula Curve

Using information collected from Wake Stone, a NCDOT approved supplier and the supplier of coarse aggregates from previous projects, aggregate gradations (#8, #9, #78, and #89M) were entered for an intermediate aggregate to analyze which size would best satisfy parameters of the Tarantula Curve while allowing for a reduction in cementitious content used. The #89M stone was selected from Wake Stone's Moncure Quarry, in Moncure, NC, because it had a peak percent retained distributed between the #4 and #8 sieves, shown in Appendix A, Figure A.2. The quantity of this intermediate aggregate used in each optimized concrete mixture was adjusted in a manner that allowed the combined, optimized aggregate gradation to meet the Tarantula Curve recommendations while trying to minimize the reduction in fine aggregate removed. Figure 3.4 shows the optimized mix H-700*-0.

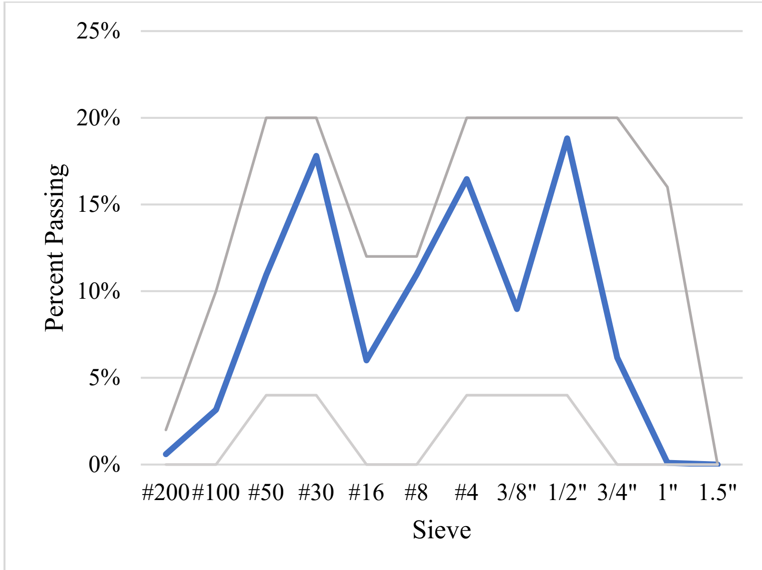

Figure 3.4: H-700*-0 optimized mixture

This process was repeated for all 24 mixtures designed previously by the research team (Biggers 2019, Cavalline et al. 2020). In addition to the twenty-one mixtures using Type I/II cement, RP 2018-14 included three mid-w/cm Class AA bridge deck/paving mixtures with PLC cement. Since PLC was not the subject of this study, these mixtures were not included in this study.

Cementitious material contents for non-optimized aggregate gradation mixtures varied from 600 pcy to 700 pcy while the cementitious material contents for optimized aggregate gradation mixtures varied between 378 pcy and 630 pcy after the 10 percent reduction. For both mixtures, the cementitious materials contents were varied, depending on the w/cm ratio and fly ash replacement rate. Fly ash was used at replacement rates ranging from 0 percent to 30 percent, resulting in quantities that varied between 0 pcy and 162 pcy.

The w/cm ratios selected for the non-optimized RP 2018-14 project mixtures (high – 0.47, moderate – 0.42, and low – 0.37) were used again in this project. Water contents for each mixture were determined based upon ACI 211 design procedures but were adjusted for the new cementitious material contents. The #89M gradation was selected to be used as an intermediate aggregate, with proportions selected based upon meeting the Tarantula Curve with the combined (fine, intermediate, and coarse) aggregate gradation, with resulting proportions for intermediate aggregate varying between 590 pcy and 697 pcy. The coarse aggregate (#67) content used in the optimized mixtures was the same as used in the non-optimized aggregate gradation mixtures. These quantities ranged from 1316 pcy to 1158 pcy. Fine aggregate amounts ranged from 1211 pcy to 1055 pcy. The finalized mixture matrix, with the optimized aggregate gradation mixtures that utilized the Tarantula Curve can be seen next to their companion mixtures in Table 3.2-Table 3.4.

Table 3.2: High (0.47) w/cm ratio mixtures

Mix ID (W-XXX*-YYY, where **W** is w/cm ratio, **XXX** is cement content where * deontes an optimized gradation, **YYY** is fly ash content)	**Mixture Characteristics**		**Mixture Proportions, pcy**						
	Mixture Type	**Cement Type**	**Fly Ash Replacement (%)**	**Cement**	**Fly Ash**	**No. 67 Coarse Aggregate**	**No. 89M Medium Aggregate**	**Fine Aggregate**	**Water**
H-700-0	AA (high and medium cm content)	OPC	0	700	0	1659	0	1072	329.0
H-700*-0			0	630	0	1175	620	1065	296.1
H-560-140			20	560	140	1659	0	1022	329.0
H-560*-140			20	504	126	1158	615	1055	296.1
H-650-0			0	650	0	1659	0	1175	305.5
H-650*-0			0	585	0	1215	640	1105	275.0
H-520-130			20	520	130	1659	0	1129	305.5
H-520*130			20	468	117	1204	632	1088	275.0
H-600-0			0	600	0	1659	0	1277	282.0
H-600*-0			0	540	0	1261	662	1130	253.8
H-480-120			20	480	120	1659	0	1235	282.0
H-480*-120			20	432	108	1243	652	1125	253.8
H-420-180			30	420	180	1659	0	1214	282.0
H-420*-180			30	378	162	1227	652	1124	253.8

Table 3.3: Mid (0.42) w/cm ratio mixtures

Mix ID (W-XXX*-YYY, where W is w/cm ratio, XXX is cement content where * deontes an optimized gradation, YYY is fly ash content)	Mixture Characteristics		Mixture Proportions, pcy						
	Mixture Type	Cement Type	Fly Ash Replacement (%)	Cement	Fly Ash	No. 67 Coarse Aggregate	No. 89 Medium Aggregate	Fine Aggregate	Water
M-700-0	AA (high and medium cm content)	OPC	0	700	0	1659	0	1163	294.0
M-700*-0			0	630	0	1206	636	1107	264.6
M-560-140			20	560	140	1659	0	111	294.0
M-560*-140			20	504	126	1193	626	1093	264.6
M-650-0			0	650	0	1659	0	1259	273.0
M-650*-0			0	585	0	1248	658	1130	245.7
M-520-130			20	520	130	1659	0	1214	273.0
M-520*-130			20	468	117	1235	650	1115	245.7
M-600-0			0	600	0	1659	0	1356	252.0
M-600*-0			0	540	0	1284	678	1162	226.8
M-480-120			20	480	120	1659	0	1313	252.0
M-480*-120			20	432	108	1277	672	1141	226.8
M-420-180			30	420	180	1659	0	1292	252.0
M-420*-180			30	378	162	1270	590	1211	226.8

Table 3.4: Low (0.37) w/cm ratio mixtures

Mix ID (W-XXX*-YYY, where **W** is w/cm ratio, **XXX** is cement content where * deontes an optimized gradation, **YYY** is fly ash content)	**Mixture Characteristics**		**Mixture Proportions, pcy**						
	Mixture Type	**Cement Type**	**Fly Ash Replacement (%)**	**Cement**	**Fly Ash**	**No. 67 Coarse Aggregate**	**No. 89 Medium Aggregate**	**Fine Aggregate**	**Water**
L-700-0	AA (low cm content and Pavement)	OPC	0	700	0	1659	0	1254	259.0
L-700*-0			0	630	0	1252	658	1122	233.1
L-560-140			20	560	140	1659	0	1205	259.0
L-560*-140			20	504	126	1224	650	1123	233.1
L-650-0			0	650	0	1659	0	1344	240.0
L-650*-0			0	585	0	1279	675	1159	216.0
L-520-130			20	520	130	1659	0	1298	240.0
L-520*-130			20	468	117	1270	668	1140	216.0
L-600-0			0	600	0	1659	0	1434	222.0
L-600*-0			0	540	0	1316	697	1186	199.8
L-480-120			20	480	120	1659	0	1392	222.0
L-480*-120			20	432	108	1297	688	1177	199.8
L-420-180			30	420	180	1659	0	1370	222.0
L-420*-180			30	378	162	1293	684	1173	199.8

3.3 Materials and Description

Materials and their sources used in the completion of the mixtures are described in the following sections. Properties of the materials have been obtained through testing at UNC Charlotte's laboratory or, where noted, were provided by the supplier or manufacturer.

3.3.1 Cementitious Material

Two cementitious materials were used for batching concrete in this project: OPC and Class F fly ash. To improve consistency between the previous research study and this work, cementitious materials from the same sources were used. Descriptions of the materials are provided in the following sections.

3.3.1.1 Portland Cement (OPC)

The OPC that was utilized in this project is typical for NCDOT paving projects. The OPC was used in all 21 mixtures completed for the research. LafargeHolcim provided the OPC from their manufacturing plant in Holly Hill, SC, which is a Type I/II cement that meets ASTM C150.

3.3.1.2 Fly Ash

Fly ash was used to reduce the amount of cement used in mixtures and to study the effects in fresh and hardened concrete. Currently, North Carolina Standard Specification 1024 "Materials for Portland Cement Concrete" allows up to 30 percent fly ash replacement by cement mass at a 1:1 replacement ratio (NCDOT 2018). In this project, percentages of 20 percent and 30 percent were used. This fly ash is a Class F fly ash from Belews Creek Power Plant in Belews Creek, NC, and a chemical analysis provided by Ash Venture can be seen in the Appendix Figure A.3.

3.3.2 Coarse Aggregate

Coarse aggregate was selected to match the previous mixtures and to reduce potential variability. The coarse aggregates selected for this project and the previous study complied with NCDOT Specification 1014-2, "Aggregate for Portland Cement – Coarse Aggregate", as well as ASTM C33, "Standard Specification for Concrete Aggregates" (Biggers 2019). The quarry that was selected to provide the aggregate was the same as used in the previous study completed by the research team. This quarry had been selected by the research team and NCDOT personnel because it represents a coarse aggregate typical of that specified in North Carolina paving and structural mixtures. Wake Stone's Triangle Quarry in Cary, North Carolina provided a No. 67

aggregate. Previous studies found the aggregate properties of the granitic gneiss aggregate to have a specific gravity of 2.63. This study found the material to have a specific gravity of 2.64. As can be observed by comparing the aggregate characterization test results from the two studies, the material was quite consistent.

3.3.3 Intermediate Aggregate

An intermediate aggregate from the Triangle area of NC was selected, based on the premise that it well represents aggregates used in North Carolina's paving mixtures in addition to being available in a gradation that would satisfy the Tarantula Curve. Wake Stone's Moncure Quarry in Moncure, North Carolina provided a No. 89M stone for this study. Aggregate properties of the granitic gneiss aggregate include a specific gravity of 2.66.

3.3.4 Fine Aggregate

The fine aggregate was selected to match previous studies completed by the research team. The aggregate was a natural sand meeting ASTM C33 specifications sourced form a natural sand pit quarry in Lemon Springs, North Carolina. Properties of the fine aggregate included a specific gravity of 2.62.

3.3.5 Chemical Admixtures

Two commercially available admixtures were used in this study, an AEA (MasterAir AE 200) and a mid-range WRA (MasterPolyheed 997) provided by BASF Construction Chemicals. The admixtures that were selected were the same admixtures used in previous studies to minimize variability in test results. These admixtures allowed the research team to meet the target w/cm ratios and cementitious contents while typically producing concrete mixtures with a target slump of 3.5 inch ± 1 inch and a fresh air content between 5 percent and 6 percent. NCDOT specification 1000-3(B), "Portland Cement Concrete for Pavement – Air Content" specifies an allowable air content of 5 percent ± 1.5 percent, a very limited range of 5 to 6 percent was used for this project to to reduce the influence of a wide range of air contents on test results (NCDOT 2018).

3.4 Testing Schedule

The testing program for this project was identical to the testing program used in NCDOT RP 2018-14 (Cavalline et al. 2020) to facilitate comparison of test results between conventional mixtures and optimized aggregate gradation mixtures. Testing was performed on fresh and hardened concrete in accordance with the AASHTO, ASTM, and other test procedures shown in Table 3.5. Since flexural strength tests are typically only required for pavement concrete, flexural strength (modulus of rupture) tests were only performed on the lower cementitious content mixtures, which were mixtures typical of pavement mixtures.

Table 3.5: Concrete testing schedule

	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 mixtures were prepared using a six cubic foot (cf) portable drum mixer. Batch sizes were computed based on the volume of material required to produce the test specimens needed to complete the required tests and estimated waste. For the 12 batches requiring modulus of rupture testing (600* pcy mixtures), it was determined 4.49 cf of concrete was required, and 2.84 cf for the remaining 12 mixtures (700* and 650* pcy mixtures). Due to waste, all mixtures were completed with 3.0 cf batches. To verify consistency between batches of the same mixture, compressive cylinders were prepared from each batch and tested.

Batching was performed in accordance with ASTM C685 (ASTM 2017). Non-paving mixtures had all specimens (excluding modulus of rupture beams) produced from one batch; including concrete for fresh concrete tests, fifteen cylinders (4 in x 8 in) for compressive strength, two (4 in x 8 in) cylinders for formation factor, two (4 in x 8 in) cylinders for rapid chloride penetration, two cylinders (6 in x 12 in) for modulus of elasticity, two hardened air content specimens, three beams (3 in x 4 in x 10 in) for shrinkage, and three beams (3 in x 3 in x 12 in) for freeze-thaw.

Specimens for low-cementitious content (paving) mixtures were prepared in two batches. One mixture included preparation of concrete for fresh tests, fifteen cylinders for compressive strength, two cylinders for formation factor, two cylinders for rapid chloride penetration, and two cylinders for modulus of elasticity. The second mixtures included preparation of concrete for fresh tests, two hardened air content specimens, three beams for shrinkage, three beams for freeze-thaw, and three beams (6 in x 6 in x 18 in) for modulus of rupture.

3.6 Testing of Fresh Concrete Properties

Testing of fresh concrete was conducted immediately after mixing to ensure that each mixture met the project requirements. These tests included slump, fresh air content, temperature, density, and SAM. SAM testing provides the fresh air content and SAM number, which relates to the quality of the air matrix in fresh concrete (Ley et al. 2017). Procedures and standards used for obtaining the fresh concrete test results are described in the following sections.

3.6.1 Slump

Slump testing was performed on each batch of concrete in accordance with ASTM C143 (ASTM 2020). Although a target slump of parameter of 3.5 inch +/- 1 inch was desired, to meet the goals of this project, it was more important that the w/cm ratios were maintained. Deviations from the target slump were deemed acceptable when (if low) the mixture could be adequately consolidated into molds following the appropriate ASTM procedures and (if high) when the slump could be attributed to the design characteristics of the mixture. For example, mixtures with a w/cm ratio of 0.47 often had slumps in excess of the target range.

3.6.2 Air Content

Air content testing was performed using a Type B pressure meter per ASTM C231 (ASTM 2017). The acceptance parameter for these concrete mixtures was 5.0 percent - 6.0 percent, which is tighter than the typical NCDOT range of 5.0 percent ± 1.5 percent (NCDOT 2018). A tighter acceptable air content range was desired and enforced to ensure consistency between mixtures, and to ensure that changes in performance could not be attributed to wide disparities in entrained air content between mixtures. The non-optimized mixtures for RP 2018-14 were also required to have air contents between 5.0 percent - 6.0 percent.

3.6.3 Super Air Meter (SAM)

SAM testing provides an insight to the durability properties of the concrete mixture by providing a SAM number that is correlated with how well the air voids are spaced in the concrete as well as their size. A network of small air voids that are evenly distributed throughout the concrete helps to mitigate the risk of damage due to freeze-thaw cycles.

The SAM test is performed in a manner similar to the ASTM C231 air content test using the Type B pressure meter. However, the SAM device is capable of being pressurized to a greater extent, and the test is controlled by a program in the digital dial gauge mounted at the top of the device. The test is performed per AASHTO TP 118 (AASHTO 2017). Upon completion, a SAM number provided that can be interpreted to determine the distribution of the air voids in the concrete. A SAM number of 0.20 or less correlates “over 95 percent of the time” to a recommended hardened spacing factor of 0.008 inches (LeFlore 2016). There was no specified SAM acceptance parameter for the project.

3.6.4 Unit Weight

Unit weight testing was performed in accordance with ASTM C138. The test was performed immediately after mixing was completed, in the same bucket used for air content (ASTM 2017). Fresh unit weight was also used to verify the proper materials and proportions were used.

3.7 Preparation and Curing of Test Specimens

Test specimen preparation was performed per ASTM C192, and other ASTM and AASHTO standards set for each test (ASTM 2018). Form release was applied to all molds prior to batching to allow an easier demolding process. Due to COVID-19, variations in the team preparing each mixture were inevitable. However, when possible, the responsibilities of each team member remained constant throughout all mixtures to minimize variability in the characteristics of the specimens made. After demolding, specimens were placed in a moist curing room in accordance with ASTM C511, except for shrinkage beams which were placed in a lime water bath in accordance the appropriate test standard (ASTM 2019). Specimens were removed from curing for testing at the appropriate age per the testing standard.

3.8 Testing of Hardened Concrete

The hardened concrete testing program can be found in Table 3.5. Tests relating to mechanical properties include compressive strength, MOR, MOE and Poisson’s ratio, and shrinkage. Tests relating to the durability that were performed were surface resistivity, rapid chloride permeability, and the Bucket Test.

3.8.1 Mechanical Properties

Mechanical property tests performed as part of this study are described in the subsections below.

3.8.1.1 Compressive Strength

Compressive strength testing was performed per ASTM C39 on 4-inch x 8-inch cylinders tested at 3, 7, 28, 56, and 90 days (ASTM 2018). The minimum acceptable compressive strength by the NCDOT 2018 Roadway Standard Specifications is 4,500 psi (NCDOT 2018).

3.8.1.2 Modulus of Rupture (MOR)

MOR testing was performed according to ASTM C78 on only the paving mixtures (600 lb/cy non-optimized mixtures; 600* lb/cy optimized mixtures). The MOR test evaluates the flexural strength of the concrete (ASTM 2018). Per the standard, the specimens were tested 28 days after the mixing date. NCDOT Roadway Standard Specifications Section 1000-3 requires a minimum MOR average of 650 psi (NCDOT 2018).

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

MOE and Poisson's ratio were determined using procedures from ASTM C469 and this testing was performed 28 days after the mixing date (ASTM 2014). For each test to have accurate loading and data recording, two team members were present for each test – one controlling the loading rate and the other using a camera to record the data on the dial gauges.

3.8.2 Durability Properties

Tests performed to evaluate each concrete mixtures durability characteristics as part of this study are described in the subsections below.

3.8.2.1 Surface Resistivity

Surface resistivity tests were performed per AASHTO T 358 on three 4-inch x 8-inch cylinders at 3, 7, 28, 56, and 90 days after the mixing date (AASHTO 2017). Tests were performed on the same cylinders that would be used for compressive testing at the same dates. The resistivity meter that was used was a Resipod surface resistivity meter, manufactured by Proceq. Since specimens were cured in a moist room, the correction for lime-water curing was not applied to the readings.

3.8.2.2 Chloride Permeability

The concrete susceptibility to chloride ion penetration was tested at ages 28 and 90 days after the mixing date following the procedure in ASTM C1202 (ASTM 2018). Two 4-inch x 8-inch cylinders were cast from each mixture. From these two cylinders, the test specimens were prepared by cutting the top and bottom from each cylinder, creating four specimens each with a radius of 4 inches and a height of 2 inches with two specimens to be used at 28-day testing and two to be used at 90-day testing. To reduce variability in the resulting from potential differences between the two cylinders, each testing day used one specimen from each of the original two cylinders.

Prior to performing the RCPT test, the specimens had to be conditioned. The conditioning process prescribed in ASTM C1202 includes bringing water to a boil, then letting it cool to room temperature to de-air the water. The specimens were air dried for at least one hour. Following the drying period, the air-dried specimens were placed in an airtight container attached to a vacuum and a second airtight container containing the de-aired water. The vacuum was turned on, and the specimens were subjected to vacuum absolute pressure less than 50 mm Hg for three hours. After three hours, the de-aired water in the airtight container was then emptied into the container with the specimens until they were fully submerged. Once submerged, the vacuum was again turned on and allowed to run for one hour. After an hour, the vacuum was turned off, the pressure was released, and the specimens soaked for 18 ± 2 hours.

After conditioning, the specimens were placed into testing cells, with the flat surfaces of the specimens exposed to sodium chloride at the negative terminal, and sodium hydroxide at the positive terminal. Rubber gaskets and bolts were used to ensure a tight seal with the specimen to prevent the solution from leaking during the test. Test cells were then connected to a testing and data collection unit that subjects each specimen to a 60 volt potential for six hours. Test results for current flow through each specimen were reported in coulombs, with lower measurements indicating a better resistance to chloride penetration.

RCPT results predict the ability for all ions, not just chloride ions, to pass through the pore solution, and can be skewed depending on the cementitious matrix of the concrete mixture. The use of admixtures such as AEA, WRA, or corrosion inhibitors, and SCMs such as fly ash or silica fume, can create misleading RCPT results that do not accurately indicate the mixtures permeability (Joshi and Chan 2002).

3.8.2.3 Shrinkage

Shrinkage tests were performed using an unrestrained shrinkage testing method, per ASTM C157 (ASTM 2017). For each mixture, three specimens were cast. Specimens were wet cured for 28 days, at which point they were transferred to an environmental chamber with a controlled environment. Per the standard, the environmental chamber is required to a constant temperature of 73.0 degrees Fahrenheit (°F) ± 3.0°F, and constant relative humidity of 50 percent ± 4.0 percent. Specimens were then measured at days specified in the standard.

3.8.2.4 Bucket Test and Formation Factor

The Bucket Test was performed per AASHTO TP 119-15 as an experimental method for evaluating the formation factor of a concrete mixture (AASHTO 2017). This method involves performing resistivity testing on concrete cylinders that have been submerged in a standardized pore solution that mimics the pore solution of concrete. Developed at Oregon State University, the solution consists of 7.6 g/L of sodium hydroxide (NaOH), 10.64 g/L of potassium hydroxide (KOH), and 2 g/L of calcium hydroxide [$Ca(OH)_2$] (Weiss et al. 2016). For convenience, this can be made in a 5-gallon bucket to create 18.9 L of solution using 13250 grams (g) of water, 102.6 g NaOH, 143.9 g of KOH, and 27 g of $Ca(OH)_2$ (AASHTO 2017).

The Bucket Test is used to calculate the Formation Factor using an assumed pore solution of 0.127 Ωm and can allow for comparison of cement mixtures with complex cementitious systems (AASHTO 2020). The mixture matrix in this research project does not have a complex cementitious system, and only consists of one type of cement and one type of fly ash. The Formation Factor was calculated in this research project for data purposes only.

CHAPTER 4: TEST RESULTS AND ANALYSIS

This chapter provides a summary of all test results for the testing program previously described in Chapter 3, followed by an analysis of these test results. The intent of this chapter is to present the results and compare them to NCDOT specification targets (where applicable), and also compare the performance of the conventional set of mixtures from RP 2018-14 to the performance of the optimized aggregate gradation mixtures batched and tested as part of this project.

As a reminder to the reader, the mixtures have been named to identify their characteristics. The first letter of each mixture designation indicates the w/cm ratio that it falls under; “H” for a w/cm of 0.47, “M” for a w/cm of 0.42, or “L” for a w/cm of 0.37. The three-digit number following the w/cm designation designates the amount of cement per cubic yard in the mixture. Mixtures will have cement per cubic yard in their names ranging from 420 to 700, however mixtures that have an asterisk (*) attached to the cement pcy, designate an optimized aggregate gradation mixture. As previously mentioned, optimized aggregate gradation mixtures have had 10 percent of the cement content removed and replaced with coarse aggregate, intermediate aggregate, and fine aggregate in proportions that cause the combined aggregate gradation to comply with the Tarantula Curve while maintaining the desired w/cm ratio. The final number in the mixture designation refers to the fly ash content, ranging from 0 to 180 pcy determined based on replacement rates of 0 to 30 percent as described in Chapter 3.

4.1 Testing of Fresh Concrete

This section provides an overview the results from tests discussed in Section 3.6 pertaining to fresh concrete. Tests conducted on fresh concrete and were performed on each mixture were slump, air content, SAM, and unit weight. The results from these tests for non-optimized and optimized aggregate mixtures can be found in Table 4.1.

Table 4.1: Fresh concrete properties

Mixture ID	Slump (in.)		Air Content (%)		Unit Weight (pcf)	
	Non-Optimized	Optimized	Non-Optimized	Optimized	Non-Optimized	Optimized
H-700-0	8.0	5.0	5.2%	5.8%	137.1	144.5
H-560-140	8.0	3.5	5.2%	5.0%	136.4	142.3
H-650-0	6.5	5.0	6.0%	6.0%	141.4	140.9
H-520-130	7.0	2.3	5.5%	5.2%	138.0	143.1
H-600-0	2.5	0.0	5.8%	6.0%	138.7	144.3
H-480-120	3.0	3.3	6.0%	5.9%	139.4	142.9
H-420-180	3.8	1.9	6.0%	5.0%	136.1	142.9
M-700-0	5.0	3.8	5.5%	5.5%	141.6	142.9
M-560-140	4.3	6.0	6.0%	5.0%	136.6	142.1
M-650-0	2.5	2.5	5.7%	6.0%	142.4	144.2
M-520-130	3.0	1.8	5.5%	5.0%	139.7	145.8
M-600-0	1.0	0.8	6.0%	5.7%	140.5	143.0
M-480-120	1.5	0.9	5.0%	5.7%	139.6	144.9
M-420-180	2.0	0.8	6.0%	5.1%	138.1	144.3
L-700-0	2.3	1.5	6.0%	5.9%	143.9	144.5
L-560-140	1.8	0.5	5.0%	5.7%	140.3	142.9
L-650-0	1.0	1.0	6.0%	6.0%	141.8	144.2
L-520-130	1.0	0.5	5.0%	5.3%	141.6	144.1
L-600-0	1.0	0.0	5.5%	5.7%	142.6	144.3
L-480-120	0.8	0.0	5.5%	5.6%	142.0	144.0
L-420-180	1.0	0.0	5.2%	5.5%	142.0	144.0

4.1.1 Slump

The target slumps for mixtures in this project were 3.5 inches ±1 inch. However, based on the objectives of the project, it was more important to ensure the desired w/cm ratios were maintained. Several mixtures had slumps that exceeded this target (H-700-0, H-700*-0, H-560-140, H-650-0, H-650*-0, H-520-130, M-700-0, M-560-140, M-560*-140) but this could be expected since these mixtures tended to be the higher w/cm ratio mixtures. Mixtures with slumps below 3.5 inches were deemed acceptable only if they could be adequately consolidated into the specimen molds.

In general, slumps for optimized aggregate gradation mixtures were lower than their companion non-optimized mixtures. The slumps measured for the optimized aggregate gradation mixtures with higher w/cm ratio (0.47 and 0.42) and higher cement content mixtures (650* and 700* pcy mixtures) mixtures required less WRA to achieve the target slump range than optimized gradation mixtures with low (0.37) w/cm ratios and low cement contents (600* pcy). Optimized aggregate gradation mixtures tended to have lower slumps than non-optimized aggregate gradation mixtures, which could be expected due to the reduced paste content. However, it should be noted M-560*-140 had a slump almost two inches higher than its companion non-optimized mixture. This could be attributed to the fact that the WRA dosage was identical to its optimized straight cement mixture, coupled with the 20% fly ash replacement led to a high slump. Two optimized aggregate gradation mixtures required no WRA (H-700*-0 and H-560*-140) compared to six non-optimized aggregate gradation mixtures (H-700-0, H-560-140, H-650-0, H-520-130, H-600-0, and M-700-0). Of the optimized aggregate gradation mixtures requiring above the average WRA dosage, 4 of 10 were straight cement mixtures, and 7 of 10 were from the lowest w/cm ratio (0.37).

4.1.2 Air Content

The air content for all mixtures batched and tested as part of this project, as well as the previous study focused on non-optimized aggregate gradation mixtures, was restricted to a range between 5.0 and 6.0 percent in order to reduce variability in test results that could be attributed to large differences in air content.

Varying air contents could be attributed to varying cement and fly ash content in the mixtures, minor changes in material temperatures, atmospheric conditions during mixing, and WRA and AEA dosages. AEA dosages for optimized aggregate gradation mixtures ranged from

0.06 – 0.76 oz/cwt. Eight optimized gradation mixtures required higher than the average AEA dosage for all mixtures. Of the eight optimized mixtures with an above average AEA dosage, 5 of the 8 mixtures had fly ash, and 7 of 8 mixtures were from the lowest w/cm ratio (0.37), and 7 of 8 mixtures also required an above average WRA dosage.

4.1.3 Super Air Meter (SAM)

SAM numbers were collected for all optimized aggregate gradation mixtures using the procedure outlined by OSU (AASHTO TP 118, 2017). The results of this testing can be found in Table 4.2. Due to the limited amount of fresh concrete could be produced with each batch, a limited batch quantity, and a required number of test specimens to be made, only one SAM test could be run for each batch. When an error was encountered, a decision was made to not run the test again since the re-test would require sacrificing concrete needed for casting test specimens.

Table 4.2: SAM numbers for optimized aggregate gradation mixtures

Mixture ID	Batch 1	Batch 2
H-700*-0	0.29	-
H-560*-140	0.40	-
H-650*-0	0.13	-
H-520*-130	0.30	-
H-600*-0	0.24	Error
H-480*-120	Error	0.18
H-420*-180	0.08	0.24
M-700*-0	0.62	-
M-560*-140	0.74	-
M-650*-0	0.43	-
M-520*-130	0.41	-
M-600*-0	Error	Error
M-480*-120	0.21	0.53
M-420*-180	Error	Error
L-700*-0	0.20	-
L-560*-140	0.39	-
L-650*-0	0.42	-
L-520*-130	0.27	-
L-600*-0	0.77	Error
L-480*-120	Error	0.51
L-420*-180	0.47	Error

4.1.4 Unit Weight

As could be reasonably expected due to the denser aggregate packing, the optimized aggregate gradation mixture unit weights were slightly higher (ranging from 140.9 pounds per cubic foot (pcf) to 145.8 pcf and averaging 143.6 pcf) than their companion non-optimized aggregate gradation unit weights (which ranged from 136.1 pcf to 143.9 pcf and averaged 139.9 pcf). The variation in unit weights can be expected as proportions change from mixture to mixture. Average unit weights of optimized aggregate gradation mixtures increased as the w/cm ratio decreased, with a less pronounced increase as cementitious contents decreased, consistent with their companion non-optimized aggregate gradation mixtures. Of the optimized aggregate gradation mixtures that had unit weights less than the average unit weight, 6 out of 10 had the highest w/cm ratio (0.47), and 5 out of the 10 mixtures contained fly ash.

4.2 Testing of Hardened Concrete

This section presents the results of mechanical and durability tests performed on optimized aggregate gradation mixtures, comparing them to non-optimized aggregate gradation mixtures where appropriate. Test results are also compared to current NCDOT specification requirements.

4.2.1 Mechanical Properties

This section presents test results for the mechanical properties of optimized and non-optimized aggregate gradation mixtures, compares the results to current NCDOT specifications, and provides analysis comparing the two types of mixtures. The mixtures mechanical properties will be compared using the average percent difference, which is calculated by taking the average of the percent difference between companion non-optimized and optimized aggregate gradation mixtures. The mixtures average test result will also be taken into consideration to determine if the average percent difference should be considered negligible or not.

4.2.1.1 Compressive Strength

Compressive strength testing was performed on three cylinders at 3, 7, 28, 56, and 90 days. Averaged test results for all test dates with test results for both non-optimized and optimized aggregate gradation mixtures broken down by cementitious material content (pcy) can be found in Table 4.3. NCDOT's 2018 Standard Specifications require paving and Type AA bridge mixtures to have a minimum 28-day compressive strength of 4,500 psi. Of the 24 optimized aggregate gradation mixtures, only H-420*-180 did not meet this requirement. This mixture contained 30 percent fly ash, which is known to provide later-age strength gain later than mixtures with portland cement alone, and did, however, meet the minimum requirement at 56 days. Sections 4.2.1.1.1 – 4.2.1.1.3 will provide average compressive strength results by cementitious material comparing optimized and non-optimized aggregate gradation mixtures by w/cm ratio and fly ash rates to understand differences between the aggregate types.

Table 4.3: Compressive strength results

Mixture ID	Compressive Strength (psi)									
	Non-Optimized					Optimized				
	3 Day	7 Day	28 Day	56 Day	90 Day	3 Day	7 Day	28 Day	56 Day	90 Day
H-700-0	3,810	4,394	5,379	6,140	6,381	3,156	4,182	5,377	6,131	6,309
H-560-140	3,461	3,950	4,994	5,961	6,087	2,682	3,855	4,513	5,661	6,574
M-700-0	5,088	5,679	6,688	7,531	8,168	4,813	5,835	6,972	7,283	7,782
M-560-140	4,019	4,854	5,688	6,114	6,322	3,485	4,806	5,814	6,729	6,894
L-700-0	5,921	7,550	7,856	8,762	9,237	6,042	7,181	7,686	7,984	8,184
L-560-140	5,045	5,267	6,729	7,316	7,808	4,367	4,685	5,900	6,797	6,915
H-650-0	4,276	5,232	6,256	7,135	7,556	3,340	4,234	5,207	6,068	6,668
H-520-130	3,705	4,323	5,319	6,921	7,233	2,701	3,599	5,094	5,751	6,134
M-650-0	5,192	5,935	6,739	7,223	8,221	4,621	5,548	6,624	7,903	7,607
M-520-130	4,258	5,129	6,375	7,705	8,416	3,654	4,435	5,582	6,293	7,964
L-650-0	6,984	7,367	7,991	8,251	9,113	5,483	6,164	6,722	8,084	8,529
L-520-130	5,194	6,005	7,203	7,591	8,062	5,002	5,508	6,478	7,659	8,219
H-600-0	3,750	4,309	5,494	5,887	6,302	3,399	4,398	5,468	5,951	6,492
H-480-120	2,784	3,150	3,982	4,418	5,148	2,598	3,750	4,736	5,779	6,509
H-420-180	2,446	3,417	4,328	4,869	5,521	2,339	2,979	4,282	4,861	5,638
M-600-0	4,526	5,362	5,873	6,418	7,995	4,806	5,507	6,296	7,000	7,422
M-480-120	4,167	4,895	5,390	5,832	6,483	3,256	4,304	5,482	6,286	7,210
M-420-180	3,991	4,260	5,007	5,590	6,216	3,151	3,807	5,365	6,401	7,210
L-600-0	5,698	6,471	7,010	7,427	7,936	6,310	6,651	8,087	7,513	8,189
L-480-120	5,510	6,184	6,814	7,107	7,650	3,697	6,287	6,633	7,342	7,383
L-420-180	5,264	5,716	6,288	6,693	7,063	3,381	4,254	5,837	6,949	6,087

4.2.1.1.1 700*/700 pcy of Cementitious Material Mixtures Compressive Strength

Figure 4.1 plots the compressive strength results for pairs of optimized and non-optimized aggregate gradation mixtures, developed with 700 pcy cementitious material (which includes the 20 percent fly ash replacement mixtures). It can be observed that optimized aggregate gradation mixtures and non-optimized aggregate gradation mixtures performed similarly on most test dates and had similar variability on each test date.

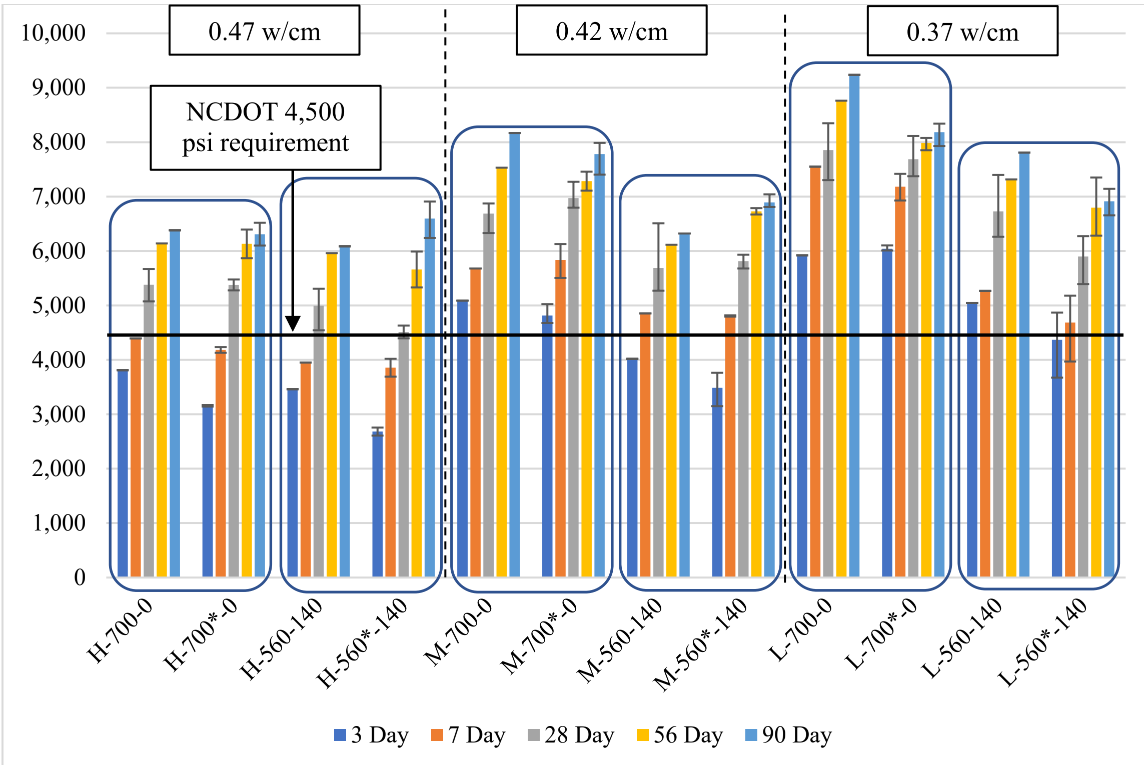

Figure 4.1: Development of average compressive strength for 700/700* pcy cementitious material mixtures

Figure 4.2 through Figure 4.4 show the strength gain for pairs of mixtures at each water cement ratio. From these plots, the similarities and differences in strength gain between pairs of optimized/non-optimized mixtures and between pairs of straight cement/fly ash replacement mixtures can be observed. It should be noted that the high (0.47) w/cm ratio mixtures were originally batched as non-optimized aggregate gradation mixtures to provide test results that are indicative of poor concrete mixtures that are not typical to NCDOT use. Optimized aggregate gradation concrete mixtures at the high w/cm ratio were batched and tested for a direct comparison between the optimized and non-optimized aggregate gradation mixtures. However, as these mixtures are representative of a higher w/cm ratio than typical NCDOT concrete mixtures, these results may not be as valuable.

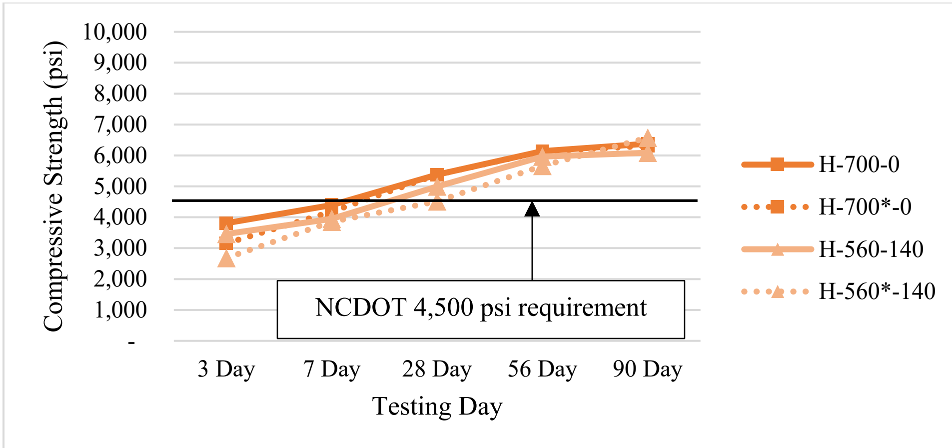

Figure 4.2: Development of average compressive strength for 700/700* pcy cementitious material mixtures at 0.47 w/cm ratio

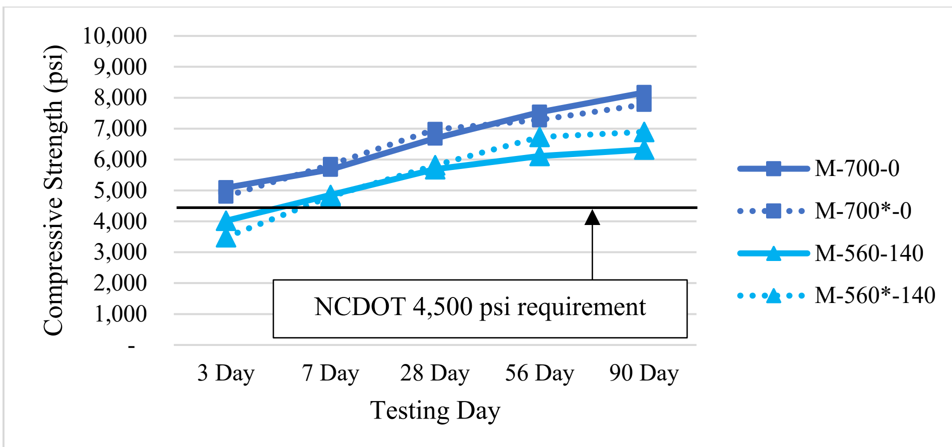

Figure 4.3: Development of average compressive strength for 700/700* pcy cementitious material mixtures at 0.42 w/cm ratio

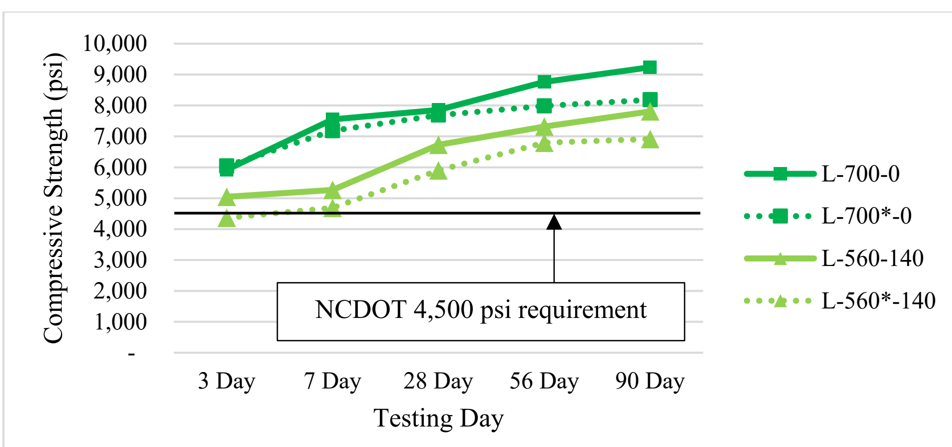

Figure 4.4: Development of average compressive strength for 700/700* pcy cementitious material mixtures at 0.37 w/cm ratio

Table 4.4 shows the average compressive strength as well as their average percent difference of average compressive strengths between 700* pcy optimized aggregate gradation mixtures when compared to their 700 pcy non-optimized companion mixtures, for both straight cement and fly ash mixtures. The average percent difference was calculated by taking the average of all mixtures percent difference between optimized and non-optimized aggregate gradation mixtures compressive strength results. 700* pcy optimized aggregate gradation mixtures exhibited an average compressive strength noticeably lower for early age (3-day) testing. However, a fairly negligible difference (less than 10 percent different on average with all mixtures meeting the 28-day compressive strength requirement of 4,500 psi) was exhibited at all other ages.

Table 4.4: Average percent difference between average compressive strength for 700* pcy optimized mixtures vs 700 pcy non-optimized mixtures

Characteristic	Mixture Type	Test Day				
		3 Day	7 Day	28 Day	56 Day	90 Day
All 700* pcy mixtures	Non-optimized	4,557	5,282	6,222	6,971	7,334
	Optimized	4,091	5,091	6,044	6,764	7,110
	Average percent difference	-14.1%	-3.9%	-3.5%	-2.8%	-2.7%
Straight cement mixtures	Non-optimized	4,940	5,874	6,641	7,478	7,929
	Optimized	4,670	5,733	6,679	7,133	7,425
	Average percent difference	-8.1%	-2.5%	0.6%	-4.4%	-6.3%
Fly ash replacement mixtures	Non-optimized	4,175	4,690	5,804	6,464	6,739
	Optimized	3,511	4,449	5,409	6,396	6,794
	Average percent difference	-20.0%	-5.3%	-7.5%	-1.3%	0.9%

Table 4.5 shows the average compressive strength as well as their average percent difference of average compressive strengths between 700* pcy optimized aggregate gradations when compared to their 700 pcy non-optimized companion mixtures grouped by w/cm ratio. As

previously described, early age (3-day) average compressive strengths were noticeably different but performed more similarly as the concrete aged. 700* pcy optimized aggregate gradation mixtures with a w/cm ratio of 0.47 and 0.42 performed more similarly to their companion non-optimized mixtures at later test dates than the 0.37 w/cm ratio optimized mixtures. However, all showed a fairly negligible average percent difference (less than 10 percent different on average with all mixtures meeting the 28-day required compressive strength of 4,500 psi) with the exception of 0.37 w/cm ratio optimized mixtures having a 90-day compressive strength an average of 12.9 percent lower than their companion non-optimized 0.37 w/cm ratio mixtures. Although greater than 10 percent, it is noted that this difference was computed based upon the optimized aggregate gradation exhibiting a 90-day average compressive strength of 7,549 psi and was deemed negligible.

Table 4.5: Average percent difference between average compressive strength for 700* pcy optimized mixtures vs 700 pcy non-optimized mixtures by w/cm ratio

w/cm ratio	Mixture Type	Test Day				
		3 Day	7 Day	28 Day	56 Day	90 Day
0.47	Non-optimized	3,636	4,172	5,187	6,051	6,234
	Optimized	2,919	4,019	4,945	5,896	6,442
	Average percent difference	-24.9%	-3.8%	-5.3%	-2.7%	3.1%
0.42	Non-optimized	4,554	5,267	6,188	6,823	7,245
	Optimized	4,149	5,321	6,393	7,006	7,338
	Average percent difference	-10.5%	0.8%	3.1%	2.9%	1.7%
0.37	Non-optimized	5,483	6,409	7,293	8,039	8,523
	Optimized	5,204	5,933	6,793	7,391	7,549
	Average percent difference	-6.8%	-8.8%	-8.1%	-8.7%	-12.9%

4.2.1.1.2 650*/650 pcy of Cementitious Material Mixtures Compressive Strength

Figure 4.5 shows the compressive strength results for pairs of optimized and non-optimized aggregate gradation mixtures, for the 650 lb straight cement mixtures (which includes the 20 percent fly ash replacement mixtures). It can be observed that optimized aggregate gradation mixtures and non-optimized aggregate gradation mixtures performed similarly on most test dates and had an expected variability on each test date.

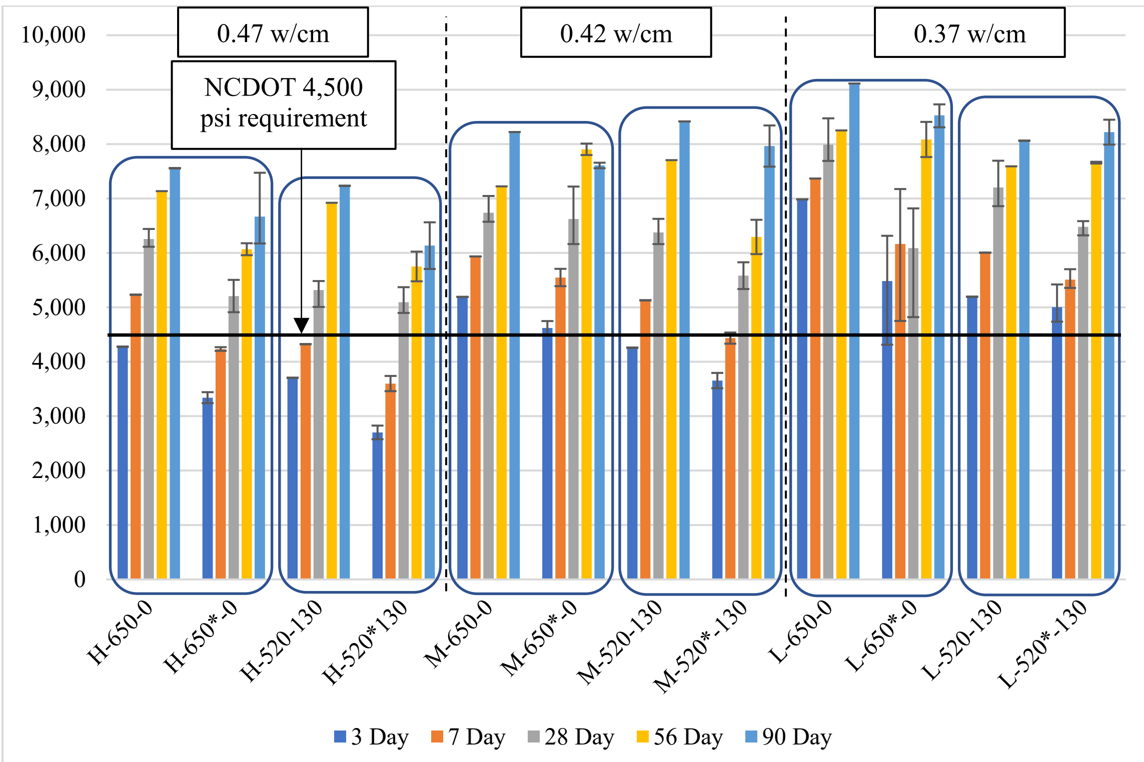

Figure 4.5: Development of average compressive strength for 650/650* pcy cementitious material mixtures

Figure 4.6 through Figure 4.8 show the strength gain for pairs of mixtures at each water cement ratio. From these plots, the similarities and differences in strength gain between pairs of optimized/non-optimized mixtures and between pairs of straight cement/fly ash replacement mixtures can be observed. It should again be noted that the high (0.47) w/cm ratio mixtures were originally batched as non-optimized aggregate gradation mixtures to provide test results that are indicative of poor concrete mixtures that are not typical to NCDOT use. Optimized aggregate gradation concrete mixtures at the high w/cm ratio were batched and tested for a direct comparison between the optimized and non-optimized aggregate gradation mixtures. However, as these mixtures are representative of a higher w/cm ratio than typical NCDOT concrete mixtures, these results may not be as valuable.

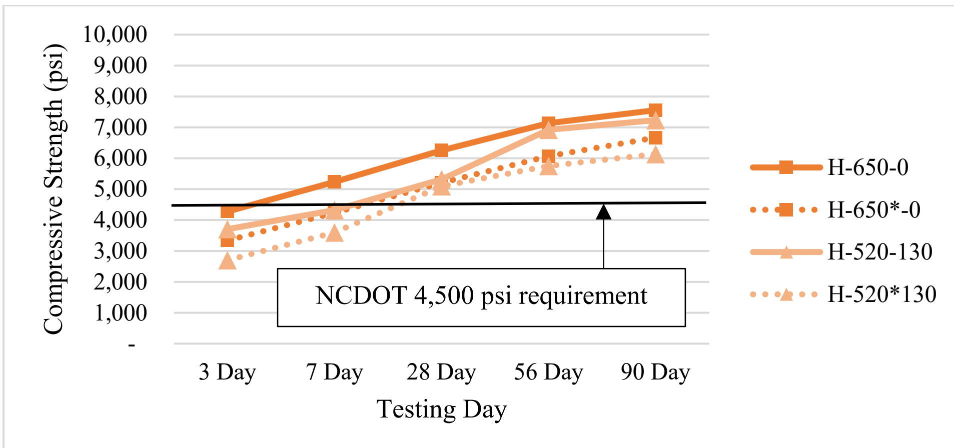

Figure 4.6: Development of average compressive strength for 650/650* pcy cementitious material mixtures at 0.47 w/cm ratio

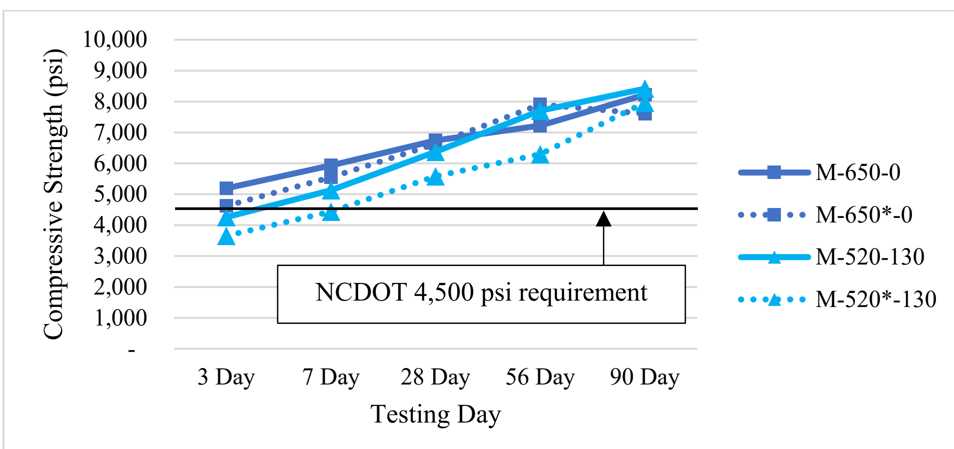

Figure 4.7: Development of average compressive strength for 650/650* pcy cementitious material mixtures at 0.42 w/cm ratio

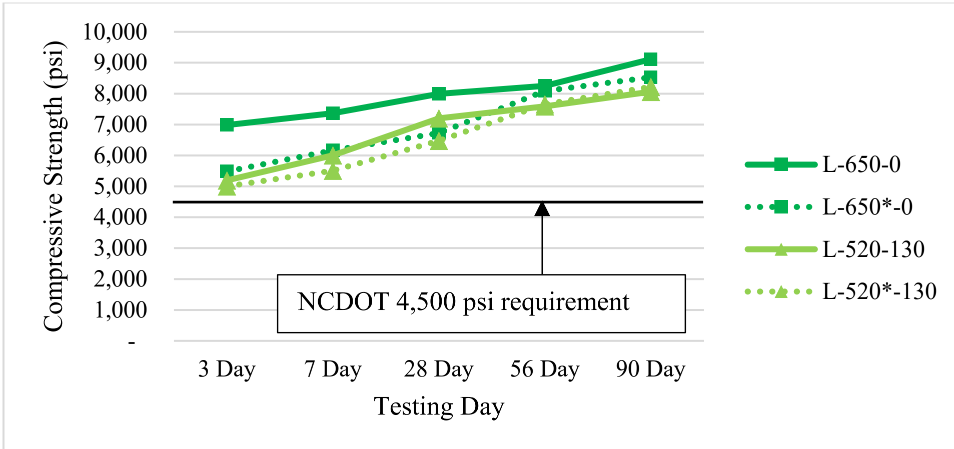

Figure 4.8: Development of average compressive strength for 650/650* pcy cementitious material mixtures at 0.37 w/cm ratio

Table 4.6 shows the average compressive strength and the average percent difference of average compressive strength for 650* pcy optimized aggregate gradation mixtures compared to their companion non-optimized 650 pcy mixtures for all mixtures, straight cement mixtures, and fly ash mixtures. The trend previously described where optimized aggregate gradation mixtures exhibited an average compressive strength noticeably lower than their companion non-optimized mixtures at an early age is present at the 3- and 7-day tests. Straight cement and fly ash optimized aggregate gradation mixtures exhibited a fairly negligible difference at almost all other testing dates (less than 10 average percent difference and all mixtures 28-day average compressive strength exceeding 5,000 psi).

The 28-day test results for straight cement optimized aggregate gradation mixtures had more than a 10 percent difference when from companion non-optimized aggregate gradation mixtures. However, the 28-day average compressive strength for optimized aggregate gradation mixtures was 6,185 psi, certainly acceptable per the NCDOT Standard Specifications target of 4,500 psi. Optimized aggregate fly ash replacement mixtures were also over 10 percent different from 56-day test results when compared to their companion non-optimized aggregate gradation mixtures. However, the average 56-day compressive strength of 6,568 for optimized aggregate gradation mixtures is also far above the current NCDOT target. All 650* pcy optimized gradation fly ash mixtures did meet the NCDOT 28-day required compressive strength of 4,500 psi.

Table 4.6: Average percent difference between average compressive strength for 650* pcy optimized mixtures vs 650 pcy non-optimized mixtures

Characteristic	Mixture Type	Test Day				
		3 Day	7 Day	28 Day	56 Day	90 Day
All 650* pcy mixtures	Non-optimized	4,935	5,665	6,647	7,471	8,100
	Optimized	4,133	4,915	5,951	6,960	7,520
	Average percent difference	-20.9%	-15.8%	-11.8%	-8.8%	-8.3%
Straight cement mixtures	Non-optimized	5,484	6,178	6,995	7,536	8,297
	Optimized	4,481	5,315	6,185	7,352	7,601
	Average percent difference	-22.6%	-16.7%	-13.6%	-3.7%	-9.4%
Fly ash replacement mixtures	Non-optimized	4,386	5,152	6,299	7,406	7,904
	Optimized	3,786	4,514	5,718	6,568	7,439
	Average percent difference	-19.2%	-14.9%	-9.9%	-14.0%	-7.2%

Table 4.7 shows the average compressive strength and the average percent difference of average compressive strengths between 650* pcy optimized aggregate gradations when compared to their 650 pcy non-optimized companion mixtures grouped by w/cm ratio. The previously described trend where the average compressive strength of optimized gradation mixtures is noticeably lower than non-optimized aggregate gradation companion mixes at early test days (3- and 7-day tests) is present. The trend continues for medium and low w/cm ratio mixtures, with the remaining test days average compressive strength being negligible (less than 10 percent) with the exception of the low w/cm ratio mixtures 28-day test. It should be noted that this percent difference was computed with optimized aggregate gradation mixtures having an

average compressive strength of 6,600 psi and was deemed negligible. High w/cm ratio optimized aggregate gradation mixtures did not perform as well as their companion non-optimized mixtures for all test dates, but did still meet the NCDOT 28-day compressive strength requirement of 4,500 psi. It should again be noted that the high w/cm ratio concrete mixtures were batched to provide test results for a range of w/cm ratios, with these mixtures having a w/cm ratio outside of what is typically used by the NCDOT.

Table 4.7: Average percent difference between average compressive strength for 650* pcy optimized mixtures vs 650 pcy non-optimized mixtures by w/cm ratio

w/cm ratio	Mixture Type	Test Day				
		3 Day	7 Day	28 Day	56 Day	90 Day
0.47	Non-optimized	3,991	4,778	5,788	7,028	7,395
	Optimized	3,021	3,917	5,151	5,909	6,401
	Average percent difference	-32.6%	-21.8%	-12.3%	-19.0%	-15.6%
0.42	Non-optimized	4,725	5,532	6,557	7,464	8,319
	Optimized	4,137	4,991	6,103	7,098	7,785
	Average percent difference	-14.4%	-11.3%	-8.0%	-6.9%	-6.9%
0.37	Non-optimized	6,089	6,686	7,597	7,921	8,588
	Optimized	5,242	5,836	6,600	7,872	8,374
	Average percent difference	-15.6%	-14.3%	-15.0%	-0.6%	-2.5%

4.2.1.1.3 600*/600 pcy of Cementitious Material Mixtures Compressive Strength

Figure 4.9 shows the compressive strength results for pairs of optimized and non-optimized aggregate gradation mixtures, for the 600 lb straight cement mixtures (which includes the 20 percent fly ash replacement mixtures). It can be observed that optimized aggregate gradation mixtures and non-optimized aggregate gradation mixtures performed similarly on most test dates and had an expected variability on each test date.

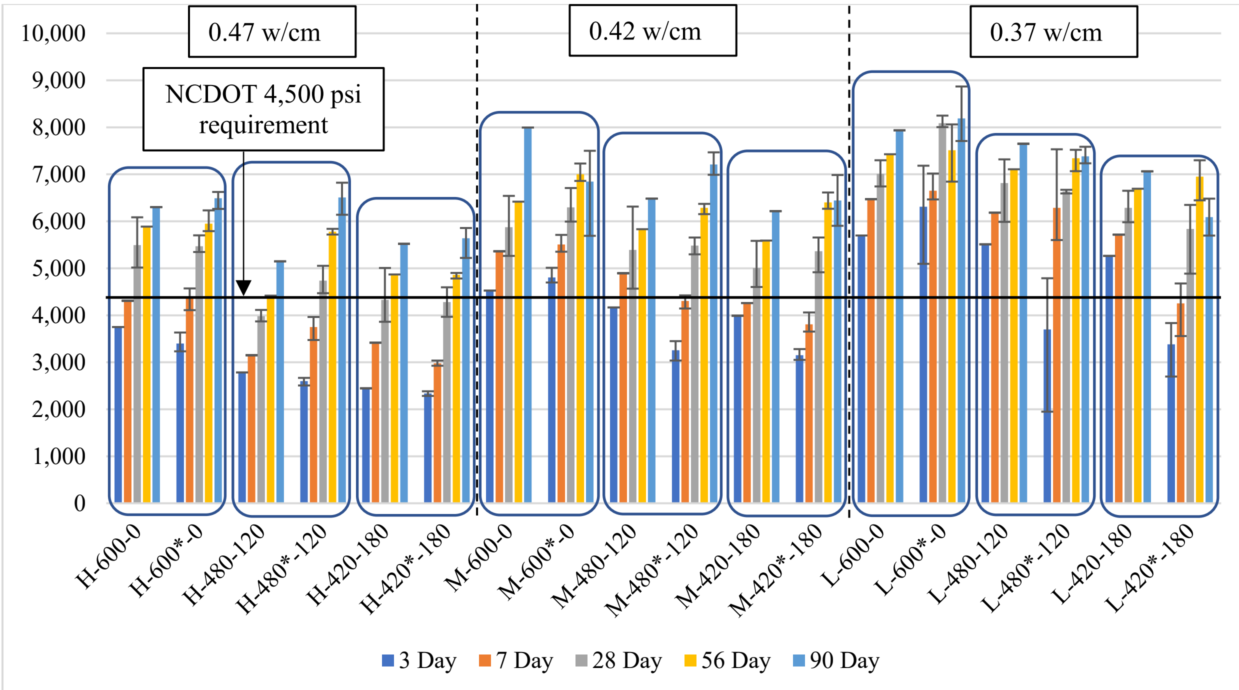

Figure 4.9: Development of average compressive strength for 600/600* pcy cementitious material mixtures

Figure 4.10 through Figure 4.12 show the strength gain for pairs of mixtures at each water cement ratio. From these plots, the similarities and differences in strength gain between pairs of optimized/non-optimized mixtures and between pairs of straight cement/fly ash replacement mixtures can be observed. Figure 4.10 shows the 600* pcy optimized aggregate gradation mixtures and 600 pcy non-optimized aggregate gradation mixtures at a high w/cm ratio and contains the only mixtures to not meet to 28-day NCDOT required 28-day compressive strength of 4,500 psi (H-480-120, H-420-180, and H-420*-180). It should again be noted that the high (0.47) w/cm ratio mixtures were originally batched as non-optimized aggregate gradation mixtures to provide test results that are indicative of poor concrete mixtures that are not typical to NCDOT use. Optimized aggregate gradation concrete mixtures at the high w/cm ratio were batched and tested for a direct comparison between the optimized and non-optimized aggregate gradation mixtures. However, as these mixtures are representative of a higher w/cm ratio than typical NCDOT concrete mixtures, these results may not be as valuable.

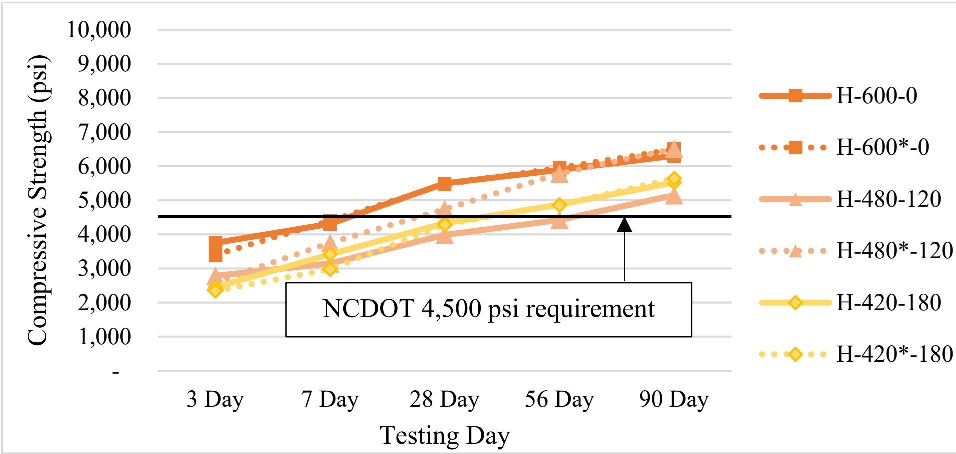

Figure 4.10: Development of average compressive strength for 600/600* pcy cementitious material mixtures at 0.47 w/cm ratio

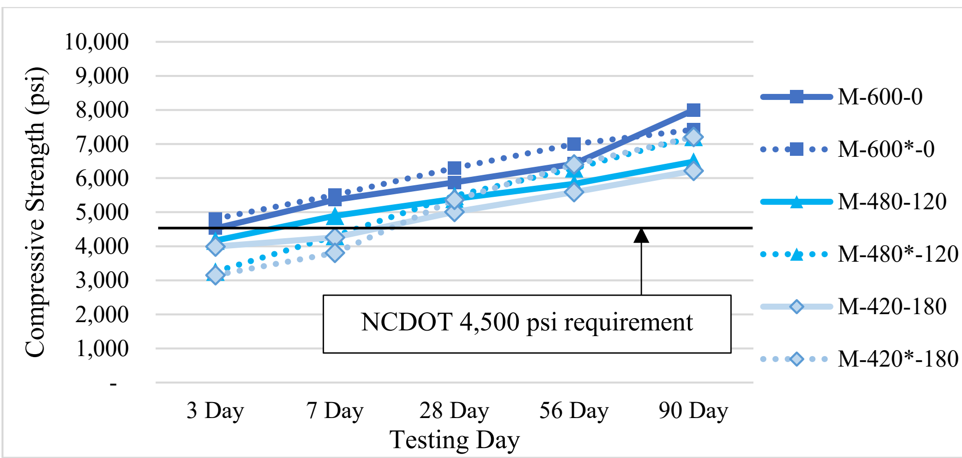

Figure 4.11: Development of average compressive strength for 600/600* pcy cementitious material mixtures at 0.42 w/cm ratio

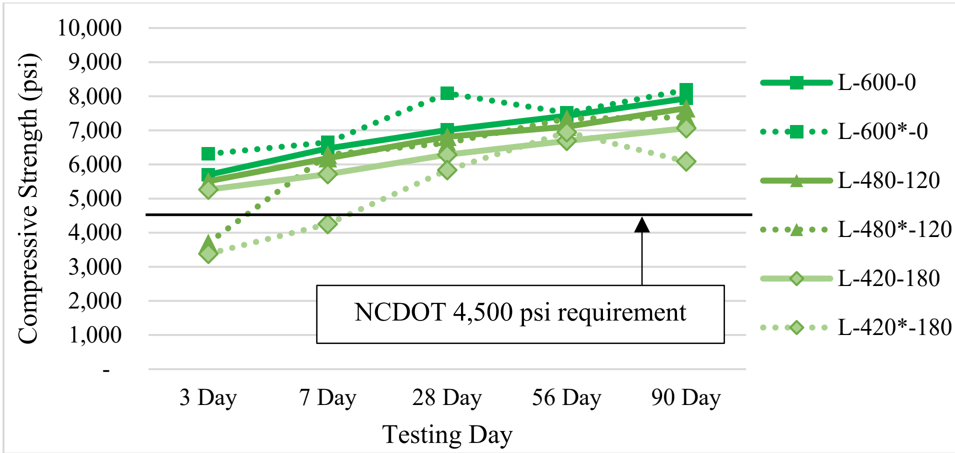

Figure 4.12: Development of average compressive strength for 600/600* pcy cementitious material mixtures at 0.37 w/cm ratio

Table 4.8 shows the average compressive strength and average percent difference of average compressive strength for 600* pcy optimized aggregate gradation mixtures compared to their companion non-optimized 600 pcy mixtures for all 600* pcy mixtures, 600* pcy straight mixtures, and 600* fly ash mixtures. The previously described trend where early age (3-day) strength of optimized aggregate gradation mixtures is noticeably lower than their companion non-optimized mixtures is present in optimized aggregate gradation fly ash mixtures. Optimized straight cement mixtures exhibited average compressive strengths slightly higher than their companion non-optimized mixtures at early age test dates (1.7 percent). For all other test dates for both straight cement mixtures and fly ash replacement mixtures there was a negligible difference in average compressive strength (less than 10 percent different on average and an average difference less than 170 psi for ages 7-days and older).

Table 4.8: Average percent difference between average compressive strength for 600* pcy optimized mixtures vs 600 pcy non-optimized mixtures

Characteristic	Mixture Type	Test Day				
		3 Day	7 Day	28 Day	56 Day	90 Day
All 600* pcy mixtures	Non-optimized	4,237	4,863	5,576	6,027	6,702
	Optimized	3,660	4,660	5,798	6,454	6,905
	Average percent difference	-18.4%	-5.5%	3.6%	6.7%	2.8%
Straight cement mixtures	Non-optimized	4,658	5,381	6,126	6,577	7,411
	Optimized	4,838	5,519	6,617	6,821	7,368
	Average percent difference	1.7%	2.5%	6.5%	3.5%	-0.6%
Fly ash replacement mixtures	Non-optimized	4,027	4,604	5,302	5,752	6,347
	Optimized	3,070	4,230	5,389	6,270	6,673
	Average percent difference	-28.5%	-9.5%	2.1%	8.4%	4.5%

Table 4.9 shows the average compressive strengths as well as the average percent difference of average compressive strengths between 600* pcy optimized aggregate gradations when compared to their 600 pcy non-optimized companion mixtures segmented by w/cm ratio. The previously described trend where early age (3-day) strength of optimized aggregate gradation mixtures is noticeably lower than their companion non-optimized mixtures is present. As the concrete aged, 600* pcy optimized aggregate gradations compressive strength showed a negligible difference when compared to their companion 600 pcy non-optimized mixtures (less than 10 percent different on average and an average difference less than 335 psi for ages 7-days and older).

H-420*-180 was the only optimized mixture to not meet the NCDOT 28-day requirement of 4,500 psi. However, it did meet this requirement by the requirement by the 56-day test. It should be noted, this mixture is at the highest w/cm ratio where lower compressive strengths are expected, as well as has 30 percent fly ash replacement which is known to reach actual compressive strengths later than non-fly ash mixtures. Additionally, the non-optimized aggregate gradation mixtures H-480-120 and H-420-180 did not meet the NCDOT 28-day requirement of 4,500 psi.

Table 4.9: Average percent difference between average compressive strength for 600* pcy optimized mixtures vs 600 non-optimized mixtures by w/cm ratio

w/cm ratio	Mixture Type	Test Day				
		3 Day	7 Day	28 Day	56 Day	90 Day
0.47	Non-optimized	2,993	3,625	4,601	5,058	5,657
	Optimized	2,778	3,709	4,829	5,530	6,213
	Average percent difference	-7.4%	1.1%	4.8%	8.2%	8.6%
0.42	Non-optimized	4,228	4,839	5,423	5,947	6,898
	Optimized	3,738	4,539	5,714	6,563	7,281
	Average percent difference	-16.3%	-7.7%	5.0%	9.4%	5.4%
0.37	Non-optimized	5,491	6,124	6,704	7,076	7,550
	Optimized	4,463	5,731	6,852	7,268	7,220
	Average percent difference	-31.7%	-10.0%	1.0%	2.7%	-5.5%

4.2.1.2 Modulus of Rupture (MOR)

Modulus of rupture testing was performed at 28-days for pavement mixtures only, and results for optimized aggregate gradation and non-optimized aggregate gradation mixtures are presented in Table 4.10. Optimized aggregate gradation mixture 28-day MOR ranged from 581 psi to 840 psi, with an average of 715 psi, while the 28-day MOR of non-optimized aggregate gradation mixtures ranged from 715 psi to 822 psi, with an average of 766 psi. All optimized aggregate gradation mixtures MOR test results were higher than the NCDOT required 28-day MOR of 650 psi, except for mixtures H-420*-180 and M-420*-120. The two mixtures that did not meet the requirement contained 30 percent fly ash, and significant additional strength gain after 28 days could be expected. H-420*-180 also did not meet the 28-day compressive strength requirement by the NCDOT but did by the 56-day test. As fly ash mixtures are known to gain strength later than straight cement mixtures, these mixtures may have met the requirement by a later testing date.

It should again be noted that the high (0.47) w/cm ratio mixtures were originally batched as non-optimized aggregate gradation mixtures to provide test results that are indicative of poor concrete mixtures that are not typical to NCDOT use. Optimized aggregate gradation concrete mixtures at the high w/cm ratio were batched and tested for a direct comparison between the optimized and non-optimized aggregate gradation mixtures. However, as these mixtures are representative of a higher w/cm ratio than typical NCDOT concrete mixtures, these results may not be as valuable.

Table 4.10: MOR, MOE, and Poisson's Ratio

Mixture ID	Modulus of Rupture (psi)		Modulus of Elasticity (psi)		Poisson's Ratio	
	Non-Optimized	Optimized	Non-Optimized	Optimized	Non-Optimized	Optimized
H-700-0	-	-	3,045,000	3,266,000	0.21	0.21
H-560-140	-	-	2,675,000	2,894,000	0.20	0.20
H-650-0	-	-	3,650,000	3,862,000	0.21	0.20
H-520-130	-	-	3,056,000	3,349,000	0.23	0.24
H-600-0	745	720	2,980,000	3,733,000	0.19	0.19
H-480-120	808	704	2,527,000	3,230,000	0.20	0.22
H-420-180	724	581	2,461,000	2,995,000	0.22	0.20
M-700-0	-	-	3,569,000	3,975,000	0.24	0.18
M-560-140	-	-	3,363,000	4,260,000	0.18	0.18
M-650-0	-	-	3,706,000	3,842,000	0.20	0.14
M-520-130	-	-	3,620,000	3,921,000	0.20	0.18
M-600-0	822	748	3,398,000	4,294,000	0.21	0.17
M-480-120	726	683	3,076,000	3,942,000	0.20	0.20
M-420-180	726	637	3,131,000	3,700,000	0.19	0.20
L-700-0	-	-	3,826,000	3,838,000	0.17	0.15
L-560-140	-	-	3,656,000	4,492,000	0.20	0.19
L-650-0	-	-	4,317,000	4,588,000	0.19	0.17
L-520-130	-	-	3,632,000	3,992,000	0.21	0.15
L-600-0	817	840	3,761,000	4,932,000	0.19	0.19
L-480-120	718	808	3,087,000	3,949,000	0.22	0.17
L-420-180	815	713	3,241,000	3,942,000	0.20	0.17

Figure 4.13 shows the MOR test results for all optimized aggregate gradation mixtures (orange bars) and non-optimized companion mixtures (blue bars). The results in Figure 4.13 are grouped by fly ash content to highlight the impact of fly ash on the MOR test results of both optimized aggregate gradation mixtures and non-optimized aggregate gradations. Figure 4.14 shows the MOR of optimized gradation mixtures and non-optimized gradation mixtures grouped by w/cm ratio.

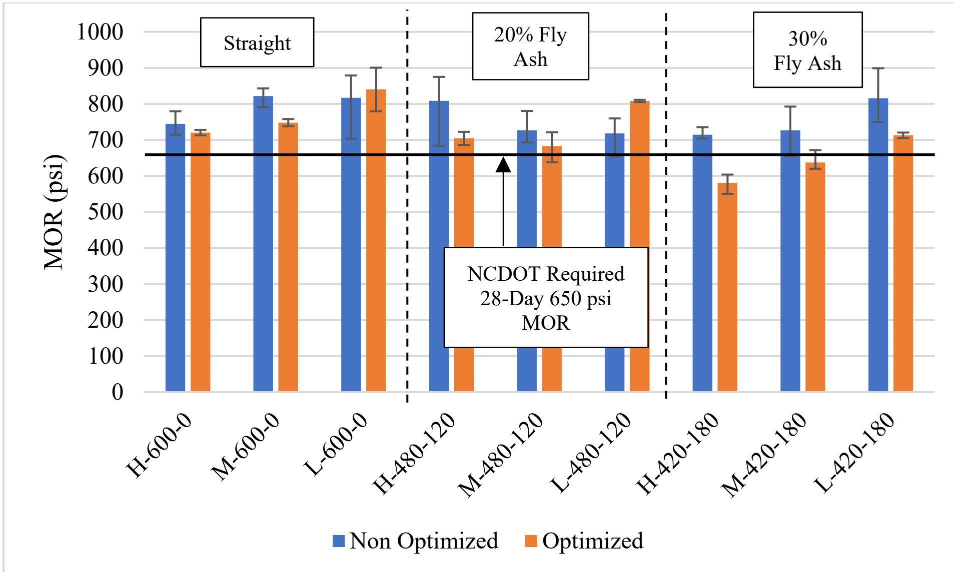

Figure 4.13: MOR of optimized and non-optimized aggregate gradation mixtures by fly ash content

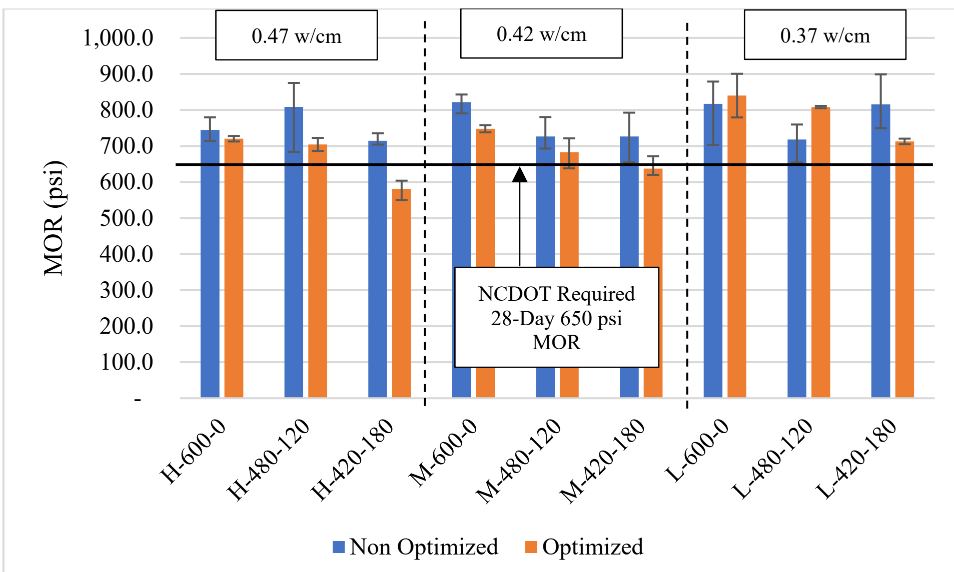

Figure 4.14: MOR of optimized and non-optimized aggregate gradation mixtures by w/cm ratio

Table 4.11 shows optimized aggregate gradation mixtures average percent difference in MOR values for straight cement mixtures and fly ash mixtures when compared to companion non-optimized 20 percent fly ash replacement mixtures, companion optimized straight cement mixtures, and their companion non-optimized straight cement mixtures. Optimized aggregate gradation mixtures with 20 percent fly ash replacement showed a negligible difference (while meeting the NCDOT 2-day required MOR of 650 psi) when compared with: companion non-optimized 20 percent fly ash replacement mixtures, companion optimized straight cement mixtures, and companion non-optimized straight cement mixtures. Optimized aggregate gradation mixtures with 30 percent fly ash replacement did, however, show a significant decrease in their MOR values when compared to all of their companion mixtures.

As mentioned previously, two optimized 30 percent fly ash replacement mixtures did not meet the NCDOT 28-day requirement of 650 psi: H-420*-180 with an MOR of 581 psi and M-420*-180 with an MOR of 637. As the mixtures that contain the highest rate of fly ash replacement, it is possible they may have reached the required MOR of 650 psi at a later date due to the delayed strength gain attributed to fly ash mixtures. It also should be noted that while these mixtures did not meet the NCDOT 28-day requirement, they did meet the 28-day recommended requirement in AASHTO PP 84 of 600 psi (AASHTO 2020).

Table 4.11: Average percent difference between optimized aggregate gradation mixtures MOR and companion mixtures

Optimized mixture characteristic	Non-optimized companion mixture	Optimized straight cement companion mixture	Non-optimized straight cement companion mixture
Straight cement mixtures	-3.5%	-	-
20% fly ash replacement mixtures	-3.3%	-5.2%	-9.0%
30% fly ash replacement mixtures	-17.1%	-19.7%	-23.9%

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

MOE results of both optimized aggregate gradation and non-optimized aggregate gradation mixtures are provided in. The 28-day MOE test results for optimized aggregate gradation mixtures ranged from 2,893,826 psi to 4,931,577 psi with an average of 3,839,444 psi, compared to the 28-day MOE test results for non-optimized aggregate gradation mixtures, which ranged from 2,461,178 psi to 4,317,210 psi with an average of 3,322,681 psi. As could be expected, the mixtures with the lowest w/cm ratio had MOE values relatively higher than the mixtures with the highest w/cm ratio. MOE values for optimized aggregate gradation mixtures tended to be higher than their companion-non-optimized mixtures. Figure 4.15 shows the results of the modulus of elasticity (MOE) tests for all pairs of non-optimized (blue bars) and optimized (orange bars) mixtures and is grouped by w/cm ratio of the mixtures.

It should again be noted that the high (0.47) w/cm ratio mixtures were originally batched as non-optimized aggregate gradation mixtures to provide test results that are indicative of poor concrete mixtures that are not typical to NCDOT use. Optimized aggregate gradation concrete mixtures at the high w/cm ratio were batched and tested for a direct comparison between the optimized and non-optimized aggregate gradation mixtures. However, as these mixtures are representative of a higher w/cm ratio than typical NCDOT concrete mixtures, these results may not be as valuable.

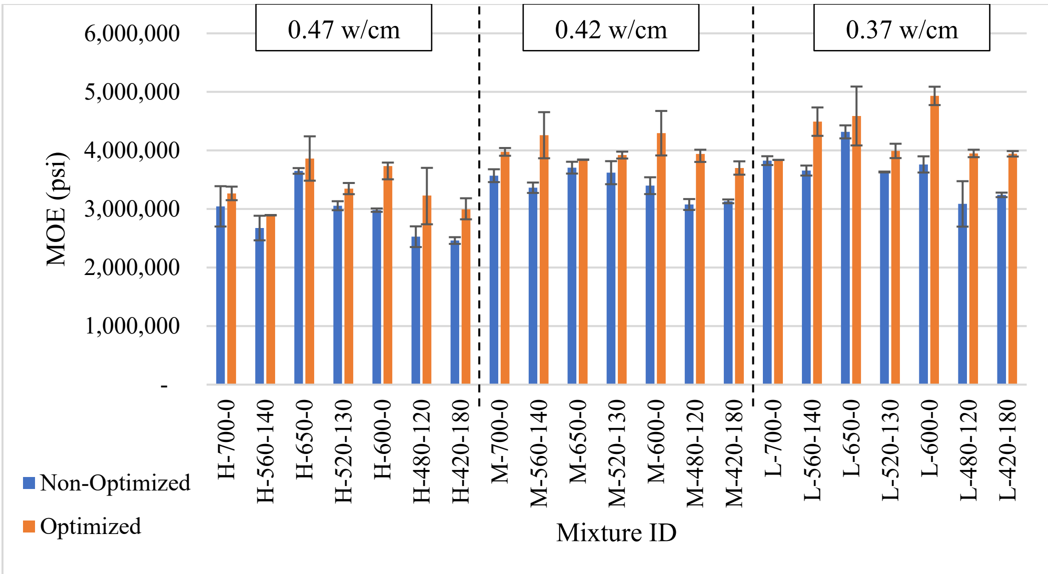

Figure 4.15: Optimized and non-optimized aggregate gradation mixtures 28-day MOE by w/cm ratio

Figure 4.16 displays the measured MOE values for both optimized and non-optimized mixtures plotted against their 28-day compressive strength in comparison with the ACI 318 calculated MOE value using equation 19.2.2.1b (Equation 4.1) and the MOE calculated using AASHTO LFRD equation C5.4.2.4-2 (Equation 4.2) (AASHTO 2017, ACI 2019). This figure displays optimized aggregate gradation mixtures and non-optimized aggregate gradations 28-day MOE values were lower than the calculated values using ACI 318, which is a finding similar to other concrete studies performed by the research team. The lower MOE's when compared to the calculated values may be attributed to either user error when viewing the dial gauges during loading. Additionally, studies have shown that the type of aggregate used in concrete mixtures can affect the MOE (Beuhausen and Ditmer 2015).

$$E_c = 57{,}000 * \sqrt{f_c'} \qquad \text{Equation 4.1}$$

$$E_c = 33{,}000 * K_1 * w_c^{1.5} * \sqrt{f_c'} \qquad \text{Equation 4.2}$$

$$K_1 = correction\ factor\ for\ source\ of\ aggregate \qquad \text{Equation 4.3}$$

$$w_c = unit\ weight\ of\ concrete\ (kcf) \qquad \text{Equation 4.4}$$

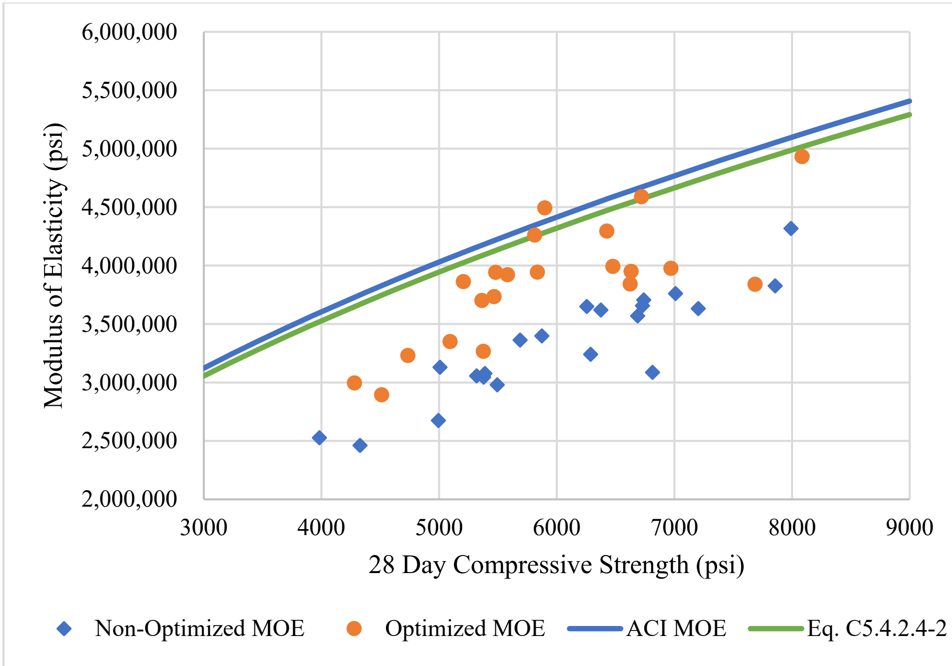

Figure 4.16: All optimized mixtures percent different than non-optimized companion mixture and ACI calculated MOE

Table 4.12 presents the average percent difference between the measured 28-day MOE values compared to their companion non-optimized companion mixtures, the ACI 318 calculated MOE and the AASHTO calculated MOE, grouped by w/cm ratio and by cementitious content. As the w/cm ratio decreased in optimized aggregate gradation mixtures, their 28-day MOE values became more similar to the calculated MOE using both ACI 318 and AASHTO LRFD equations but did not change significantly when compared to their companion non-optimized aggregate gradation mixtures. As cementitious content decreased in optimized aggregate gradation mixtures, their measured 28-day MOE became more similar to the calculated MOE values using ACI 318 and ASHTO LRFD equations.

All optimized gradation mixtures had measured 28-day MOE values an average of 13.6 percent higher than their companion non-optimized gradation mixture. As the w/cm ratio decreased in optimized aggregate gradation mixtures the average percent difference remained fairly consistent. As the cementitious content decreased in optimized aggregate gradation mixtures there was no trend in MOE change observed. However, medium and low cementitious content optimized aggregate gradation mixtures had the highest 28-day MOE, which could be expected (as shown in Table 4.13), while non-optimized aggregate gradation mixtures with low cementitious content mixtures had the lowest 28-day MOE.

Table 4.12: Average percent difference in MOE between optimized mixtures, non-optimized companion mixtures, and calculated per ACI 318 by w/cm ratio and cementitious content

Optimized mixture characteristic	Average % difference vs non-optimized companion mixture	Average % difference vs ACI 318 calculated MOE	Average % difference vs AASHTO calculated MOE
All mixtures	13.6%	-14.3%	-13.9%
0.47 w/cm ratio	12.6%	-21.0%	-19.8%
0.42 w/cm ratio	14.4%	-10.9%	-10.8%
0.37 w/cm ratio	13.9%	-10.9%	-11.0%
700* pcy of cementitious material	10.8%	-18.3%	-17.4%
650* pcy of cementitious material	6.7%	-12.4%	-12.1%
600* pcy of cementitious material	20.1%	-12.9%	-12.7%

Table 4.13: Average 28-day MOE of optimized aggregate gradation mixtures and non-optimized aggregate gradation mixtures by cementitious content

Cementitious content (pcy)	Mixture Characteristic	Non-optimized	Optimized
700	All Mixtures	3,355,667	3,787,500
	Straight Cement	3,480,000	3,693,000
	Fly Ash Replacement	3,231,333	3,882,000
650	All Mixtures	3,663,500	3,925,667
	Straight Cement	3,891,000	4,097,333
	Fly Ash Replacement	3,436,000	3,754,000
600	All Mixtures	3,073,556	3,857,444
	Straight Cement	3,379,667	4,319,667
	Fly Ash Replacement	2,920,500	3,626,333

Table 4.14 shows the percent difference in measured MOE test results between optimized aggregate gradation straight cement and fly ash mixtures when compared to: companion non-optimized mixtures, companion optimized straight cement mixtures, companion non-optimized straight cement mixtures, and the calculated MOE using ACI 318 and AASHTO LRFD equations. Optimized aggregate gradation fly ash mixtures performed as expected, with an increase in fly ash replacement causing lower 28-day MOE values when compared to all mixtures.

Table 4.14: Average percent difference in 28-day MOE between all optimized fly ash replacement mixtures and their companion mixture by fly ash replacement rat

Optimized mixture characteristic	Average % difference vs non-optimized companion mixture	Average % difference vs companion optimized straight cement mixture	Average % difference vs companion non-optimized straight cement mixture	Average % difference vs ACI 318 calculated MOE	Average % difference vs AASHTO calculated MOE
Straight cement mixtures	10.8%	-	-	-14.4%	-14.0%
All optimized fly ash mixtures	15.8%	-11.3%	4.5%	-14.2%	-13.8%
20% fly ash replacement	12.6%	-6.3%	3.7%	-11.2%	-10.8%
30% fly ash replacement	17.0%	-21.9%	4.4%	-15.9%	-15.6%

Poisson's ratios for both optimized aggregate gradation and non-optimized aggregate gradation mixtures are shown in Table 4.10. Optimized aggregate gradation mixtures exhibited Poisson's ratios in the range of 0.14 to 0.24, with an average of 0.19. Non-optimized aggregate gradation mixtures exhibited Poisson's ratios that ranged of 0.17 to 0.24 with an average of 0.20. Figure 4.17 shows the Poisson's ratio for all pairs of non-optimized aggregate gradation mixtures (blue bars) and optimized (orange bars) mixtures, with mixtures grouped by the w/cm ratio of the mixtures. It can be observed that measured Poisson's ratios for optimized aggregate gradation mixtures and non-optimized aggregate gradation mixtures were similar on most test dates and had an expected variability on each test date.

It should again be noted that the high (0.47) w/cm ratio mixtures were originally batched as non-optimized aggregate gradation mixtures to provide test results that are indicative of poor concrete mixtures that are not typical to NCDOT use. Optimized aggregate gradation concrete mixtures at the high w/cm ratio were batched and tested for a direct comparison between the optimized and non-optimized aggregate gradation mixtures. However, as these mixtures are representative of a higher w/cm ratio than typical NCDOT concrete mixtures, these results may not be as valuable.

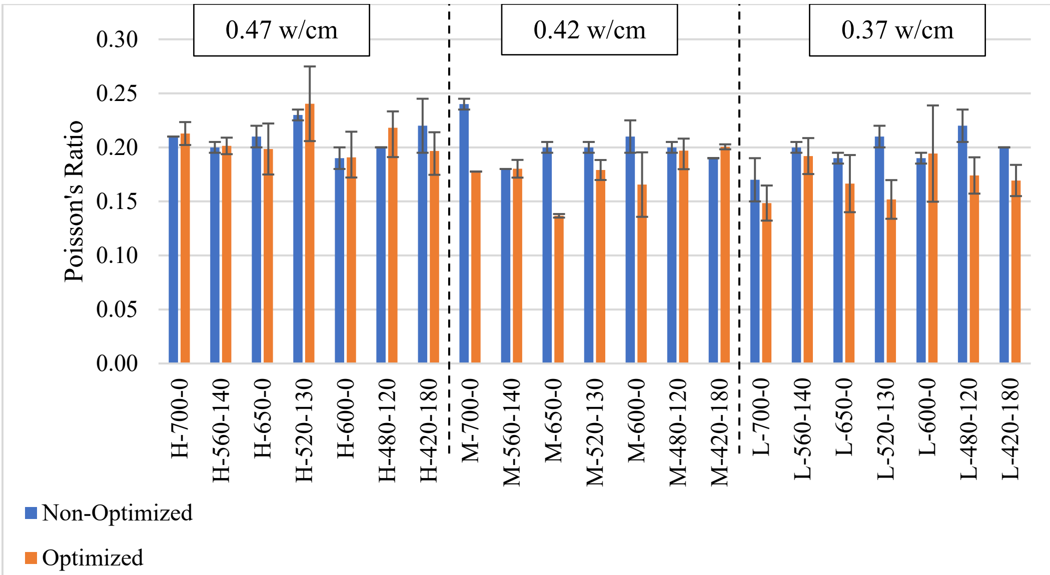

Figure 4.17: Poisson's ratio for optimized and non-optimized mixtures at 28-days

Table 4.15 shows optimized aggregate gradation mixtures 28-day Poisson's ratios compared with the 28-day Poisson's ratios for their companion non-optimized mixtures, grouped by w/cm ratio and cementitious content (pcy). As the w/cm ratio of optimized aggregate gradation mixtures decreased, the average Poisson's ratio for each w/cm ratio decreased from 0.21 to 0.17 while the average Poisson's ratio for non-optimized aggregate gradation mixtures decreased from 0.21 to 0.19.

Optimized gradation mixtures with a cementitious material content of 700* pcy and 600* pcy had average Poisson's ratios negligibly different (less than 10 percent different) when compared to companion non-optimized gradation mixtures, while optimized aggregate gradation mixtures with 650* pcy of cementitious material were noticeably different. The percent difference for 650/650* pcy mixtures could be skewed, as the two lowest Poisson's ratios for optimized aggregate gradation mixtures (M-650*-0, 0.14; and L-520*-130, 0.15) were from the 650*/650 pcy cementitious material content.

Table 4.15: Average percent difference between Poisson's ratios for optimized mixtures and companion non-optimized mixtures

Optimized mixture characteristic	Average % difference in Poisson's ratio vs non-optimized companion mixture
All mixtures	-11.1%
0.47 w/cm ratio	-0.4%
0.42 w/cm ratio	-16.6%
0.37 w/cm ratio	-16.2%
700* pcy of cementitious material	-8.6%
650* pcy of cementitious material	-18.7%
600* pcy of cementitious material	-7.6%

Table 4.16 shows the average percent difference between the measured Poisson's ratios of optimized aggregate gradation fly ash replacement mixtures compared to their companion non-optimized mixtures, companion optimized straight cement mixtures, and their companion non-optimized straight cement mixtures. Optimized aggregate gradation fly ash mixtures all exhibited with a negligible difference (less than 10 percent) in measured Poisson's ratios when compared with their companion mixtures.

Table 4.16: Average percent difference in Poisson's ratios between all optimized fly ash replacement mixtures and their companion mixtures, grouped by fly ash replacement rate

Optimized mixture characteristic	Average % difference in Poisson's ratio vs non-optimized companion mixture	Average % difference in Poisson's Ratio vs companion optimized straight cement mixture	Average % difference in Poisson's Ratio vs companion non-optimized straight cement mixture
Straight cement mixtures	-15.4%	-	-
All optimized fly ash mixtures	-7.8%	6.0%	-5.6%
20% fly ash replacement	-6.3%	6.1%	-4.8%
30% fly ash replacement	-8.3%	1.9%	-4.5%

4.2.2 Durability Performance Testing

This section provides an overview of the durability testing results including surface resistivity, rapid chloride penetration, shrinkage, and formation factor, comparing optimized aggregate gradation mixtures and non-optimized aggregate gradation mixtures where applicable. Test results are also compared to proposed specification targets for these tests proposed by AASHTO PP 84, previous research studies for NCDOT, and other published values corresponding to different performance levels. The mixtures durability performance will be compared using the average percent difference, which is calculated by taking the average of the percent difference between companion non-optimized and optimized aggregate gradation mixtures. The mixtures average test result will also be taken into consideration to determine if the average percent difference should be considered negligible or not.

It should again be noted that the high (0.47) w/cm ratio mixtures were originally batched as non-optimized aggregate gradation mixtures to provide test results that are indicative of poor concrete mixtures that are not typical to NCDOT use. Optimized aggregate gradation concrete mixtures at the high w/cm ratio were batched and tested for a direct comparison between the optimized and non-optimized aggregate gradation mixtures. However, as these mixtures are representative of a higher w/cm ratio than typical NCDOT concrete mixtures, these results may not be as valuable.

4.2.2.1 Surface Resistivity

Surface resistivity testing was performed on three concrete cylinder specimens aged 3, 7, 28, 56, and 90 days. Average surface resistivity for the optimized aggregate gradation mixture and non-optimized aggregate gradation mixture specimens tested at each age are shown in Table 4.17, along with results for rapid chloride ion permeability testing (RCPT). Surface resistivity test results for optimized aggregate gradation mixtures tended to be lower than their companion non-optimized aggregate gradation mixtures. This is not necessarily mean less permeable mixtures and could be a function of reduced cement content, the increased volume of interfacial transition zone (ITZ) offered by the increased aggregate volume, the 10% paste reduction, or some other reason.

Table 4.17: Surface resistivity and RCPT results for optimized aggregate gradation and non-optimized aggregate gradation mixtures

Mixture ID	Non-optimized mixtures - Surface Resistivity (kΩ*cm)		Optimized mixtures - Surface Resistivity (kΩ*cm)		Non-optimized mixtures – RCPT (coulombs passed)		Optimized mixtures – RCPT (coulombs passed)	
	28 Day	90 Day	28 Day	90 Day	28 Day	90 Day	28 Day	90 Day
H-700-0	7.3	14.0	8.0	8.2	4253	3070	6976	5080
H-560-140	6.6	18.8	7.1	13.1	3860	2118	7067	3407
H-650-0	8.7	9.8	6.8	8.1	4687	4018	6538	4832
H-520-130	10.6	21.8	6.2	12.3	4480	2879	7746	3575
H-600-0	8.1	17.6	8.4	9.6	4159	3439	6208	4922
H-480-120	9.5	17.1	7.0	13.5	3766	2266	7204	3358
H-420-180	11.2	20.7	6.0	15.9	3571	1980	6699	3148
M-700-0	10.9	12.5	8.6	11.5	4479	3822	5261	4275
M-560-140	6.4	18.4	7.5	16.8	4354	2148	6930	3356
M-650-0	10.7	11.9	8.6	12.7	3506	3008	5580	4355
M-520-130	12.1	26.9	6.5	12.8	4247	2154	5486	3439
M-600-0	10.0	22.7	8.5	9.2	3943	3087	5192	4450
M-480-120	9.4	20.3	7.3	13.4	3632	2132	7421	3377
M-420-180	6.1	19.6	8.8	22.0	3391	1768	5687	2362
L-700-0	9.3	15.7	10.2	13.7	4766	2947	4497	3332
L-560-140	12.3	20.2	10.6	26	4094	2136	3831	1559
L-650-0	14.8	18.6	9.1	14	4239	2197	4107	3293
L-520-130	13.1	23.3	9.5	27	2532	1409	4389	1848
L-600-0	9.9	17.0	10.0	16.5	3572	1962	4351	3227
L-480-120	9.1	19.8	12.0	29.3	2987	1840	3644	1441
L-420-180	8.4	18.7	10.2	30	2879	1557	4041	1648

4.2.2.1.1 700*/700 pcy of Cementitious Material Surface Resistivity

Figure 4.18 provides the surface resistivity test results for pairs of optimized and non-optimized mixtures, for the 700 lb straight cement series of mixtures (which includes the 20 percent fly ash mixtures). It can be observed that optimized aggregate gradation mixtures consistently exhibited surface resistivity values lower than their companion non-optimized aggregate gradation mixtures and had an expected variability on each test date. Also shown on

Figure 4.18 are the proposed surface resistivity targets for pavement mixtures (11 kΩ*cm), and bridge mixtures (15 kΩ*cm) developed as part of the previous study (Biggers 2019, Cavalline et al. 2020).

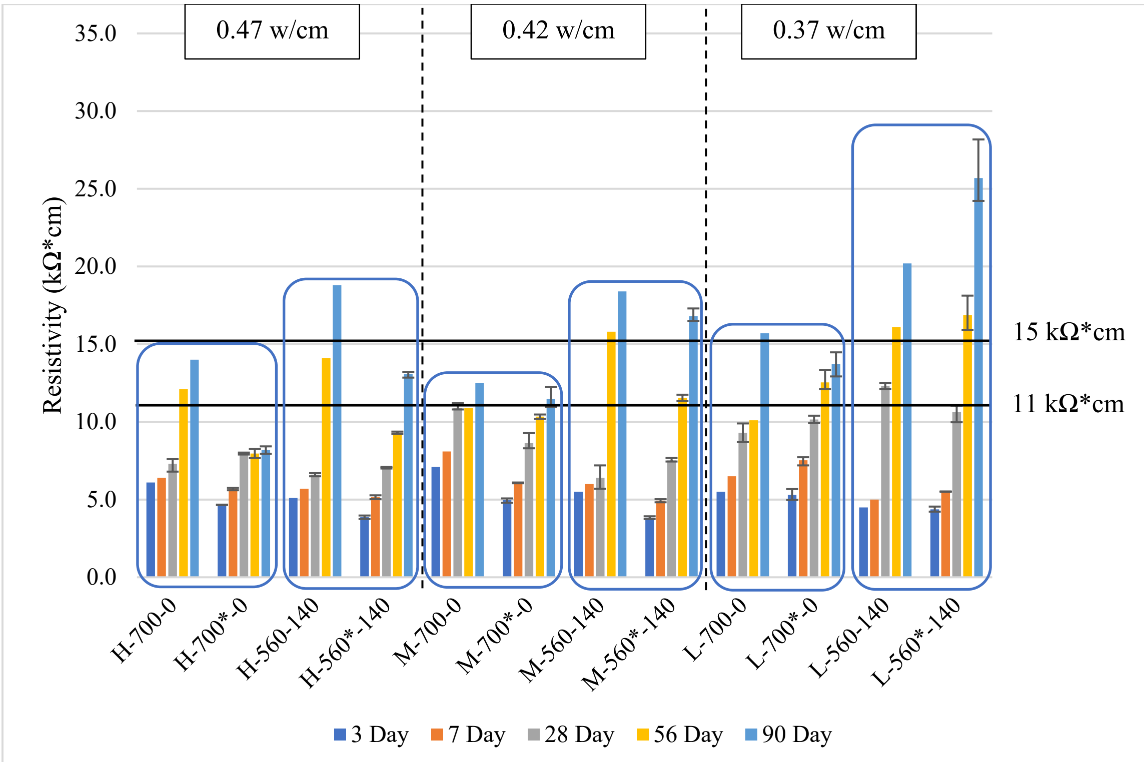

Figure 4.18: Average surface resistivity for 700/700* pcy cementitious material mixtures

Figure 4.19 through Figure 4.21 show the changes in surface resistivity measurements by testing date for mixture pairs at each w/cm ratio. From these plots, the similarities and differences in surface resistivities between pairs of optimized/non-optimized mixtures and between pairs of straight cement/fly ash replacement mixtures can be observed.

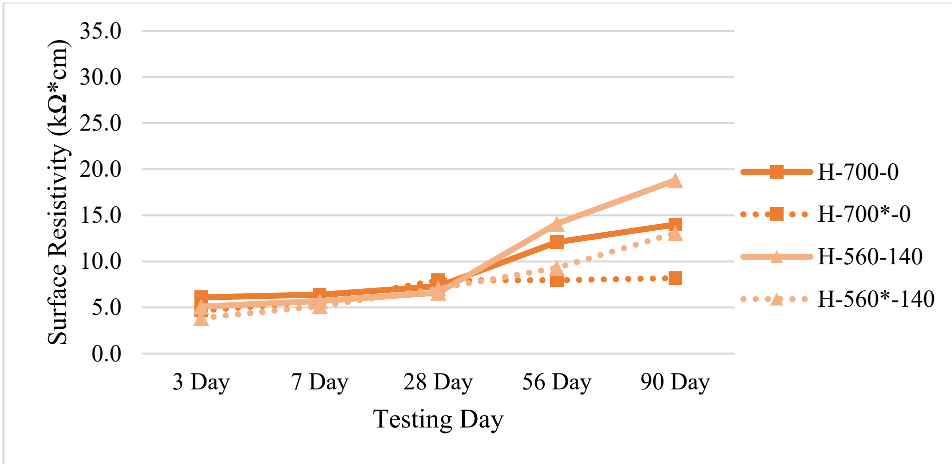

Figure 4.19: Average surface resistivity test results for 700/700* pcy cementitious material mixtures at 0.47 w/cm ratio

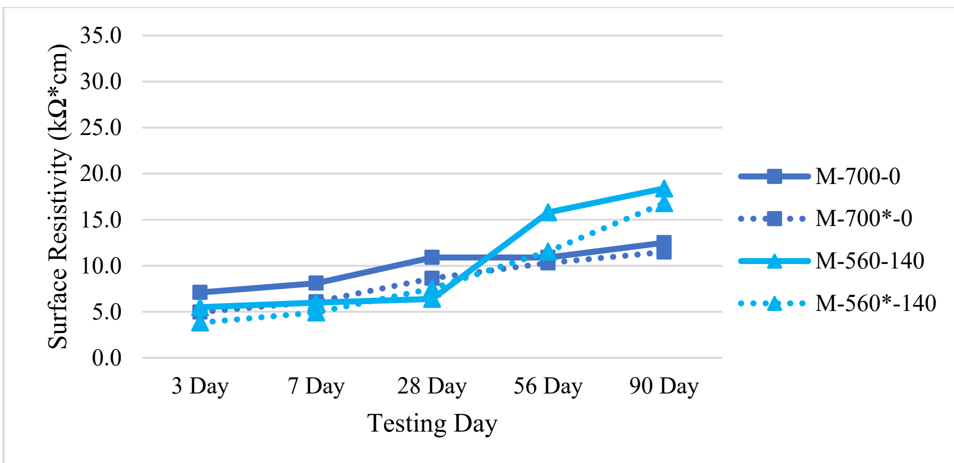

Figure 4.20: Average surface resistivity test results for 700/700* pcy cementitious material mixtures at 0.42 w/cm ratio

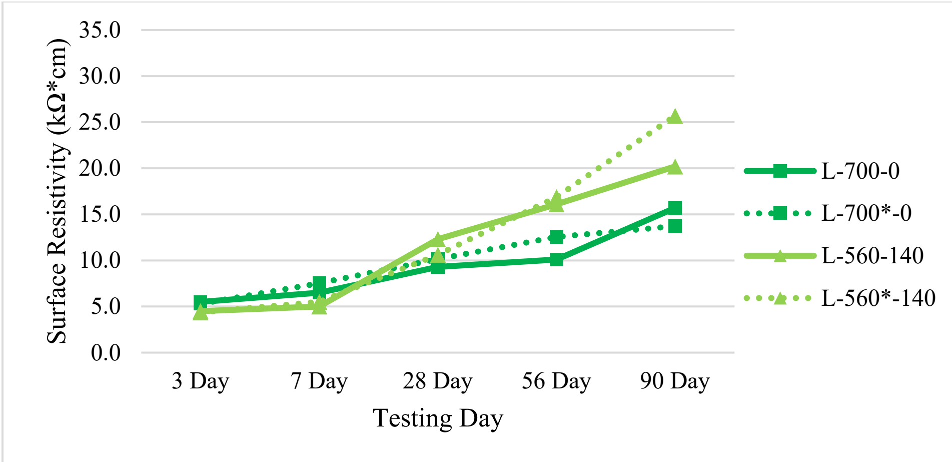

Figure 4.21: Average surface resistivity test results for 700/700* pcy cementitious material mixtures at 0.37 w/cm ratio

This trend could also relate to the 10 percent cementitious material reduction of the optimized aggregate gradation mixtures, as lower cementitious material mixtures have demonstrated improved durability performance. Table 4.18 shows the average percent difference between the average surface resistivities 700* pcy optimized aggregate gradation mixtures average surface resistivities compared to their companion 700 pcy non-optimized mixtures segmented by w/cm ratio. At most ages, and for most mixtures, there is an improvement (increase) in surface resistivity as the w/cm ratio decreases at later ages (after 28 days of age). This trend reinforces the importance of controlling (reducing) the w/cm ratio as the primary tool for producing quality concrete. This trend could also relate to the 10 percent cementitious material reduction of the optimized aggregate gradation mixtures, as lower cementitious material mixtures have demonstrated improved durability performance.

Table 4.18: Average percent difference between surface resistivity of 700* pcy optimized mixtures and companion 700 pcy non-optimized mixtures

w/cm ratio	Test Day				
	3 Day	7 Day	28 Day	56 Day	90 Day
0.47	-31.4%	-11.7%	7.4%	-51.8%	-57.4%
0.42	-43.4%	-27.5%	-5.7%	-21.1%	-9.2%
0.37	-3.4%	11.5%	-3.7%	12.0%	3.5%

Table 4.19 shows average percent differences between the average surface resistivities of the 700* pcy optimized aggregate gradation straight cement and fly ash replacement mixtures compared to their companion 700 pcy non-optimized aggregate gradation straight cement and fly ash replacement mixtures. Both straight cement mixtures and fly ash replacement mixtures with 700* pcy of cementitious material performed similarly to their companion non-optimized 700 pcy mixtures. This is consistent with findings from previous studies performed by the research team, where 700 pcy non-optimized straight cement mixtures outperformed 700 pcy non-optimized fly ash mixtures at the 28-day tests, with fly ash replacement mixtures outperforming straight cement mixtures at 56- and 90-day tests.

As mentioned previously, the 700* mixture series has a cementitious content typical of structural concrete mixtures, and the proposed surface resistivity target is 15 kΩ*cm. Of the 700* pcy optimized mixtures, only L-560*-140 met the recommended 15 kΩ*cm structural concrete surface resistivity target by the 56-day test, and M-560*-140 met the recommended 15 kΩ*cm surface resistivity target by the 90-day test. Of the 700 pcy non-optimized mixtures only M-560-140 and L-560-140 met the recommended 15 kΩ*cm surface resistivity target by the 56-day test, with H-560-140 and L-700-0 meeting the recommended 15 kΩ*cm surface resistivity target by the 90-day test.

Table 4.19: Average percent difference between surface resistivity of 700* pcy optimized straight cement and fly ash replacement mixtures surface resistivity compared with companion 700 pcy non-optimized mixtures

Mixture Characteristic	Test Day				
	3 Day	7 Day	28 Day	56 Day	90 Day
All mixtures	-26.1%	-9.2%	-0.7%	-20.3%	-21.0%
Straight cement mixtures	-25.9%	-10.7%	-3.2%	-12.6%	-31.4%
Fly ash replacement mixtures	-26.3%	-7.8%	1.9%	-27.9%	-10.6%

4.2.2.1.2 650*/650 pcy of Cementitious Material Surface Resistivity

Figure 4.22 has the surface resistivity test results for pairs of optimized and non-optimized mixtures, for the 650 lb straight cement series of mixtures (which includes the 20 percent fly ash mixtures). It can be observed that optimized aggregate gradation mixtures consistently exhibited lower surface resistivity values than their companion non-optimized mixtures. However, some non-optimized aggregate gradation mixtures exhibited low surface resistivity values at early ages and high w/cm ratios as well.

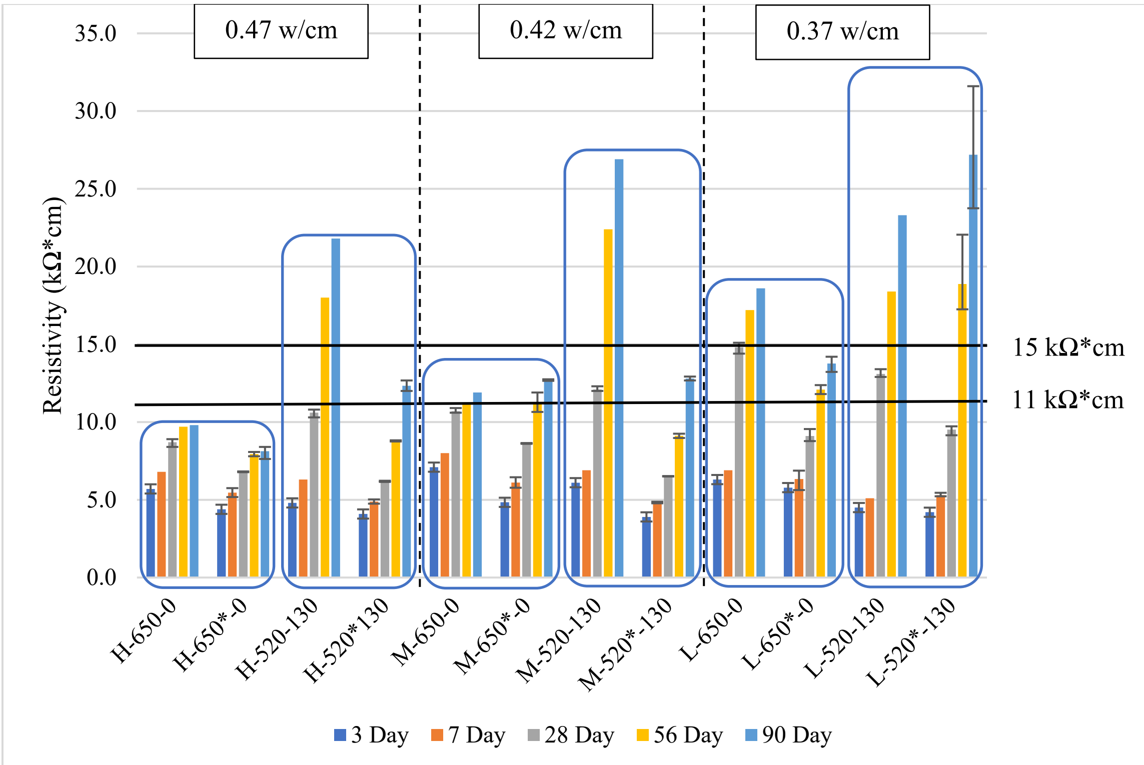

Figure 4.22: Average surface resistivity for 650/650* pcy cementitious material mixtures

Figure 4.23 through Figure 4.25 show the surface resistivity changes by testing age for mixture pairs at each w/cm ratio. From these plots, the similarities and differences resistivity between pairs of optimized/non-optimized mixtures can be observed. It should again be noted that the high (0.47) w/cm ratio mixtures were originally batched as non-optimized aggregate gradation mixtures to provide test results that are indicative of poor concrete mixtures that are not typical to NCDOT use. Optimized aggregate gradation concrete mixtures at the high w/cm ratio were batched and tested for a direct comparison between the optimized and non-optimized aggregate gradation mixtures, however as these mixtures are representative of a higher w/cm ratio than typical NCDOT concrete mixtures, these results may not be as valuable.

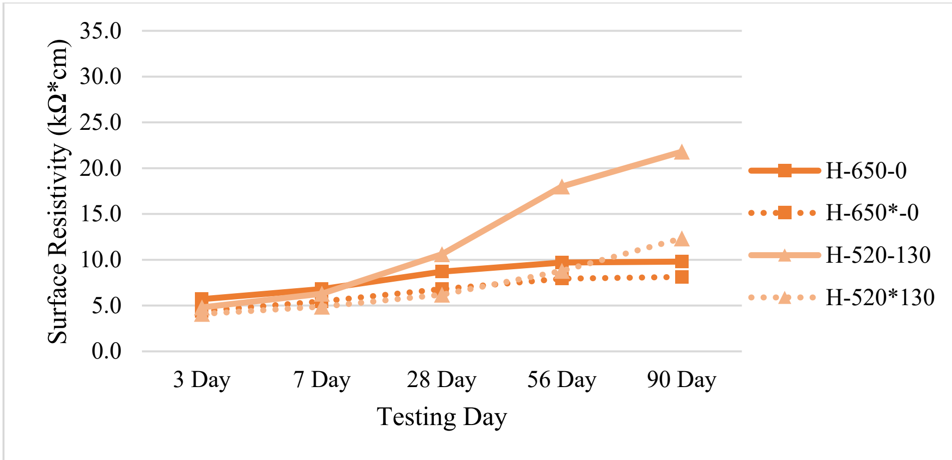

Figure 4.23: Average surface resistivity test results for 650/650* pcy cementitious material mixtures at 0.47 w/cm ratio

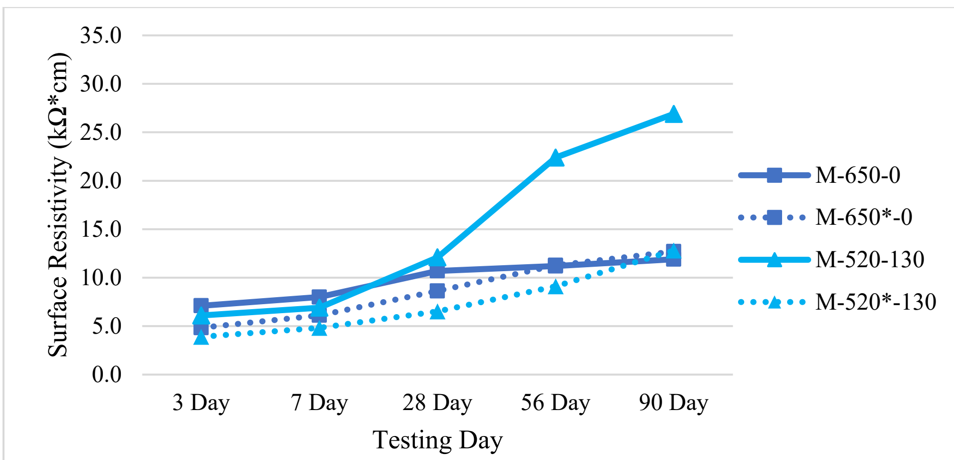

Figure 4.24: Average surface resistivity test results for 650/650* pcy cementitious material mixtures at 0.42 w/cm ratio

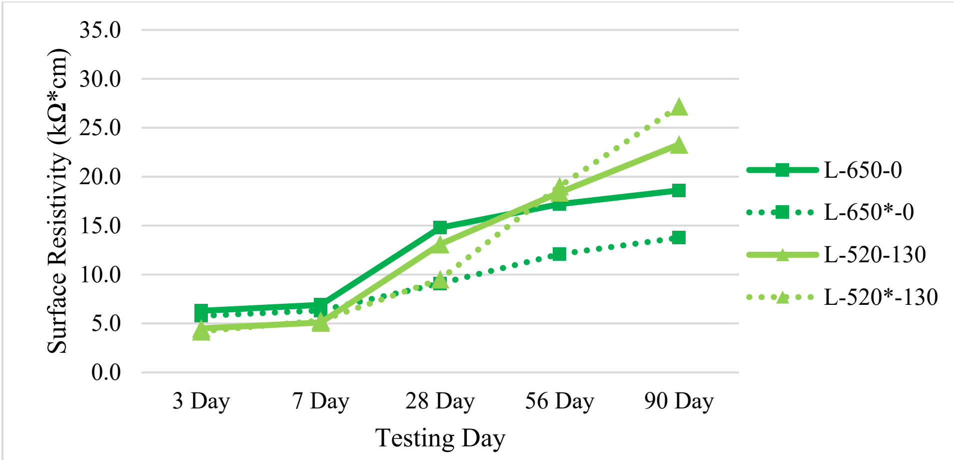

Figure 4.25: Average surface resistivity test results for 650/650* pcy cementitious material mixtures at 0.37 w/cm ratio

Table 4.20 shows the average percent difference in the average surface resistivities between the 650* pcy optimized aggregate gradation compared to their companion 650 pcy non-optimized mixtures grouped by w/cm ratio. The previously mentioned trend where optimized mixtures performed more similarly to their companion non-optimized mixtures as the w/cm ratio decreased present in 650* pcy optimized mixtures was observed for all specimen ages except for 28-day tests. These trends are deviated by two fly ash mixtures, where H-520*-130 had a resistivity 71.2 percent lower than H-520-130, M-520*-130 had a resistivity 85.8 percent lower than M-520-130, and one straight cement mixture, L-650*-0 had a resistivity 65.6 percent lower than L-650-0. For 56- and 90-day tests, mixtures H-520*-130 and M-520*-130 continued to be outperformed by their companion non-optimized mixtures, a trend which is best viewed in Figure 4.23 and Figure 4.24 above. Of note, mixture while L-650*-0 showed the opposite trend, with higher surface resistivity than its companion non-optimized mixture.

Table 4.20 Average percent difference between 650* pcy optimized surface resistivity and their companion 650 pcy non-optimized mixtures

w/cm ratio	Test Day				
	3 Day	7 Day	28 Day	56 Day	90 Day
0.47	-23.7%	-26.7%	-49.6%	-63.5%	-48.7%
0.42	-51.6%	-36.9%	-54.9%	-72.6%	-51.9%
0.37	-8.0%	-2.6%	-50.1%	-19.9%	-10.4%

Table 4.21 shows the average percent difference in the average surface resistivities between 650* pcy optimized aggregate gradation straight cement and fly ash replacement mixtures compared to their companion 650 pcy non-optimized straight cement and fly ash replacement mixtures. The previously observed trend where optimized fly ash replacement mixtures outperformed straight cement optimized mixtures at later dates is not present for this series of mixtures. As mentioned before, H-520*-130 and M-520*-130 were significantly outperformed by their non-optimized companion mixtures. Despite the fly optimized aggregate gradation fly ash replacement mixtures at high and medium w/cm ratios exhibiting lower resistivity, L-520*-130 was the only optimized aggregate gradation mixture to meet the recommended 15 kΩ*cm target for bridge mixtures by the 56-day test at the low w/cm ratio.

Table 4.21: Average percent difference between 650* pcy optimized straight cement and fly ash replacement mixtures surface resistivity compared with companion 650 pcy non-optimized straight cement and fly ash replacement mixtures

Mixture Characteristic	Test Day				
	3 Day	7 Day	28 Day	56 Day	90 Day
All mixtures	-27.8%	-22.1%	-51.5%	-52.0%	-37.0%
Straight cement mixtures	-28.5%	-21.4%	-38.2%	-21.3%	-16.5%
Fly ash replacement mixtures	-27.0%	-22.7%	-64.9%	-82.7%	-57.5%

4.2.2.1.3 600*/600 pcy of Cementitious Material Surface Resistivity

Figure 4.26 provides the surface resistivity test results for pairs of optimized and non-optimized mixtures, for the 600 lb straight cement series of mixtures (which includes the 20 percent fly ash mixtures). It can be observed that optimized aggregate gradation mixtures consistently exhibited surface resistivity values lower than their companion non-optimized aggregate gradation mixtures especially at later testing dates. Both optimized aggregate gradation mixtures and non-optimized aggregate gradations exhibited a similar variability on each test date.

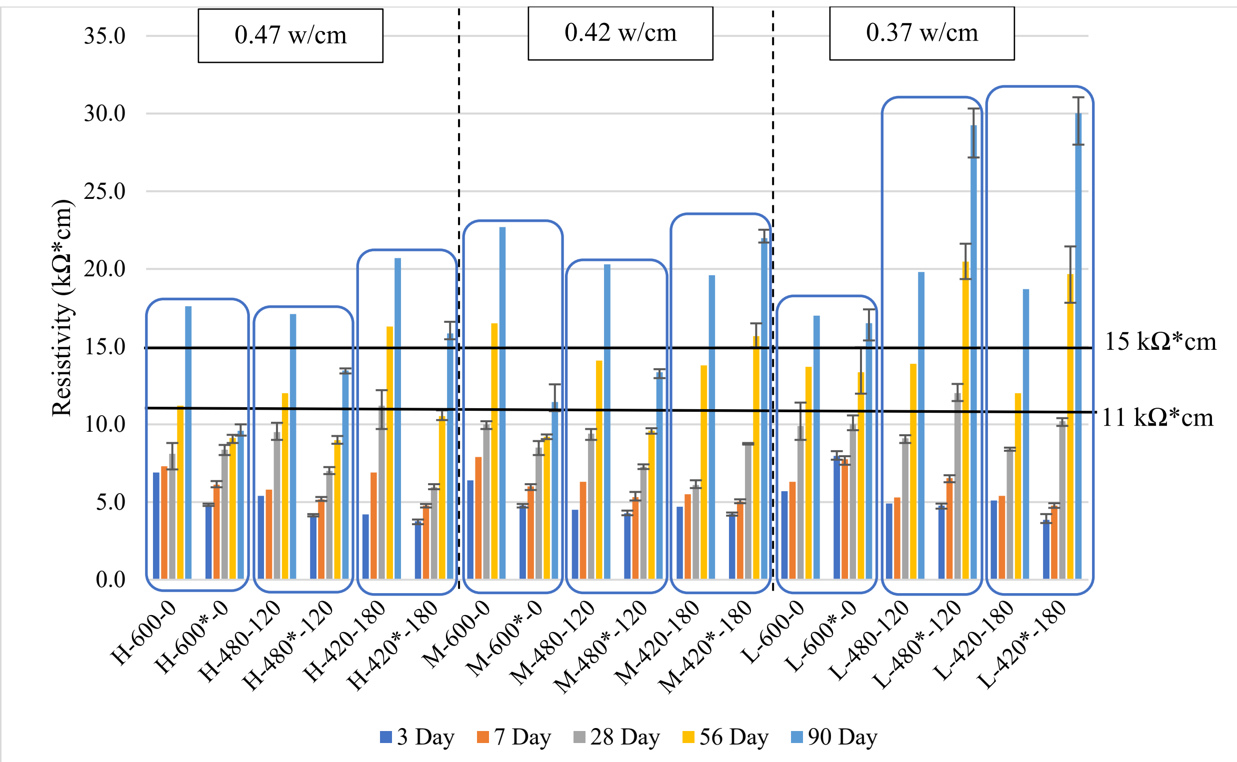

Figure 4.26: Average surface resistivity for 600/600* pcy cementitious material mixtures

Figure 4.27 through Figure 4.29 show the surface resistivity changes by testing date for mixture pairs at each w/cm ratio. From these plots, the differences in higher w/cm ratio mixtures surface resistivity of optimized aggregate gradation mixtures can be noticed at later testing dates. It should again be noted that the high (0.47) w/cm ratio mixtures were originally batched as non-optimized aggregate gradation mixtures to provide test results that are indicative of poor concrete mixtures that are not typical to NCDOT use. Optimized aggregate gradation concrete mixtures at the high w/cm ratio were batched and tested for a direct comparison between the optimized and non-optimized aggregate gradation mixtures, however as these mixtures are representative of a higher w/cm ratio than typical NCDOT concrete mixtures, these results may not be as valuable.

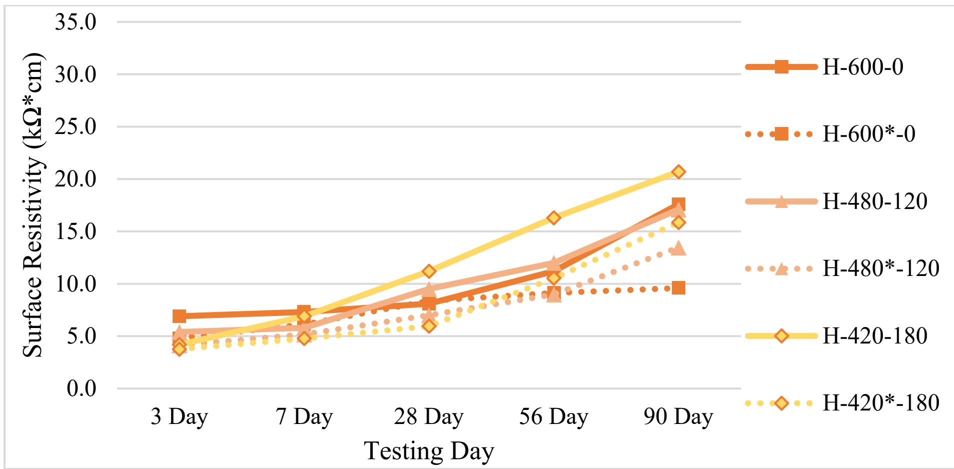

Figure 4.27: Average surface resistivity test results for 600/600* pcy cementitious material mixtures at 0.47 w/cm ratio

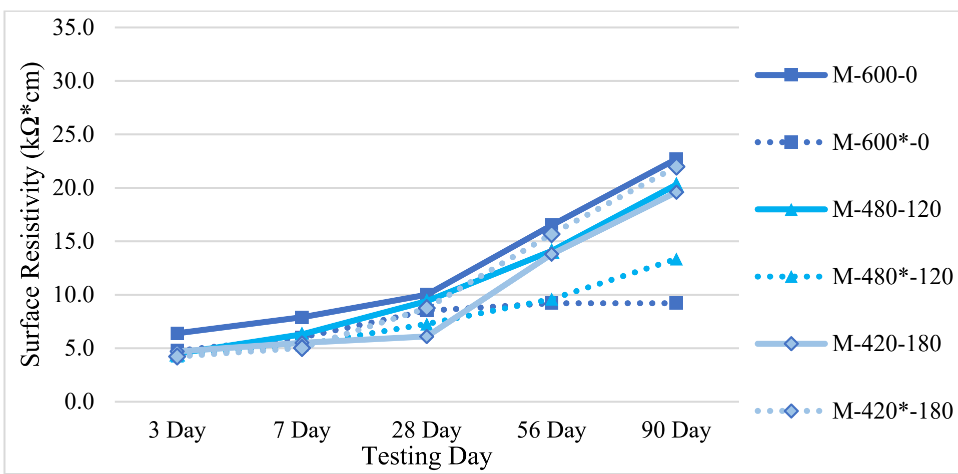

Figure 4.28: Average surface resistivity test results for 600/600* pcy cementitious material mixtures at 0.42 w/cm ratio

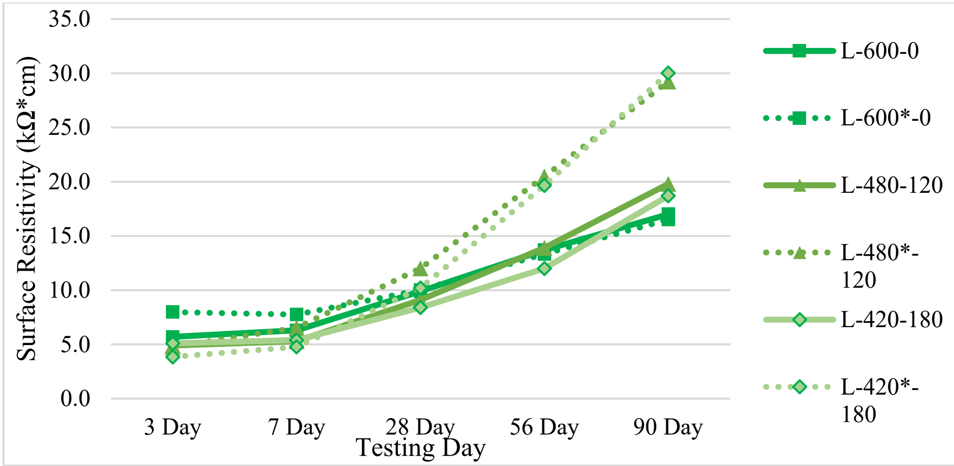

Figure 4.29: Average surface resistivity test results for 600/600* pcy cementitious material mixtures at 0.37 w/cm ratio

Table 4.22 shows the average percent difference between the average surface resistivities of the 600* pcy optimized aggregate gradation mixtures compared to their companion 600 pcy non-optimized mixtures grouped by w/cm ratio. The previously observed trend of optimized aggregate gradation mixtures performing more similarly to their companion non-optimized mixtures as w/cm ratio decreases is present. This trend again reinforces the importance of controlling (reducing) the w/cm ratio as the primary tool for producing quality concrete.

Table 4.22: Average percent difference in surface resistivity between 600* pcy optimized and companion 600 pcy non-optimized mixtures

w/cm ratio	Test Day				
	3 Day	7 Day	28 Day	56 Day	90 Day
0.47	-28.5%	-25.3%	-40.2%	-37.0%	-47.0%
0.42	-16.9%	-19.8%	-5.5%	-38.0%	-46.5%
0.37	-2.2%	8.2%	14.4%	22.8%	22.4%

Table 4.23 shows the average percent difference in average surface resistivities of the 600* pcy optimized aggregate gradation straight cement and fly ash replacement mixtures when compared to their companion 600 pcy non-optimized straight cement and fly ash replacement

mixtures. The previously observed trend of the optimized aggregate gradation fly ash replacement mixtures exhibiting surface resistivities more similar to their companion non-optimized fly ash replacement mixtures at later ages (56- and 90-day tests) is present.

As mentioned previously, the 600 pcy cementititious material mixtures could be more representative of pavement concrete mixtures, which have a proposed target resistivity of 11 kΩ*cm. Of the 600* optimized mixtures, mixtures M-420*-180, L-600*-0, L-480*-120, and L-480*-120 met the recommended 11 kΩ*cm surface resistivity target by the 56-day test, and H-480*-120, H-420*-180, M-600*-0, and M-480*-120 met the recommended 11 kΩ*cm surface resistivity target by the 90-day tests with H-600*-0 being the only 600* optimized mixture to not meet the requirement. All 600 pcy cementitious material non-optimized mixtures met the recommended 11 kΩ*cm surface resistivity values by the 56-day test.

Table 4.23: Average percent difference between 600* pcy optimized straight cement and fly ash replacement mixtures surface resistivity compared with companion 600 pcy non-optimized straight cement and fly ash replacement mixtures

Mixture Characteristic	Test Day				
	3 Day	7 Day	28 Day	56 Day	90 Day
All mixtures	-15.9%	-12.3%	-10.4%	-17.4%	-23.7%
Straight cement mixtures	-16.3%	-10.7%	-4.4%	-34.8%	-61.6%
Fly ash replacement mixtures	-15.7%	-13.1%	-13.5%	-8.7%	-4.8%

4.2.2.1.4 Additional Discussion on Surface Resistivity Results

The previously observed trend that optimized aggregate gradation mixtures perform more similarly to their companion non-optimized mixtures as the w/cm decreases holds true across all cementitious contents, and is highlighted in Table 4.24, where the average surface resistivity values are shown for optimized aggregate gradation mixtures, non-optimized aggregate gradation mixtures, along with the percent difference between companion mixtures. This trend reinforces that producing low w/cm ratio mixtures is very important to support production of quality concrete. However, it should be noted that by reducing the cementitious materials by 10% in optimized aggregate gradation mixtures, the cementitious paste systems of optimized and non-optimized aggregate gradations are not are not identical and could have resulted in the varying surface resistivity results.

Table 4.24: Average surface resistivity of all mixtures grouped by w/cm ratio, along with their average percent difference between pairs of optimized and non-optimized mixtures

w/cm ratio	Mixture type	3 Day	7 Day	28 Day	56 Day	90 Day
0.47	Non-optimized	5.5	6.5	8.9	13.3	17.1
	Optimized	4.2	5.3	7.0	8.9	11.5
	Average percent difference	-28.0%	-21.8%	-29.3%	-48.8%	-50.5%
0.42	Non-optimized	5.9	7.0	9.4	15.0	18.9
	Optimized	4.4	5.5	8.0	11.0	14.4
	Average percent difference	-34.4%	-26.9%	-19.7%	-43.1%	-37.4%
0.37	Non-optimized	5.2	5.8	11.0	14.5	19.0
	Optimized	5.2	6.2	10.2	16.3	22.3
	Average percent difference	-4.2%	6.1%	-9.2%	7.5%	7.6%

Table 4.24 shows the average surface resistivity test results between optimized aggregate gradation mixtures and their companion non-optimized aggregate gradation mixtures and their average percent difference. As cementitious material contents(pcy) decreased in optimized aggregate gradation mixtures there was no noticeable trend that showed optimized mixtures performing more similarly to their companion non-optimized mixtures. However, the 600* pcy optimized aggregate gradation mixtures did perform slightly better than the 700* pcy optimized mixtures at later ages, and significantly better than the 650* pcy optimized mixtures at all ages. This could be related to additional ITZ volume that is added when optimizing the gradation of a

concrete mixture. As the cementitious material content increases in a mixture, so does the volume of the ITZ, likely causing its surface resistivity measurements to decrease. This will be discussed further in section 4.2.2.3.

Table 4.25: Average surface resistivity of all mixtures grouped by cementitious content (pcy) and the average percent difference between optimized and non-optimized mixtures

Cementitious material (pcy)	Mixture type	3 Day	7 Day	28 Day	56 Day	90 Day
700*/700	Non-optimized	5.6	6.3	8.8	13.2	16.6
	Optimized	4.5	5.8	8.7	11.4	14.8
	Average percent difference	-26.1%	-9.2%	-0.7%	-20.3%	-21.0%
650*/650	Non-optimized	5.8	6.7	11.7	16.2	18.7
	Optimized	4.5	5.5	7.8	11.3	14.5
	Average percent difference	-27.8%	-22.1%	-51.5%	-52.0%	-37.0%
600*/600	Non-optimized	5.3	6.3	9.1	13.7	19.3
	Optimized	4.7	5.7	8.7	13.0	17.9
	Average percent difference	-15.9%	-12.3%	-10.4%	-17.4%	-23.7%

Table 4.26 shows the average percent difference of the average surface resistivity measurements of optimized aggregate gradation straight cement and fly ash replacement mixtures. The previously observed trend where w/cm ratio decreases, surface resistivity values improve is visible in optimized fly ash replacement mixtures. Additionally, the trend where the surface resistivity measurements for optimized fly ash replacement mixtures improve at later ages is also visible in optimized fly ash replacement mixtures. By the 56-day test all optimized aggregate gradation fly ash replacement mixtures have an average surface resistivity higher than their companion optimized aggregate gradation straight cement mixtures.

Table 4.26: Average surface resistivity and the average percent difference of all optimized aggregate gradation mixtures straight cement and fly ash mixtures grouped by w/cm ratio

w/cm ratio	Mixture type	3 Day	7 Day	28 Day	56 Day	90 Day
0.47	Optimized Straight Cement mixtures	4.6	5.8	7.7	8.3	8.6
	Optimized Fly Ash Replacement mixtures	4.0	5.0	6.6	9.4	13.7
	Average percent difference	-18.2%	-17.1%	-20.8%	9.0%	34.9%
0.42	Optimized Straight Cement mixtures	4.9	6.1	8.6	10.3	11.9
	Optimized Fly Ash Replacement mixtures	4.1	5.0	7.5	11.5	16.2
	Average percent difference	-19.6%	-20.7%	-15.3%	8.0%	23.7%
0.37	Optimized Straight Cement mixtures	6.4	7.2	9.8	12.7	14.7
	Optimized Fly Ash Replacement mixtures	4.3	5.5	10.6	19.0	28.0
	Average percent difference	-58.4%	-34.3%	6.8%	32.1%	46.1%

As cementitious content decreases in optimized aggregate gradation fly ash mixtures, there is no noticeable trend. However, when optimized aggregate gradation fly ash mixtures are compared with their companion optimized straight cement mixtures in Table 4.27 below, the previously mentioned trend that optimized fly ash mixtures do not perform as well until their later age tests (56- and 90-day tests) can be observed.

Table 4.27: Average surface resistivity and the average percent difference of all optimized aggregate gradation mixtures straight cement and fly ash mixtures grouped by cementitious content (pcy)

w/cm ratio	Mixture type	3 Day	7 Day	28 Day	56 Day	90 Day
700*	Optimized Straight Cement mixtures	5.0	6.4	8.9	10.3	11.1
	Optimized Fly Ash Replacement mixtures	4.0	5.2	8.4	12.6	18.5
	Average percent difference	-24.0%	-23.5%	-7.8%	16.9%	38.5%
650*	Optimized Straight Cement mixtures	5.0	6.0	8.2	10.4	11.5
	Optimized Fly Ash Replacement mixtures	4.1	5.0	7.4	12.3	17.4
	Average percent difference	-23.0%	-19.3%	-12.6%	7.3%	28.1%
600*	Optimized Straight Cement mixtures	5.9	6.6	9.0	10.6	12.5
	Optimized Fly Ash Replacement mixtures	4.2	5.3	8.5	14.2	20.7
	Average percent difference	-40.6%	-26.6%	-9.3%	20.7%	36.5%

4.2.2.2 Rapid Chloride Permeability (RCPT)

RCPT results for 28- and 90-days tests are shown in Table 4.17. Figure 4.30 and Figure 4.31 display the 28- and 90-day test results for optimized aggregate gradation and non-optimized aggregate gradation mixtures, grouped by w/cm ratio. These figures highlight a decrease in chloride permeability as the concrete specimens age for both optimized and non-optimized aggregate gradation mixtures, as typical. With a few exceptions, optimized aggregate gradation mixtures typically exhibited higher chloride permeability than their companion non-optimized aggregate gradation mixtures indicating more permeable mixtures, particularly at early ages and at higher w/cm ratios. However, optimized aggregate gradation mixtures exhibited chloride permeability more similar to their companion non-optimized aggregate gradation mixtures as the w/cm ratio decreases. This again, further reinforcing that producing low w/cm ratio mixtures is important to support the production of quality concrete. Additionally, previous studies have indicated that 90-day tests more accurately predict the permeability of concrete mixtures than 28-day tests, which can be influenced by curing conditions (Joshi and Chan 2002).

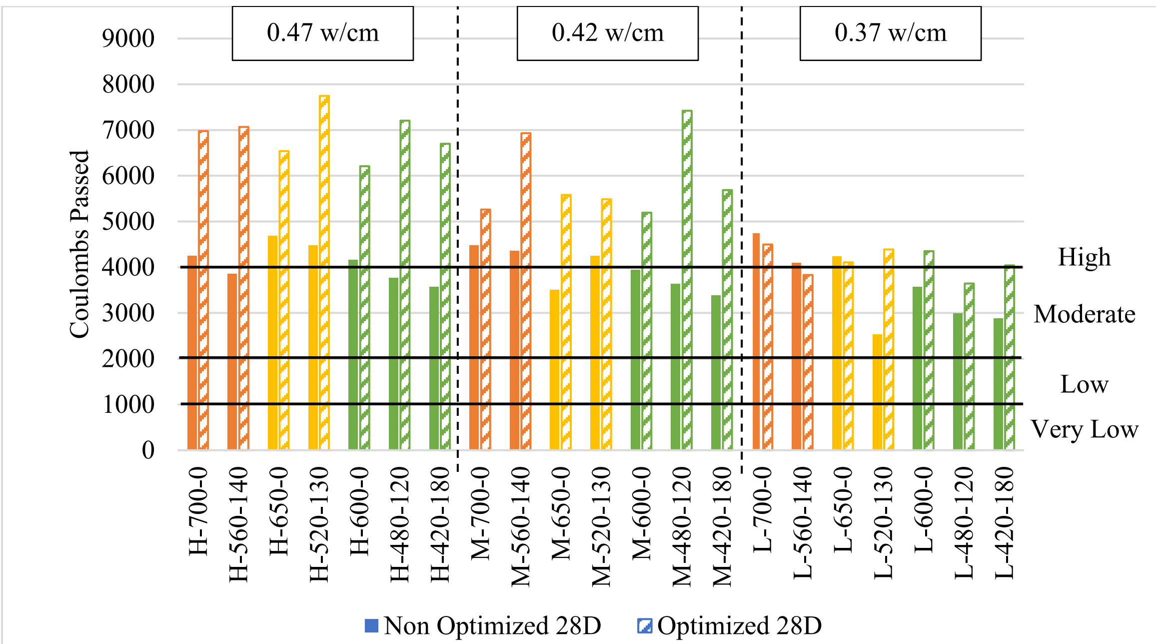

Figure 4.30: 28-day RCPT results for optimized aggregate gradation and non-optimized aggregate gradation mixtures grouped by w/cm ratio

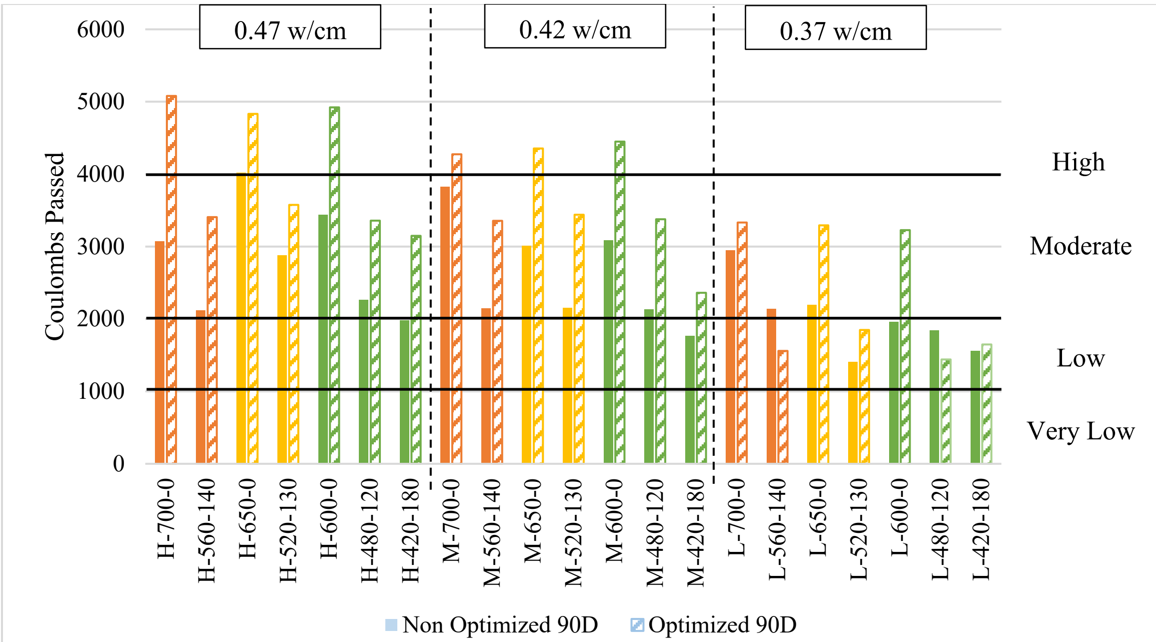

Figure 4.31: 90-day RCPT results for optimized aggregate gradation and non-optimized aggregate gradation mixtures grouped by w/cm ratio

Table 4.28 shows average RCPT results for optimized aggregate gradation mixtures and non-optimized mixtures, grouped by cementitious content (pcy). There was no noticeable trend as the cementitious content decreased in optimized aggregate gradation mixtures. However, regardless w/cm ratio used, optimized aggregate gradation mixtures did show improvement when compared to their non-optimized aggregate gradation mixtures by the 90-day test.

Table 4.28: Average percent difference between RCPT test results for optimized aggregate gradation and non-aggregate gradation mixtures grouped by cementitious content

Cementitious Content (pcy)	Mix Type	Test Day	
		28 Day (Coulombs)	90 Day (Coulombs
700	Non-optimized	4301	2707
	Optimized	5760	3501
	Average percent different	20.6%	16.4%
650	Non-optimized	3949	2611
	Optimized	5641	3557
	Average percent different	28.2%	26.9%
600	Non-optimized	3544	2226
	Optimized	5605	3104
	Average percent different	34.2%	23.3%

Table 4.29 shows average RCPT results for optimized aggregate gradation mixtures and non-optimized gradation mixtures, grouped by w/cm ratio. As the w/cm ratio decreased, there was a noticeable improvement in the chloride ion penetrability of optimized mixtures (lower number of coulombs was measured). This trend again reinforces that producing low w/cm ratio mixtures is very important to ensure quality concrete is produced. Additionally, this trend may support the discussion presented in Section 5.4.4.1, where a higher w/cm ratio mixture combined with the added aggregate likely increases the volume of the ITZ, leading to permeability, as exhibited by the charge passed in the RCPT.

Table 4.29: Average percent difference between RCPT test results of optimized aggregate gradation and non-aggregate gradation mixtures grouped by w/cm ratio

w/cm Ratio	Mix Type	Test Day	
		28 Day (Coulombs)	90 Day (Coulombs)
0.47	Non-optimized	4111	2824
	Optimized	6920	4046
	Average percent different	20.6%	16.4%
0.42	Non-optimized	3936	2588
	Optimized	5937	3659
	Average percent different	28.2%	26.9%
0.37	Non-optimized	3581	2007
	Optimized	4123	2335
	Average percent different	34.2%	23.3%

Table 4.30 shows average RCPT results for optimized aggregate gradation straight cement mixtures and optimized aggregate gradation fly ash replacement mixtures, grouped by w/cm ratio. As previously observed, there was no improvement in permeability of these mixtures as cementitious content decreased. Both optimized straight cement mixtures and optimized fly ash replacement mixtures did show reduced permeability as the w/cm ratio decreased. As the concrete aged, optimized fly ash replacement mixtures outperformed their companion optimized

straight cement mixtures as would be expected from fly ash replacement mixtures due to the later-age hydration and pozzolanic effects.

Table 4.30: Average percent difference between RCPT test results of optimized aggregate gradation and non-aggregate gradation straight cement and fly ash replacement mixtures grouped by w/cm ratio

w/cm Ratio	Mix Type	Test Day	
		28 Day (Coulombs)	90 Day (Coulombs)
0.47	Optimized straight cement	6574	4945
	Optimized fly ash replacement	7179	3372
	Average percent different	8.4%	-46.6%
0.42	Optimized straight cement	5344	4360
	Optimized fly ash replacement	6381	3133
	Average percent different	16.2%	-39.1%
0.37	Optimized straight cement	4318	3284
	Optimized fly ash replacement	3976	1624
	Average percent different	-8.6%	-102.2%

Figure 4.32 indicates that the surface resistivity (kΩ*cm) vs the RCPT test results (charge passed in coulombs) for both non-optimized and optimized mixtures exhibit a similar correlation, supporting the findings of RP 2018-14 (the non-optimized aggregate gradation mixtures) in Biggers (2019) and Cavalline et al. (2019). As can be seen in Figure 4.32, the correlation between surface resistivity tests and RCPT is best modeled using a power-curve, as indicated in a seminal study on surface resistivity by Rupnow and Icenogle (2012).

As previously observed, optimized aggregate gradation mixtures in this study routinely exhibited electrical test results that indicated more permeable mixtures compared to their companion non-optimized aggregate gradation mixtures. This is possibly due to the additional aggregate introducing additional ITZ volume and is discussed in Section 4.2.2.3. Additionally, the higher RCPT results could be attributed to the use of admixtures such as AEA and WRA, or fly ash that was used in the mixtures. Studies have shown that the use of admixtures and SCMs can influence RCPT results, and not accurately indicate a mixtures permeability (Joshi and Chan 2002).

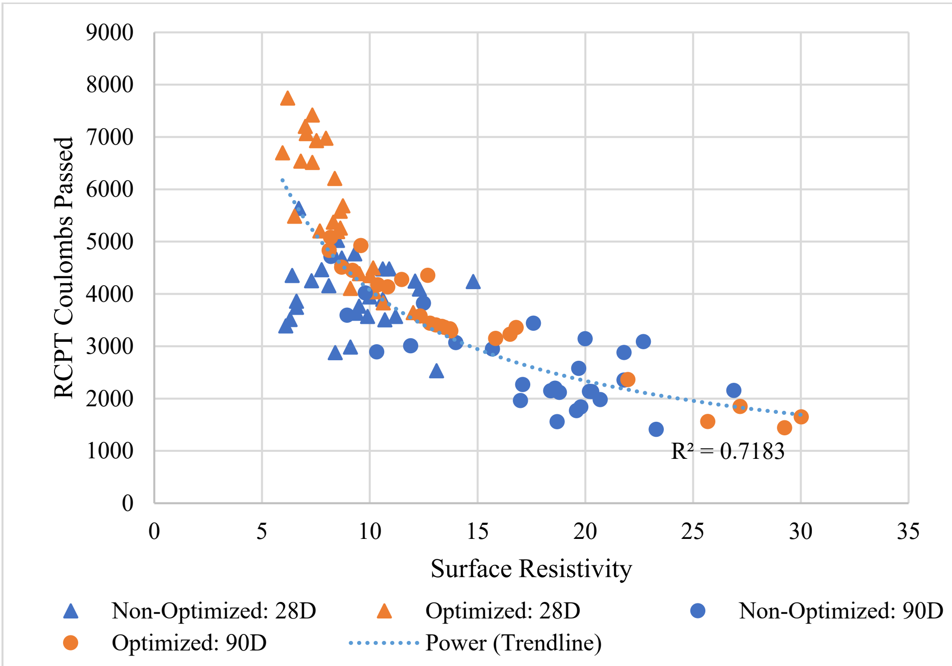

Figure 4.32: Surface resistivity plotted against RCPT test results for optimized aggregate gradation and non-optimized aggregate gradation mixtures

4.2.2.3 Influence of the Interfacial Transition Zone on Electrical Tests

In almost all optimized aggregate gradation mixtures, the surface resistivity and RCPT results showed increased permeability (lower surface resistivity and higher RCPT results) than their companion non-optimized mixtures. This reduction in surface resistivity (and commensurate increase in RCPT charge passed) is likely due to three causes: 1) the reduction in cementitious content of 10 percent between companion mixture pairs, 2) the influence of the additional interfacial transition zone (ITZ) in the paste due to the additional aggregate content, and 3) the potential for the resistivity of the additional aggregate volume to influence the measurement of electrical tests 4) the influence of different quantities of admixtures across non-optimized and optimized aggregate gradation mixtures.

To examine a potential influence of the volume of the ITZ in mixtures, the ITZ volume in optimized aggregate gradation and non-optimized mixtures, the ITZ volume in each mixture was estimated. To compute this estimate, it was assumed that all ITZ around each aggregate would have a unit thickness, enabling a calculation of the surface areas of aggregates to be used to compare the ITZ volume between optimized and non-optimized aggregate gradation mixtures. The surface area of the coarse (#67) and intermediate (#89M) aggregate was estimated using a method developed at the National Center for Asphalt Technology (NCAT) at Auburn University used when calculating the asphalt film thickness, and uses the formula and table below (NCAT 2009). Because we are interested in the added ITZ volume, and the volume of sand was fairly consistent between companion mixtures, only the surface area of the coarse and intermediate aggregate were considered. Table 4.32 shows the estimated surface area of each mixture, computed using the NCAT procedure.

$$SA = \sum P_i * FA_i \qquad \text{Equation 4.5}$$

$$P_i = percent\ passing\ sieve \qquad \text{Equation 4.6}$$

$$FA_i = surface\ area\ factor\ of\ aggregates\ (\text{Table 4.25}\ below) \qquad \text{Equation 4.7}$$

Table 4.31: Surface area factor of aggregates

Sieve opening size	Surface area factor, m^2/kg (ft^2/lb)
1"	0.41 (2)
3/4"	0.41 (2)
1/2"	0.41 (2)
3/8"	0.41 (2)
#4	0.41 (2)
#8	0.82 (4)
#16	1.64 (8)
#30	2.87 (14)
#50	6.14 (30)
#100	12.29 (60)
#200	32.77 (160)

Table 4.32: Estimated surface for optimized and non-optimized mixtures

Mixture ID	Non-optimized mixture estimated surface area (ft^2/yd^3)	Optimized mixture estimated surface area (ft^2/yd^3)
H-700-0	1,367	1,818
H-560-140	1,369	1,794
H-650-0	1,367	1,875
H-520-130	1,368	1,855
H-600-0	1,367	1,944
H-480-120	1,368	1,914
H-420-180	1,369	1,901
M-700-0	1,375	1,863
M-560-140	1,368	1,838
M-650-0	1,367	1,928
M-520-130	1,368	1,906
M-600-0	1,367	1,984
M-480-120	1,368	1,970
M-420-180	1,369	1,853
L-700-0	1,367	1,931
L-560-140	1,369	1,895
L-650-0	1,367	1,977
L-520-130	1,369	1,959
L-600-0	1,367	2,035
L-480-120	1,368	2,009
L-420-180	1,369	1,997

Once the surface areas were calculated, the percent difference was between optimized aggregate gradation mixtures and non-optimized aggregate gradation mixtures calculated. Using this data, Table 4.33 was created, showing the 28-day and 90-day surface resistivities and RCPT results for pares of mixtures, along with the percent difference between the estimated ITZ volumes of the optimized aggregate gradation mixture and the non-optimized aggregate gradation mixtures.

Table 4.33: Surface resistivity, RCPT, and difference of surface area from aggregates of all mixtures

Mix ID	Test Day	Optimized RCPT Results (Coulombs)	Non-Optimized RCPT Results (Coulombs)	Optimized Surface Resistivity (kΩ*cm)	Non-Optimized Surface Resistivity (kΩ*cm)	% Different Surface Area
H-700-0	28 Day	4235	6976	8.0	7.3	25%
	90 Day	3070	5080	8.2	14	
H-560-140	28 Day	3860	7067	7.1	6.6	24%
	90 Day	2118	3407	13.1	18.8	
H-650-0	28 Day	4687	6538	6.8	8.7	27%
	90 Day	4018	4832	8.1	9.8	
H-520-130	28 Day	4480	7746	6.2	10.6	26%
	90 Day	2879	3575	12.3	21.8	
H-600-0	28 Day	4159	6208	8.4	8.1	30%
	90 Day	3439	4922	9.6	17.6	
H-480-120	28 Day	3766	7204	7.0	9.5	29%
	90 Day	2266	3358	13.5	17.1	
H-420-180	28 Day	3571	6699	6.0	11.2	28%
	90 Day	1980	3148	15.9	20.7	
M-700-0	28 Day	4479	5261	8.6	10.9	26%
	90 Day	3822	4275	11.5	12.5	
M-560-140	28 Day	4354	6930	7.5	6.4	26%
	90 Day	2148	3356	16.8	18.4	
M-650-0	28 Day	3506	5580	8.6	10.7	29%
	90 Day	3008	4355	12.7	11.9	
M-520-130	28 Day	4247	5486	6.5	12.1	28%
	90 Day	2154	3439	12.8	26.9	
M-600-0	28 Day	3943	5192	8.5	10	31%
	90 Day	3087	4450	9.2	22.7	
M-480-120	28 Day	3632	7421	7.3	9.4	31%
	90 Day	2132	3377	13.4	20.3	
M-420-180	28 Day	3391	5687	8.8	6.1	26%
	90 Day	1768	2362	22.0	19.6	
L-700-0	28 Day	4766	4497	10.2	9.3	29%
	90 Day	2947	3322	13.7	15.7	
L-560-140	28 Day	4094	3831	10.6	12.3	28%
	90 Day	2136	1559	26.0	20.2	
L-650-0	28 Day	4239	4107	9.1	14.8	31%
	90 Day	2197	3293	14.0	18.6	
L-520-130	28 Day	2532	4389	9.5	13.1	30%
	90 Day	1409	1848	27.0	23.3	
L-600-0	28 Day	3572	4351	10.0	9.9	33%
	90 Day	1962	3227	16.5	17	
L-480-120	28 Day	2987	3644	12.0	9.1	32%
	90 Day	1840	1441	29.3	19.8	
L-420-180	28 Day	2879	4041	10.2	8.4	31%
	90 Day	1557	1648	30.0	18.7	

Figure 4.33 through Figure 4.44 were created to show the correlation of ITZ volume and the influence on surface resistivity and RCPT test results. Figure 4.33 through Figure 4.35 plot the percent aggregate by volume against the surface resistivity of optimized aggregate gradation mixtures only against the surface resistivity results grouped by w/cm ratio. Figure 4.36 through Figure 4.38 plot the percent aggregate by volume against the surface resistivity results of optimized and non-optimized aggregate gradation mixtures grouped by w/cm ratio. Figure 4.39 through Figure 4.41 plot the percent aggregate by volume against the RCPT results of optimized aggregate gradation mixtures grouped by w/cm ratio. Figure 4.42 through Figure 4.44 plot the percent aggregate by volume against the RCPT test result of optimized and non-optimized aggregate gradation mixtures. These figures were created to provide a potential explanation for the discrepancy in electrical test results for optimized and non-optimized aggregate gradation mixtures. However, it should be noted that by changing the cementitious material content in the optimized aggregate gradation mixtures, the cementitious matrix of the mixtures was changed as well making a direct comparison between the mixtures inaccurate.

As the w/cm ratio decreases, the quality of the ITZ likely decreases as well, since there is less water present to increase the thickness and permeability of the ITZ. This could explain the reason that the difference between electrical test results (surface resistivity and RCPT) between optimized and non-optimized mixtures to be reduced. The increase in ability to carry electrical current does not necessarily mean optimized aggregate gradation mixtures are more permeable (and inherently less durable). Instead, it may indicate that performance targets for optimized aggregate gradation mixtures may need to be adjusted. Additional discussion is presented following these figures, and research into the relationship between the ITZ and electrical test results is recommended.

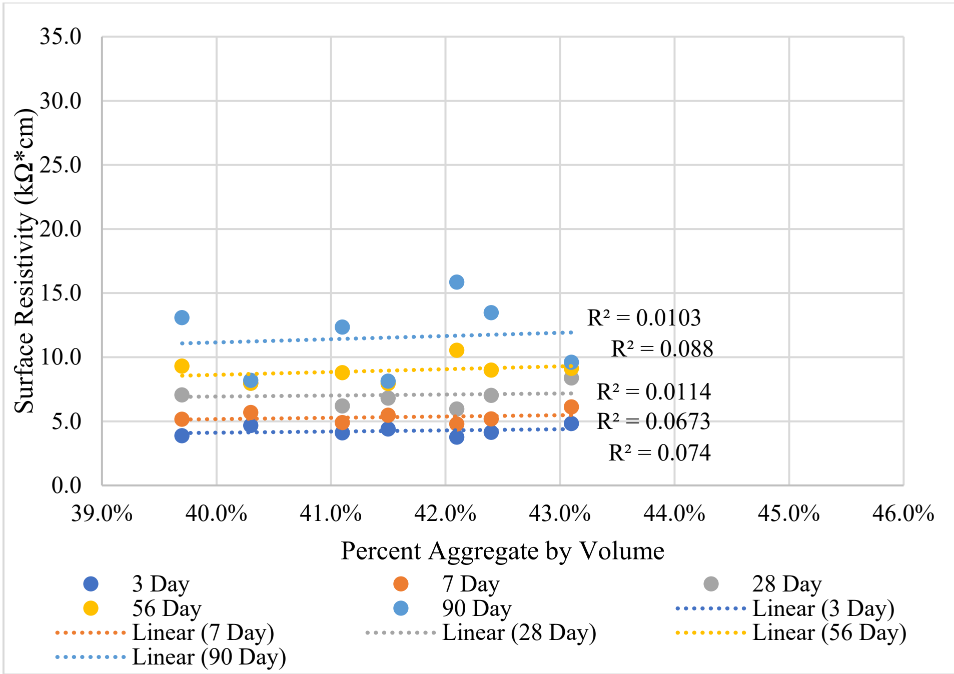

Figure 4.33: Surface resistivities of high w/cm ratio optimized concrete mixtures vs percent coarse aggregate volume

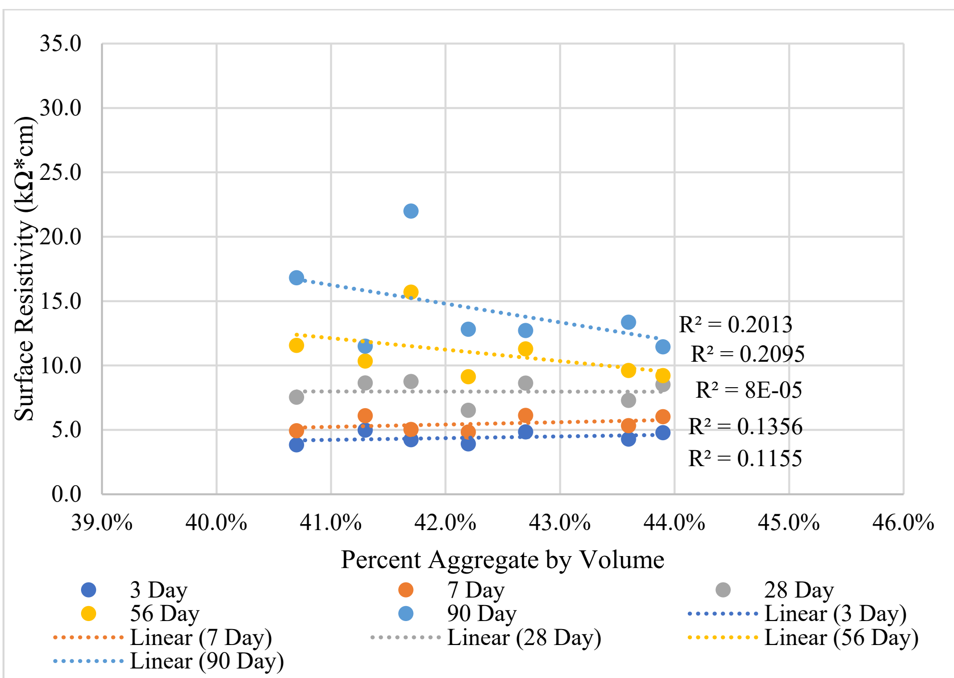

Figure 4.34: Surface resistivities of medium w/cm ratio optimized concrete mixtures vs percent coarse aggregate volume

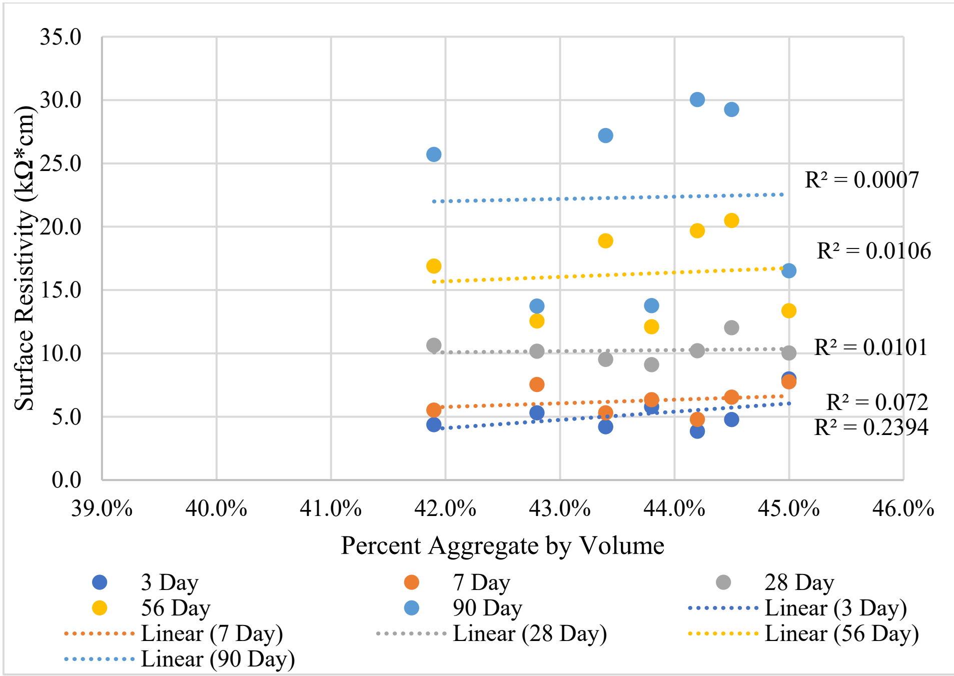

Figure 4.35: Surface resistivities of low w/cm ratio optimized concrete mixtures vs percent coarse aggregate volume

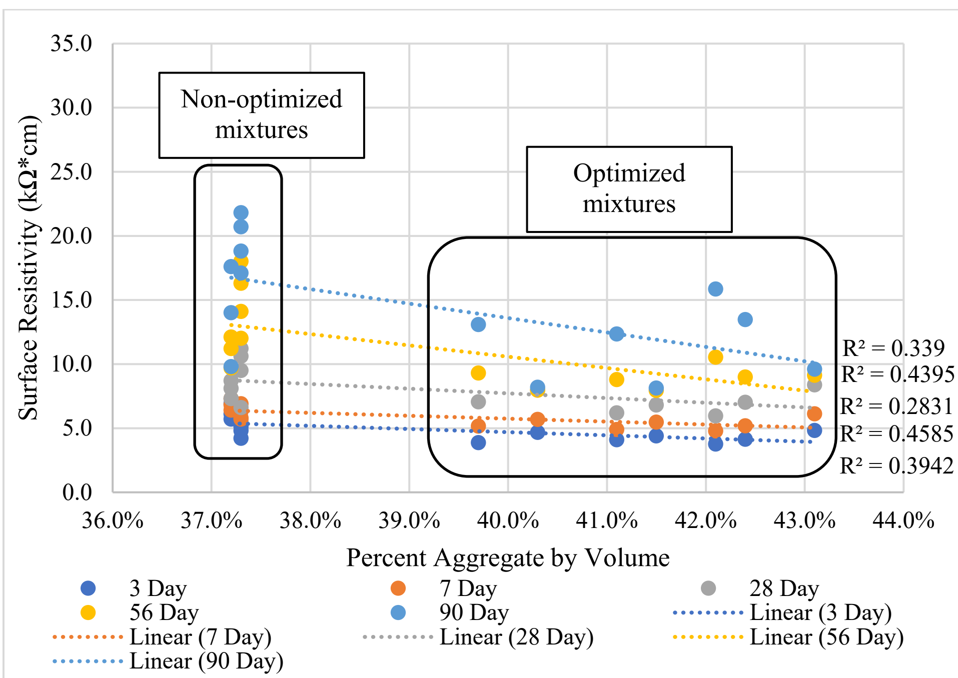

Figure 4.36: Surface resistivities of high w/cm ratio optimized and non-optimized concrete mixtures vs percent coarse aggregate volume

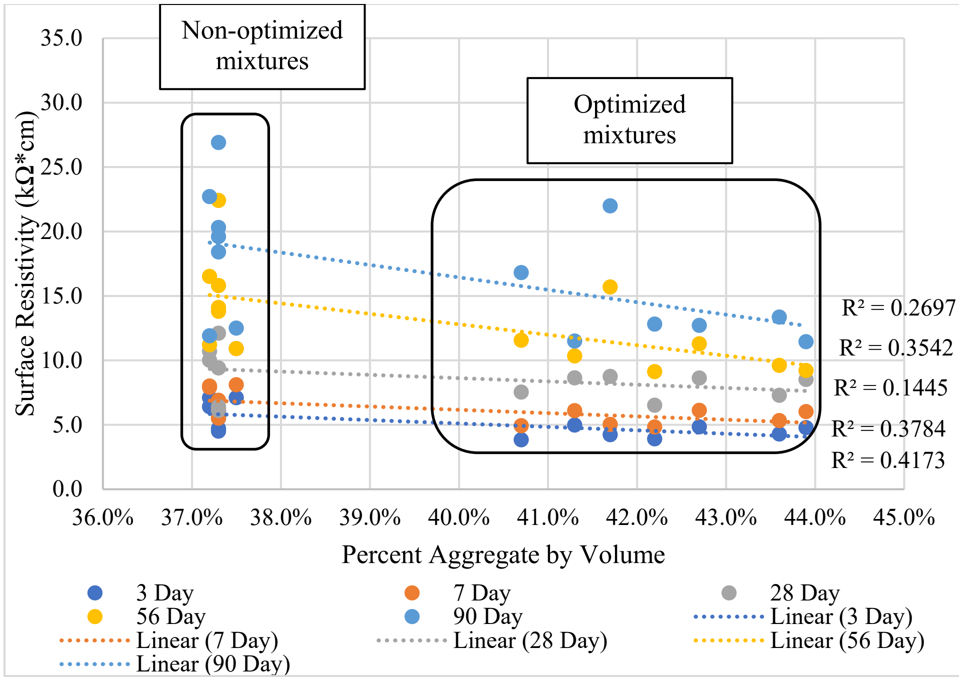

Figure 4.37: Surface resistivities of medium w/cm ratio optimized and non-optimized concrete mixtures vs coarse aggregate volume

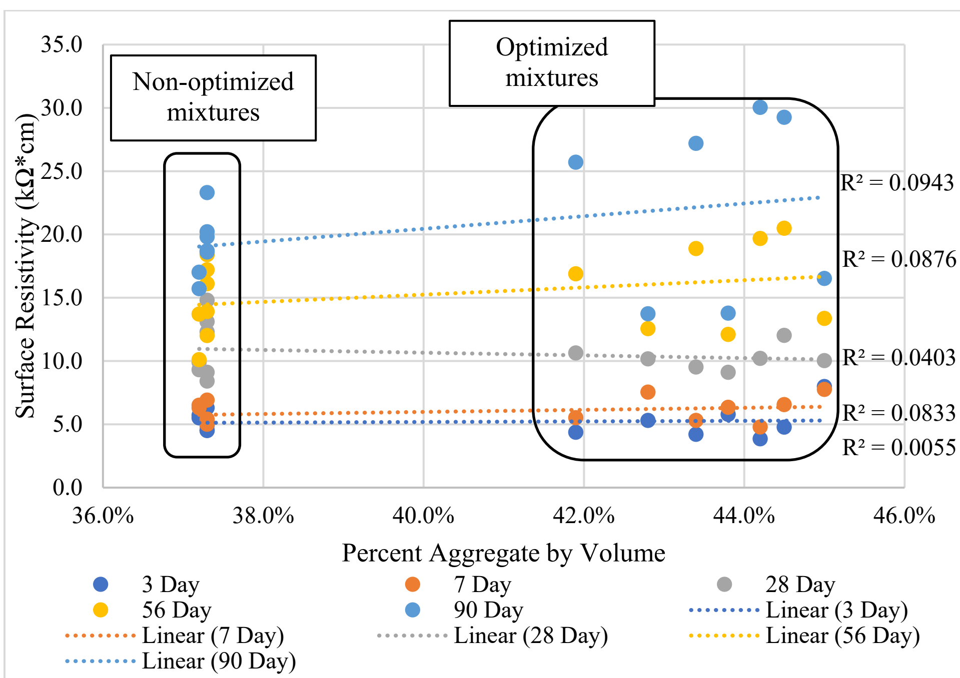

Figure 4.38: Surface resistivities of low w/cm ratio optimized and non-optimized concrete mixtures vs coarse aggregate volume

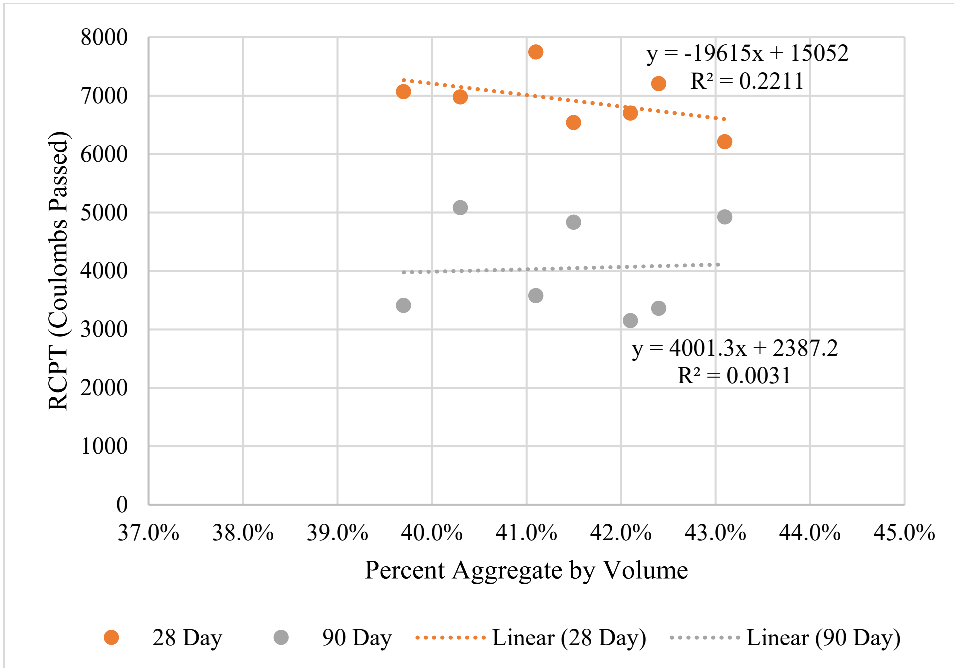

Figure 4.39: RCPT results of high w/cm ratio optimized concrete mixtures vs coarse aggregate volume

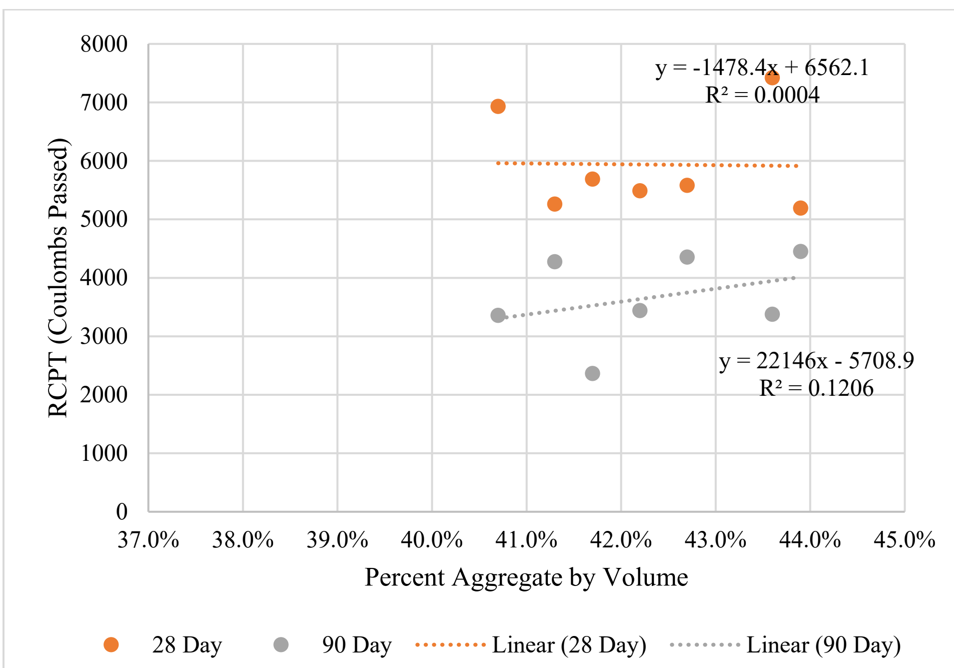

Figure 4.40: RCPT results of medium w/cm ratio optimized concrete mixtures vs coarse aggregate volume

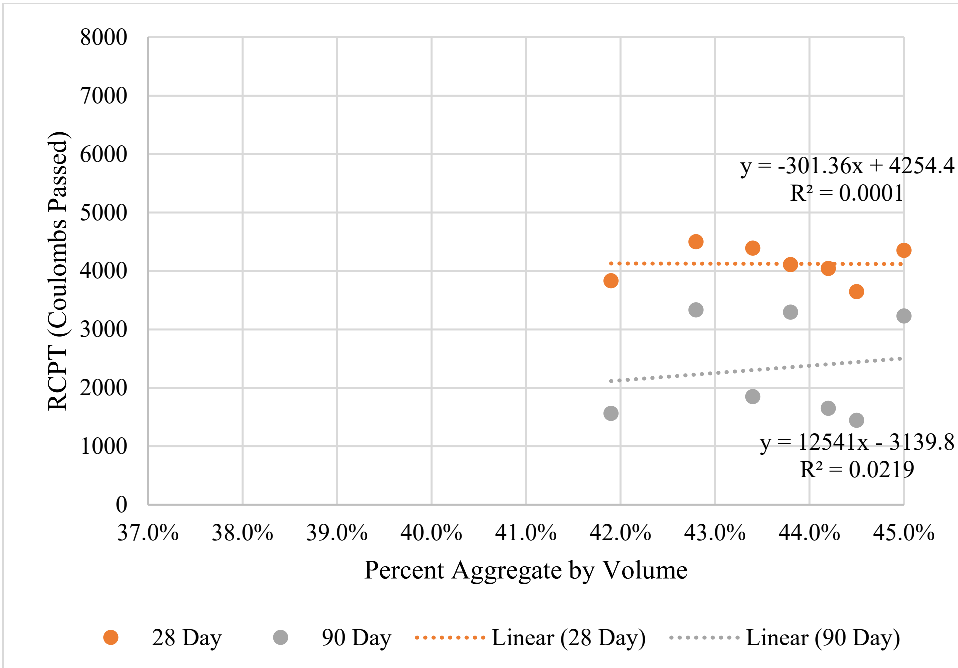

Figure 4.41: RCPT results of low w/cm ratio optimized concrete mixtures vs coarse aggregate volume

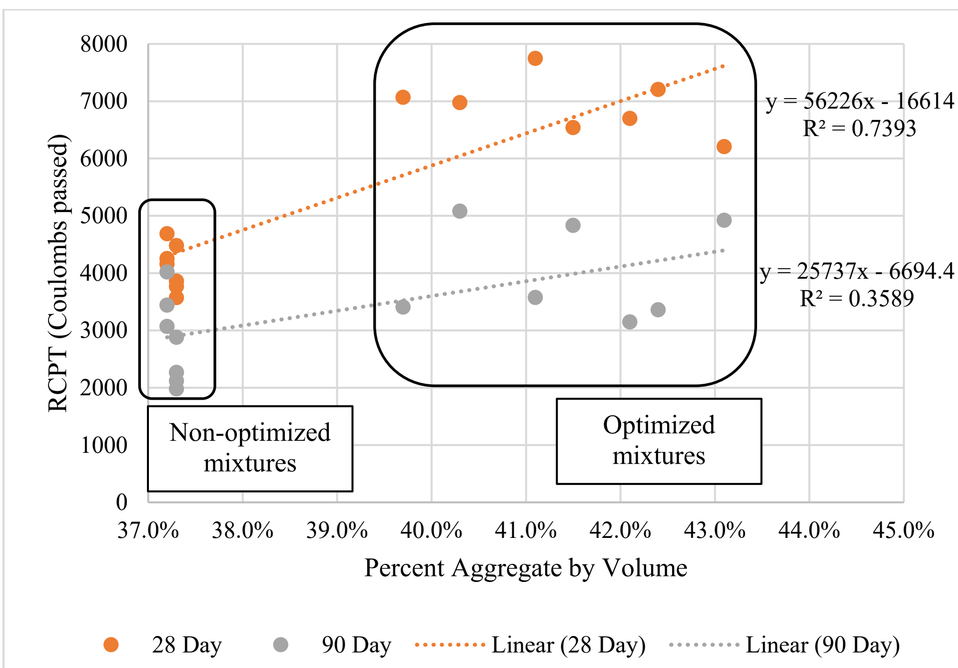

Figure 4.42: RCPT results of high w/cm ratio optimized and non-optimized concrete mixtures vs coarse aggregate volume

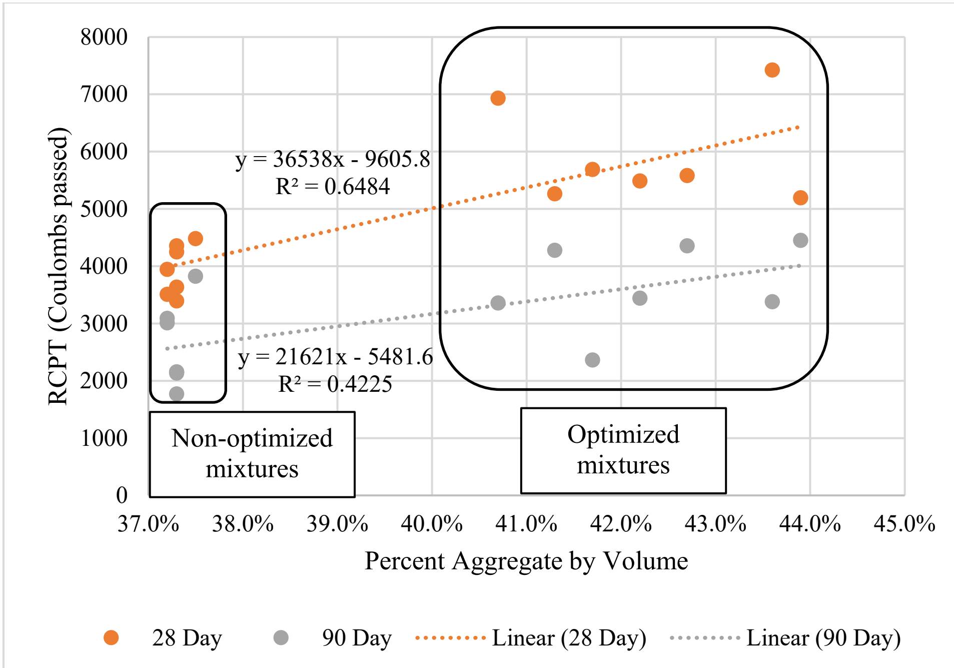

Figure 4.43: RCPT results of medium w/cm ratio optimized and non-optimized concrete mixtures vs coarse aggregate volume

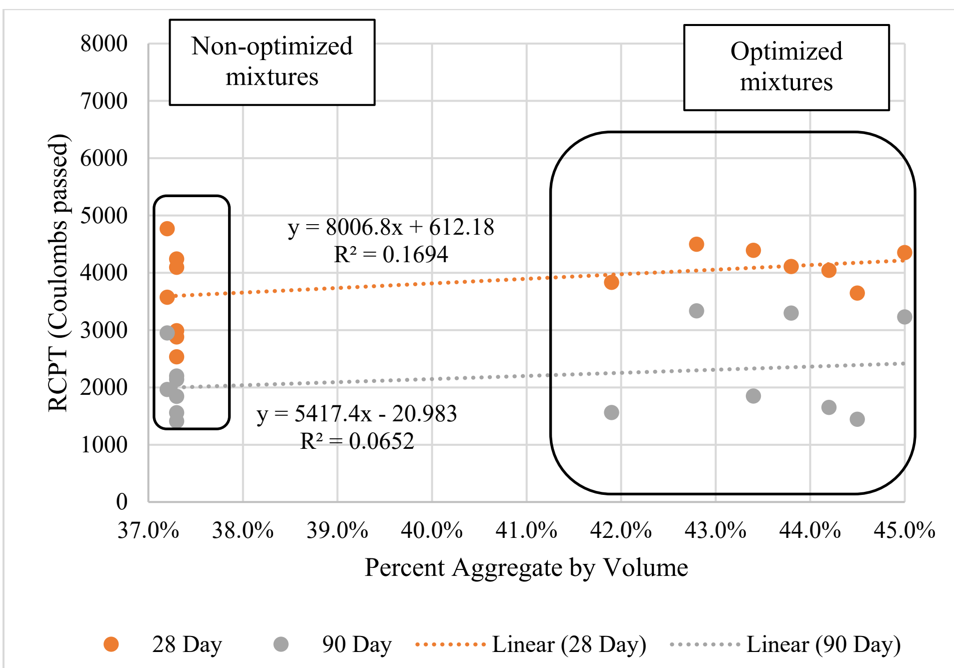

Figure 4.44: RCPT results of low w/cm ratio optimized and non-optimized concrete mixtures vs coarse aggregate volume

Research has been completed and presented to the Transportation Research Board on the influence of recycled aggregate concrete by a team at Rowan University suggesting that the additional mortar contained in recycled concrete aggregates influences electrical tests, and target thresholds for chloride ion penetrability classifications should be lower (Lomboy 2021). Research has also been performed at Oklahoma State University (OSU) that presented the idea that the gradation of aggregate used in a concrete mixture impacts the surface resistivity (Govindbhai 2012). Figure 4.45 and Figure 4.46 is a relationship they found between coarse aggregate volume and the measured surface resistivity of mixtures with a w/cm ratio of 0.45 (Govindbhai 2012). This trend indicates that the resistivity of the cylinder could be thought of as a composite resistivity: influenced by the resistivity of the paste, the resistivity of the fine aggregate, and the resistivity of the coarse aggregate.

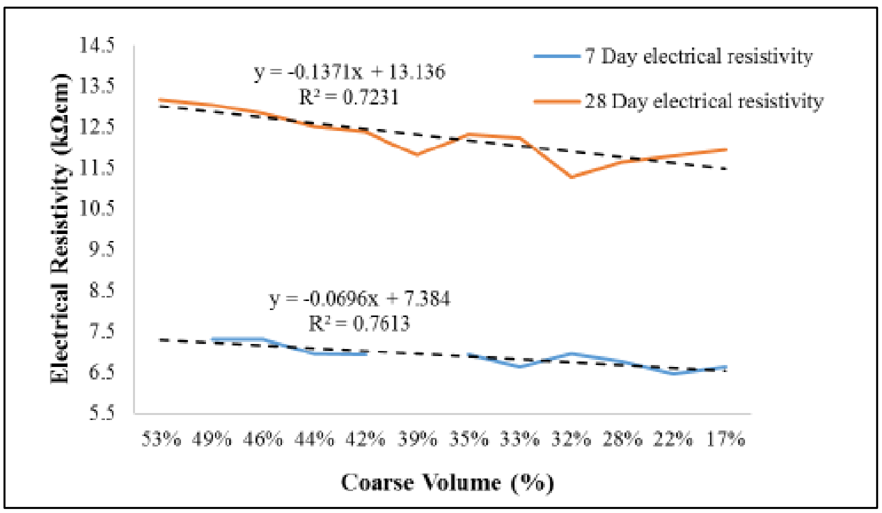

Figure 4.45: Resistivity vs. coarse aggregate volume (%) of mixtures containing 6oz/cwt of water reducer in Govindbhai (2012)

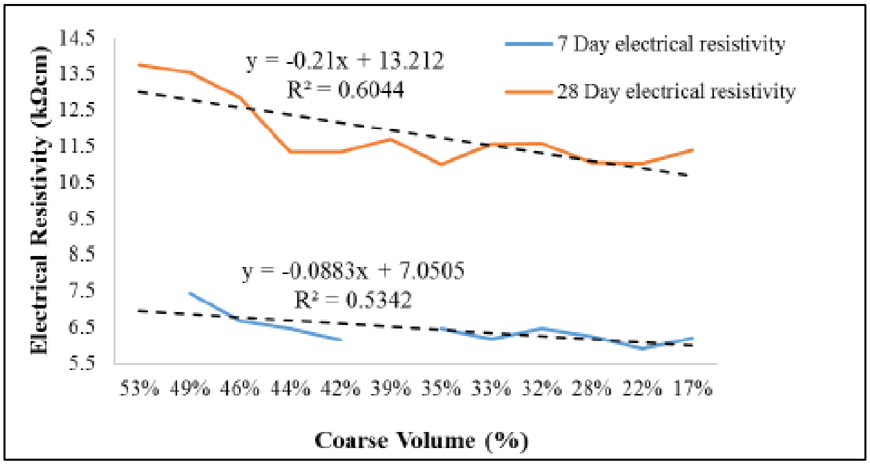

Figure 4.46: Resistivity vs. coarse aggregate volume (%) of mixtures containing 9oz/cwt of water reducer in Govindbhai (2012)

It is noted that the study performed at OSU found a relationship that was inverse to the relationship finding in this study. However, the research performed by Dr. Lomboy at Rowan University on recycled aggregate concrete seems to support the findings of this study, showing that an increased ITZ decreases electrical resistivity, and the electrical tests of this concrete is also influenced by the w/cm ratio and the volume of coarse aggregate in the concrete mixture. More research on the relationship between coarse aggregate volume and electrical tests is recommended.

While the optimized aggregate gradation mixtures electrical durability testing indicates more permeable concrete mixtures, factors such as introducing additional aggregate and possibly increasing the ITZ may be causing these deviations. Additionally, as the cementitious material of optimized aggregate gradation mixtures was reduced by 10%, the cementitious paste system in the non-optimized aggregate gradation mixtures has been changed as mentioned before. This change, in addition to changes in chemical admixture dosages for the optimized aggregate gradation mixtures, may have played a role in the skewed surface resistivity results of the optimized aggregate gradation mixtures.

4.2.2.4 Shrinkage

Volumetric shrinkage measurements in microstrain for optimized and non-optimzed mixtures are reported in Table 4.34 for 28-day, 8 week, 16 week, and 32 week testing days (after 28 day curing period). This table has been prepared using shrinkage values calculated using the measurement directly after curing as the initial comparator reading to facilitate comparison between optimized aggregate gradation mixtures with their companion non-optimized mixtures. Table 4.35 provides the microstrain of optimized aggregate gradation mixtures with the microstrains for 28-day, 8 week, 16 week, and 32 week testing days, but has been calculated using the initial reading directly after demolding as the comparator reading as per ASTM C157. For optimized aggregate gradation mixtures, the difference in the two methods with comparator reading at 28-day and 0-day directly after demolding resulted in an average difference of 78 microstrain across all mixtures and all test dates. Table 4.34 displays the volumetric shrinkage results for the 28-day test for both optimized and non-optimized gradation mixtures, with the values calculated using the measurement taken directly after curing as the initial comparator reading. Table 4.35 displays the volumetric shrinkage results for the 28-day test for both optimized and non-optimized gradation mixtures by calculations using the measurement directly after de-molding as the initial comparator reading per the ASTM C157 standard. For optimized aggregate gradation mixtures, the difference in the two methods with comparator reading at 28-day and 0-day directly after demolding resulted in an average difference of 78 microstrain across all mixtures and all test dates.

Table 4.34: Optimized and non-optimized aggregate gradation mixtures volumetric shrinkage (in microstrain) 28-day reading is used as initial reading

Mixture ID	Non-Optimized				Optimized			
	28 Day	8 Week	16 Week	32 Week	28 Day	8 Week	16 Week	32 Week
H-700-0	312	382	424	504	350	493	590	667
H-560-140	301	376	424	937	297	397	487	583
H-650-0	-	-	-	-	350	480	580	653
H-520-130	286	342	439	-	330	460	540	613
H-600-0	261	322	429	829	340	473	553	603
H-480-120	258	329	420	683	400	527	577	640
H-420-180	246	336	439	592	327	457	520	557
M-700-0	322	401	498	567	417	577	630	643
M-560-140	318	387	448	1185	387	497	553	613
M-650-0	310	380	462	515	430	580	633	647
M-520-130	304	389	389	-	403	497	560	610
M-600-0	274	328	378	835	293	423	443	480
M-480-120	279	339	401	778	410	493	547	603
M-420-180	292	361	415	618	357	450	500	547
L-700-0	314	414	513	-	390	510	577	653
L-560-140	347	447	546	-	400	503	587	643
L-650-0	333	401	483	1140	370	495	575	587
L-520-130	318	414	501	-	440	567	640	707
L-600-0	298	371	430	703	370	440	480	550
L-480-120	304	375	437	964	370	463	513	587
L-420-180	309	367	419	599	377	453	503	577

Table 4.35: Optimized aggregate gradation mixtures volumetric shrinkage (in microstrain) (0-day reading is comparator reading per ASTM C157 standard)

Mixture ID	28 Day	8 Week	16 Week	32 Week
H-700*-0	297	440	537	613
H-560*-140	240	350	450	477
H-650*-0	287	417	517	590
H-520*-130	217	347	427	500
H-600*-0	253	387	467	517
H-480*-120	287	413	463	527
H-420*-180	250	380	443	480
M-700*-0	323	483	537	550
M-560*-140	330	440	497	557
M-650*-0	297	447	500	513
M-520*-130	293	387	450	500
M-600*-0	237	367	387	423
M-480*-120	303	387	440	497
M-420*-180	307	400	450	497
L-700*-0	337	457	523	600
L-560*-140	317	420	503	560
L-650*-0	275	400	480	477
L-520*-130	337	463	537	603
L-600*-0	347	417	457	527
L-480*-120	327	420	470	543
L-420*-180	317	393	443	517

Figure 4.47 displays the volumetric shrinkage results for the 28-day test for both optimized and non-optimized gradation mixtures by calculations using the measurement directly after curing as the initial comparator reading. Figure 4.48 displays the volumetric shrinkage results for the 28-day test for both optimized and non-optimized gradation mixtures using the shrinkage values calculated using the measurement directly after de-molding as the initial comparator reading per the ASTM C157 standard.

When using the measurement after curing as the comparator reading, optimized aggregate gradation mixtures did not perform as well as their companion non-optimized mixtures at the early age testing requirements as shown in Figure 4.47. However, all optimized aggregate gradation mixtures did meet the AASHTO PP 84 420 microstrain requirement at their 28-day test except for H-520*-130, which had a 28-day microstrain of 440. All mixtures did meet the AASHTO PP 84 420 microstrain requirement at their 28-day test when using the measurement directly after demolding per ASTM C157.

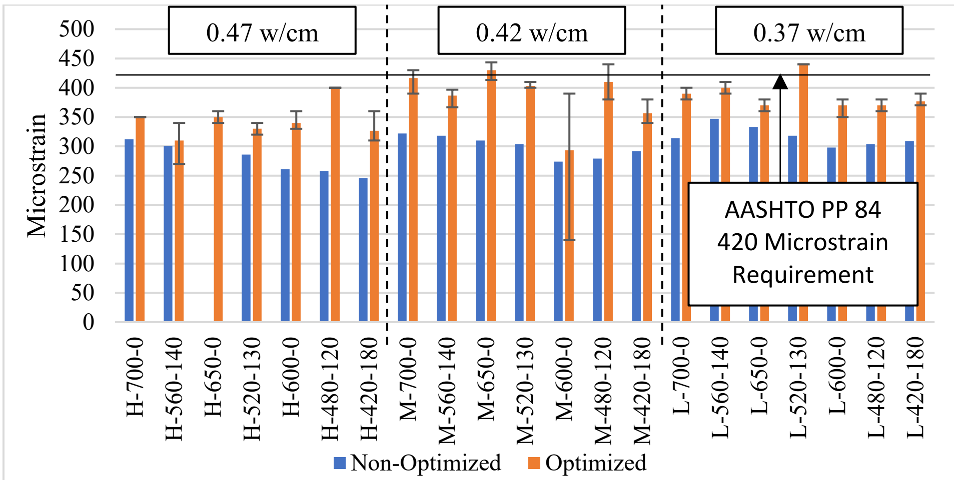

Figure 4.47: Volumetric shrinkage measurements (in microstrain) for optimized and non-optimized aggregate gradation mixtures, 28 day as initial reading

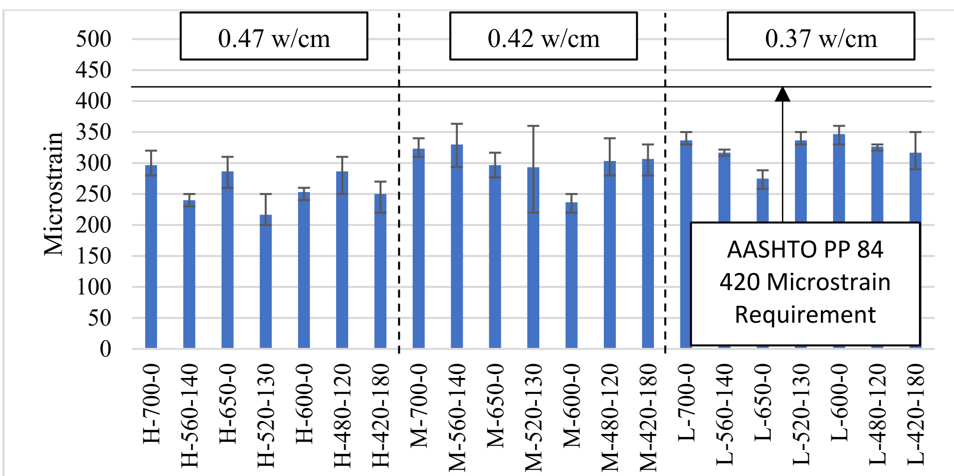

Figure 4.48: Volumetric shrinkage measurements (in microstrain) for optimized aggregate gradation mixtures (0-day reading is comparator reading per ASTM C157 standard)

Table 4.36 shows the average shrinkage (in microstrain) and the percent difference between optimized aggregate gradation mixtures and non-optimized mixtures, for all mixtures, straight cement mixtures, and fly ash replacement mixtures. As the concrete continues to age, optimized aggregate gradations mixtures began to exhibit less shrinkage (lower microstrain) when compared to their companion non-optimized mixtures. These optimized mixtures eventually performing better than their non-optimized mixtures (showing lower shrinkage) by the 32-week test date as shown in Figure 4.49 and Figure 4.50.

Table 4.36: Volumetric shrinkage results and average percent difference of optimized and non-optimized straight cement and fly ash replacement mixtures microstrains by w/cm ratio

Mixture Characteristic	Mixture type	28 Day (microstrain)	8 Week (microstrain)	16 Week (microstrain)	32 Week (microstrain)
All	Non-optimized	299	373	445	763
	Optimized	372	487	552	603
	Average percent different	19.1%	23.1%	18.9%	-29.8%
Straight cement	Non-optimized	303	375	452	728
	Optimized	368	497	562	611
	Average percent different	17.5%	24.4%	18.9%	-24.8%
Fly ash replacement	Non-optimized	298	375	440	795
	Optimized	379	482	544	593
	Average percent different	20.1%	22.2%	19.0%	-34.8%

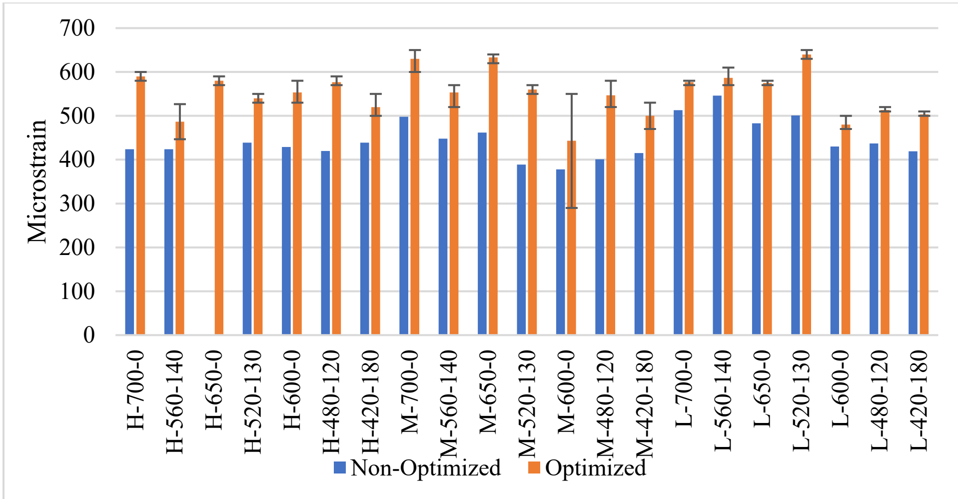

Figure 4.49: 16-week volumetric shrinkage measurements (in microstrain) for optimized and non-optimized aggregate gradation mixtures, 28 day as initial reading

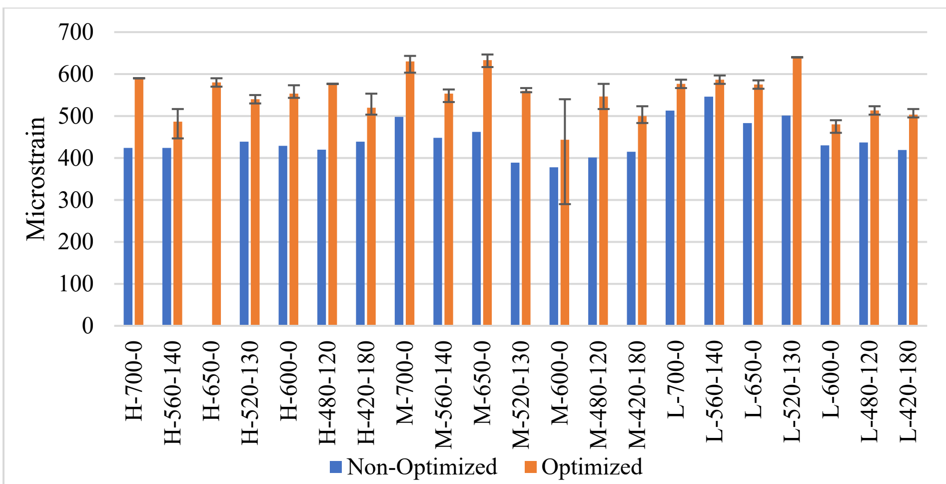

Figure 4.50: 32-week volumetric shrinkage measurements (in microstrain) for optimized and non-optimized aggregate gradation mixtures, 28 day as initial reading

Table 4.37 shows the average volumetric shrinkage (in microstrain), along with the percent difference in volumetric shrinkage between optimized aggregate gradation mixtures and non-optimized mixtures grouped by cementitious content. From this table, it can be observed that optimized aggregate gradation mixtures exhibited lower shrinkage at later dates. Both optimized and non-optimized mixtures exhibited lower shrinkage as the cementitious material content decreases. However, optimized aggregate gradation mixtures with 650* pcy of cementitious material had slightly more volumetric shrinkage than the average shrinkage than the average of optimized 700* pcy mixtures until the 32-week test date.

Table 4.37: Average volumetric shrinkage and average percent difference of optimized and non-optimized mixtures grouped by w/cm ratio

Cementitious material (pcy)	Mixture type	28 Day (microstrain)	8 Week (microstrain)	16 Week (microstrain)	32 Week (microstrain)
700	Non-optimized	319	401	476	798
	Optimized	373	496	571	627
	Average percent different	13.8%	18.4%	16.5%	-29.4%
650	Non-optimized	310	385	455	828
	Optimized	387	513	588	622
	Average percent different	20.7%	25.5%	22.8%	-37.0%
600	Non-optimized	280	348	419	733
	Optimized	360	464	515	572
	Average percent different	21.7%	24.8%	18.4%	-27.7%

Table 4.38 shows the average volumetric shrinkage (in microstrain) and the percent difference in shrinkage between optimized aggregate gradation mixtures and non-optimized mixtures grouped by w/cm ratio. The trend that optimized aggregate gradation mixtures exhibit lower shrinkage than non-optimized mixtures at later dates is visible again. Non-optimized aggregate gradation mixtures showed an increase in average shrinkage as the w/cm decreased. This trend was also evident in optimized aggregate gradation mixtures at 28-day and 8-week testing dates, however, the volumetric shrinkage stabilized at the 16-week testing date with average volumetric shrinkage results being very similar across all w/cm ratios.

Table 4.38: Average volumetric shrinkage and average percent difference of optimized and non-optimized mixtures grouped by cementitious material content

W/c ratio	Mixture type	28 Day (microstrain)	8 Week (microstrain)	16 Week (microstrain)	32 Week (microstrain)
0.47	Non-optimized	277	348	429	709
	Optimized	342	470	550	617
	Average percent different	17.7%	24.9%	20.8%	-17.3%
0.42	Non-optimized	300	369	427	750
	Optimized	385	502	552	592
	Average percent different	21.4%	26.0%	22.3%	-29.5%
0.37	Non-optimized	318	398	476	852
	Optimized	388	490	554	587
	Average percent different	18.0%	18.5%	14.0%	-94.3%

When using the measurement after curing as the comparator reading, optimized aggregate gradation mixtures did not perform as well as their companion non-optimized mixtures at the early age tests. However, all optimized aggregate gradation mixtures did meet the AASHTO PP 84 420 microstrain requirement at the 28-day test except for H-520*-130, which had a 28-day microstrain of 440. All mixtures did meet the AASHTO PP 84 420 microstrain requirement at their 28-day test when using the measurement directly after demolding per ASTM C157.

4.2.2.5 Formation Factor

Since electrical test results can be influenced by pore solution chemistry, there is a desire of some practitioners in the field to account for changes in pore solution chemistry driven by use of a variety of cementitious materials. The Formation Factor is a test intended to compare concrete mixtures from a wide range of cementitious materials and SCMs by accounting for pore solution chemistry in the results and is calculated using the Bucket Test developed by Dr. Jason Weiss. As per AASHTO PP 84, a pore solution resistivity of 0.127 Ωm was assumed and used in the Formation Factor calculations of optimized aggregate gradation mixtures (AASHTO 2020). Table 4.39 through Table 4.41 provide 28- and 56-day test results for surface resistivity, Bucket Test, and Formation Factor. Table 4.39 shows test results of optimized aggregate gradation mixtures for 28- and 56-day surface resistivity, Bucket Test, and the calculated formation factor using the assumed resistivity of the pore solution of 0.127 Ωm from AASHTO PP 84 (AASHTO 2020). It is noted that the assumed resistivity of the pore solution has changed between the 2017 and 2020 versions of AASHTO PP 84 used for NCDOT RP 2018-14 and RP 2020-13 (this study), respectively. Table 4.40 shows a sample of the test results for 28- and 56-day surface resistivity, Bucket Test, and the calculated formation factor using the assumed resistivity of the pore solution of 0.10 Ωm from used previously by the research team (Cavalline et al. 2018). Table 4.41 shows the 28- and 56-day Formation Factor of all optimized aggregate gradation mixtures. It should be noted, Bucket Test and Formation Factor calculations for optimized aggregate gradation mixtures H-700*-0, H-560*-140, and H-650*-0 were not obtained due to a lack of specimens.

Table 4.39: 28- and 56-day surface resistivities and formation factors for optimized aggregate gradation mixtures

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	8.0	-	-	8.0	-	-
H-560*-140	7.1	-	-	9.3	-	-
H-650*-0	6.8	-	-	7.9	-	-
H-520*-130	6.2	17.8	1398	8.8	25.6	2012
H-600*-0	8.4	21.9	1720	9.1	22.0	1728
H-480*-120	7.0	19.1	1504	9.0	26.5	2087
H-420*-180	6.0	18.8	1480	10.5	27.8	2189
M-700*-0	8.6	24.8	1953	10.3	26.4	2075
M-560*-140	7.5	39.2	3087	11.6	38.7	3047
M-650*-0	8.6	27.0	2122	11.3	24.9	1961
M-520*-130	6.5	21.9	1724	9.1	29.9	2350
M-600*-0	8.5	25.0	1965	9.2	24.6	1933
M-480*-120	7.3	22.2	1748	9.6	32.4	2551
M-420*-180	8.8	27.3	2150	15.7	41.9	3299
L-700*-0	10.2	29.3	2307	12.5	31.6	2488
L-560*-140	10.6	27.5	2161	16.9	50.0	3933
L-650*-0	9.1	26.4	2079	12.1	36.0	2831
L-520*-130	9.5	30.1	2366	19.0	49.3	3882
L-600*-0	10.0	30.2	2374	13.4	34.4	2705
L-480*-120	12.0	34.8	2736	20.5	54.2	4268
L-420*-180	10.2	36.0	2835	19.7	45.4	3571

Table 4.40: 28- and 56-day surface resistivities and formation factors for non-optimized aggregate gradation mixtures

Bucket Test						
Mix 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	14.4	1450
L-700-0	9.3	10.4	1040	10.1	10.5	1050
L-420-180	8.4	10.1	1010	12.0	13.2	1320

Table 4.41: Formation factor results for optimized aggregate gradation and non-optimized aggregate gradation mixtures

Mixture ID	28 Day Formation Factor	56 Day Formation Factor
H-700*-0	-	-
H-560*-140	-	-
H-650*-0	-	-
H-520*-130	1398	2012
H-600*-0	1720	1728
H-480*-120	1504	2087
H-420*-180	1480	2189
M-700*-0	1953	2075
M-560*-140	3087	3047
M-650*-0	2122	1961
M-520*-130	1724	2350
M-600*-0	1965	1933
M-480*-120	1748	2551
M-420*-180	2150	3299
L-700*-0	2307	2488
L-560*-140	2161	3933
L-650*-0	2079	2831
L-520*-130	2366	3882
L-600*-0	2374	2705
L-480*-120	2736	4268
L-420*-180	2835	3571

Figure 4.51 shows all optimized mixtures Formation Factor plotted in comparison to the chloride ion penetrability classification from AASHTO PP 84 (located in Table 4.42). Figure 4.52 and Figure 4.53 display the 28- and 56-day optimized aggregate gradation mixture formation factors plotted against the same testing days surface resistivity.

Table 4.42: Chloride ion penetrability associated with various formation factor values

Chloride Ion Classification	Formation Factor Value
High	520
Moderate	520 - 1,040
Low	1,040 - 2080
Very Low	2,080 - 20,700
Negligible	20,700

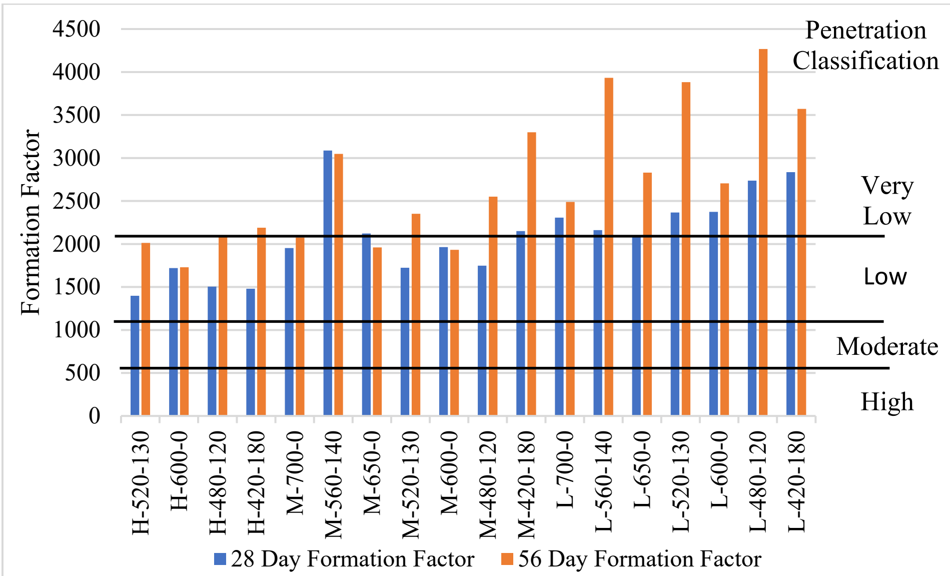

Figure 4.51: Formation factors for optimized aggregate gradation mixtures at 28- and 56-day

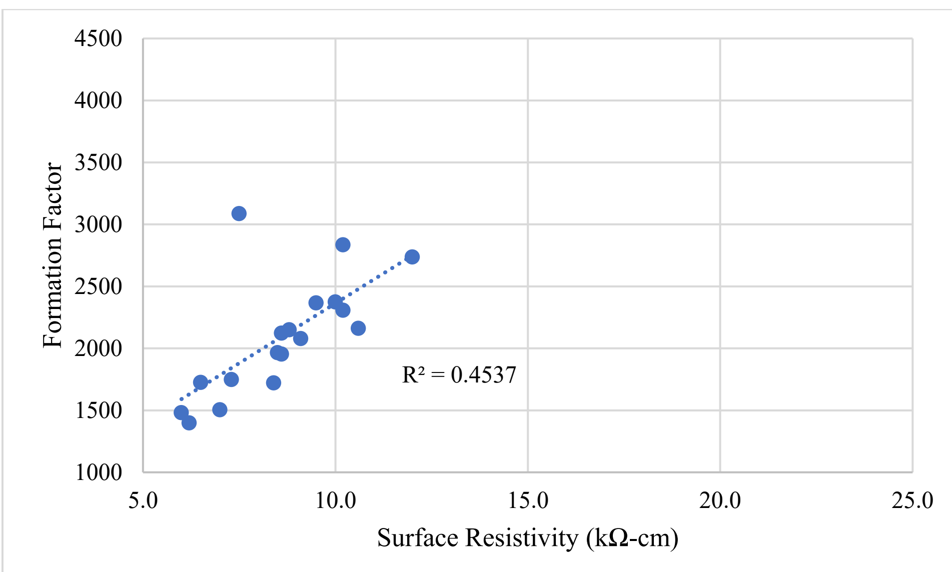

Figure 4.52: 28-day formation factor vs resistivity

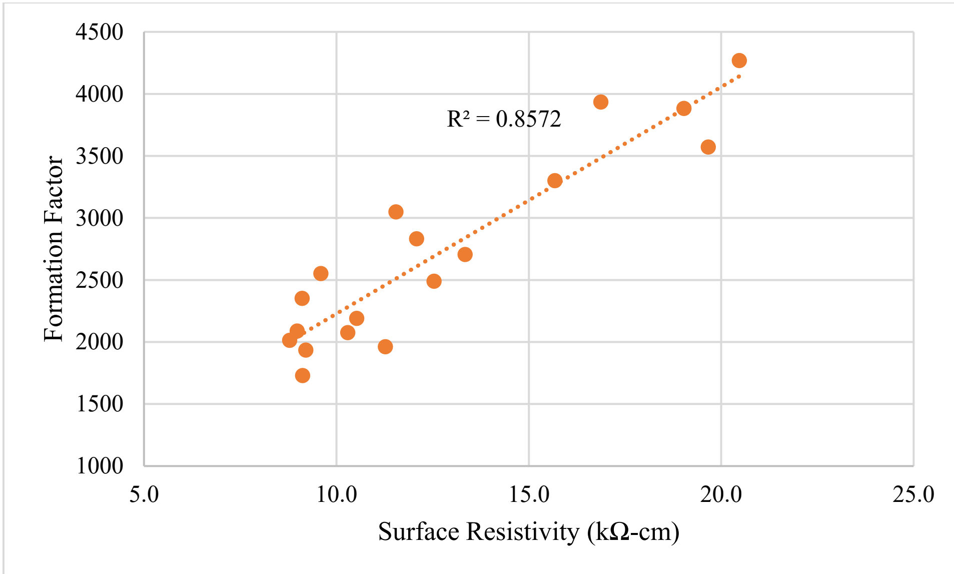

Figure 4.53: 56-day formation factor vs resistivity

All mixtures showed improved performance in at later testing dates, similar to surface resistivity and RCPT test results. The average formation factors for optimized mixtures with fly ash were only slightly higher than their companion optimized straight cement mixtures at 28 days (1.4%). However, the formation factors improved at the 56-day test mark considerably with an average formation factor 27.7 percent higher than their companion optimized straight cement mixtures.

Similar to a previous study performed by the research team which included two cements (this study only used one cement) and one SCM, the formation factor is corelated with resistivity and RCPT test results. The Formation Factor is a useful test when a used to compare a wide range of cementitious materials and SCMs are used. As the NCDOT continues to test mixtures with different cementitious systems, these results could be used to compare results and offer insights as to useful target values for future consideration if it is decided that Formation Factor testing is more useful than surface resistivity due to its ability to account for pore solution chemistry in the results.

CHAPTER 5: CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE WORK

This chapter presents the conclusions based upon test results that characterized 21 non-optimized aggregate gradation and 21 optimized aggregate gradation concrete mixtures that are otherwise similar in proportions. The mixtures were representative of mixtures used by the NCDOT for concrete pavement and structural elements such as bridge decks. This work has been completed in an effort to support the NCDOT's effort to research and implement QA and QC methods from the FHWA's PEM initiative. This chapter also provides recommendations for future research.

Overall, the analysis demonstrated that the cementitious content of concrete mixtures could be reduced by 10 percent and can still meet current NCDOT specification provisions. For moderate (w/cm = 0.42) and low (w/cm = 0.37) mixtures, many optimized gradation mixtures produce mechanical property test results that were similar to concrete mixtures that did not have a reduction in cementitious material. Electrical tests indicated greater permeability in optimized aggregate gradation mixtures, but the influence of the additional aggregate content, ITZ, and a 10% reduction in cementitious materials effectively changing the cement paste structure may have influenced the results. Such indirect measurements may require adjustments to performance targets for optimized mixtures. The use of optimized aggregate mixtures may reduce costs and emissions of greenhouse gases via the reduction of cement, and additionally may result in mixtures with improved durability characteristics, a longer service life, and lower cracking through volumetric shrinkage. While this research provided test results where optimized aggregate gradation mixtures with reduced cementitious materials demonstrated more permeable characteristics, it does not mean that these mixtures are actually less durable. This will be expanded upon in Section 5.2.

5.1 Conclusions

Laboratory test results for the 21 optimized aggregate gradation mixtures were compared to the test results for 21 non-optimized aggregate gradation mixtures that are similar to in proportions. Findings for each test are provided below:

Fresh Properties:

- Low (0.37) w/cm ratio and low (600* pcy) cementitious material content optimized aggregate gradation mixtures required the most WRA to achieve acceptable workability.
- Optimized aggregate gradation concrete mixtures had lower slumps than the companion non-optimized mixtures, with higher cement content mixtures requiring less WRA to achieve target slump.
- Low (0.37) w/cm ratio and low (600* pcy) cementitious material contentment optimized aggregate gradation mixtures required the most AEA to achieve the target range for entrained air.
- Unit weights of optimized aggregate gradation mixtures increased as the w/cm ratio decreased; this trend is also present in non-optimized aggregate gradation mixtures.
- Optimized aggregate gradation mixtures typically have a slightly higher unit weight than the companion non-optimized aggregate gradation mixtures.

Mechanical Properties:

- Mechanical properties of both non-optimized and optimized aggregate gradation mixtures improved as the w/cm ratio decreased.
- Mechanical properties of optimized aggregate gradation straight cement mixtures with reduced paste showed a negligible impact at all testing dates.

- Mechanical properties of optimized aggregate gradation mixtures of fly ash mixtures at 20 percent replacement rates showed a negligible impact to mechanical properties for tests at ages greater than 3-days, and mechanical properties of fly ash mixtures at 30 percent replacement rates showed negligible impact to mechanical properties for tests past 28-days. These findings may indicate that NCDOT should continue to encourage the use of SCMs at replacement rates of up to 30 percent.

Compressive and Flexural Strength

- In compressive strength testing, optimized aggregate gradation straight cement mixtures aged 3-days had an average 9.7 percent lower than companion non-optimized aggregate gradation straight cement mixtures. Optimized aggregate gradation straight cement mixtures exhibited compressive strengths more similar at later ages.
    - Average compressive strength test results of optimized aggregate gradation straight cement mixtures as the w/cm ratio decreased were more similar to the companion non-optimized aggregate gradation straight cement mixtures. Average compressive strengths of high (0.47) w/cm ratio straight cement optimized aggregate gradation mixtures were 19.7 percent lower at 3-day tests but were within 10 percent of non-optimized mixtures at all other test ages.
    - Average compressive strength test results of optimized aggregate gradation straight cement mixtures with 650* pcy of cementitious material were noticeably lower than their companion non-optimized aggregate gradation straight cement mixtures at ages of 3- and 7-day tests (22.6 percent and 16.7 percent respectively) but performed more similar to the non-optimized straight cement mixtures at later ages. However, the 700* pcy and 600* pcy optimized aggregate gradation straight

cement mixtures did not exhibit this trend and were within 10 percent at all ages, sometimes outperforming non-optimized companion mixtures.

    - Average 28-day MOR test results of optimized aggregate gradation straight cement mixtures showed essentially no difference in flexural strength than companion non-optimized aggregate gradation straight cement mixtures (3.5 percent lower).

- In compressive strength testing, optimized aggregate gradation mixtures with fly ash did not perform as well as their companion non-optimized aggregate gradation mixtures with fly ash at early test dates. However, the average compressive strength of all optimized aggregate gradation mixtures with fly ash did perform as well (within in 1 percent) as their companion non-optimized aggregate gradation fly ash mixtures at later testing dates.
    - The average compressive strength of optimized aggregate gradation fly ash (20 percent and 30 percent replacement rates) mixtures was 24.0 percent lower than companion non-optimized aggregate fly ash mixtures at 3-day tests. Optimized aggregate gradation mixtures with 20 percent fly ash showed a negligible difference compared to their companion non-optimized mixtures by the 7-day tests (6.3 percent lower), optimized aggregate gradation mixtures with 30 percent fly ash average compressive strengths was almost equivalent to non-optimized companion mixtures by the 28-day tests (0.7 percent lower).
    - Average compressive strengths of high (0.47) w/cm ratio optimized and non-optimized aggregate gradation fly 30 percent ash mixtures did not meet the 28-day NCDOT 4,500 psi requirement, but both mixtures had average compressive strength results above the 4,500 psi requirement by 56-day tests.

- The 28-day MOR of optimized aggregate gradation 20 percent fly ash mixtures showed no difference in flexural strength when compared to the companion non-optimized aggregate gradation 20 percent fly ash mixtures (3.3 percent lower). However, optimized aggregate gradation 30 percent fly ash mixtures had 28-day MOR 17.1 percent lower than companion non-optimized aggregate gradation 30 percent fly ash mixtures, with two mixtures not meeting the 28-day NCDOT requirement of 650 psi.

Modulus of Elasticity

- Measured 28-day MOE values of optimized aggregate gradation mixtures were 13.6 percent higher than companion non-optimized aggregate gradation mixtures
    - Average 28-day MOE values of optimized aggregate gradation mixtures were 14.3 percent lower than the MOE calculated using the ACI 318 equation; 11.9 percent lower than the MOE calculated with AASHTO LFRD equation C5.4.2.4-2. These differences were roughly consistent across all optimized aggregate gradation straight cement and fly ash mixtures.
    - The 28-day MOE of optimized aggregate gradation mixtures with fly ash showed similar trends of decreasing average 28-day MOE as fly ash content increased when compared to their companion non-optimized mixtures. The average 28-day MOE of optimized aggregate gradation mixtures with 20 percent and 30 percent fly ash decreased by 6.8 percent and 13.8 percent respectively when compared to their companion optimized straight cement mixtures. The average 28-day MOE of non-optimized aggregate gradation mixtures with 20 percent and 30 percent fly

ash decreased by 12.4 percent and 21.7 percent respectively when compared to their non-optimized straight cement mixtures.

Durability Performance:

- Durability performance test results improved as the w/cm ratio decreased in both optimized and non-optimized aggregate gradation mixtures, suggesting the NCDOT may want to further explore prescriptive specification provisions to reduce the w/cm ratio of their mixtures. This prescriptive change would result in less permeable concrete, lower shrinkage, potentially lower paste contents, and overall improved durability performance.
- Optimized aggregate gradation mixtures with fly ash exhibited improved durability performance characteristics at later ages when compared to companion optimized and non-optimized aggregate gradation mixtures, suggesting the NCDOT may want to explore prescriptive specifications to encourage the use of SCMs at replacement rates up to 30 percent to improve durability performance.

Volumetric Shrinkage

- Volumetric shrinkage of all optimized aggregate gradation mixtures met the AASHTO PP 84 suggested limit of 420 microstrain at 28-days.
- In the previous study, the research team used the measurement after the 28-day wet curing period as the initial measurement. Therefore, comparisons were made between companion optimized and non-optimized aggregate gradation mixtures using this approach, despite ASTM C157 indicating that the measurement immediately following demolding should be used as the initial measurement.

- Optimized aggregate gradation mixtures had higher average 28-day volumetric shrinkage (in microstrain) than companion non-optimized aggregate gradation mixtures.
- Optimized aggregate gradation mixtures had lower average volumetric shrinkage (in microstrain) than non-optimized aggregate by the 32-week measurement.

RCPT and Surface Resistivity

- RCPT results of optimized aggregate gradation mixtures were typically higher than the companion non-optimized aggregate gradation mixtures at both 28- and 90-day tests.
  - Average RCPT results of optimized and non-optimized aggregate gradation mixtures improved as the w/cm decreased.
  - The optimized aggregate mixtures with fly ash had higher RCPT results than straight cement mixtures at 28-day tests (5.1 percent higher than optimized straight cement mixtures and 25.1 percent higher than non-optimized straight cement mixtures), but had lower RCPT results than companion straight cement mixtures by the 90-day tests (64.4 percent lower than companion optimized straight cement mixtures and 16.7 percent lower than non-optimized straight cement mixtures).
- Average surface resistivity test results for optimized aggregate gradation mixtures were typically lower than the companion non-optimized aggregate gradation mixtures.
  - Average surface resistivity results of optimized and non-optimized aggregate gradation mixtures improved as the w/cm ratio decreased at later age tests. Non-optimized aggregate gradation mixtures exhibited this trend by the 28-day tests while optimized aggregate gradation mixtures exhibited this trend at all test dates.

- Average surface resistivity results of optimized aggregate gradation mixtures with fly ash out had higher resistivities than the companion optimized aggregate gradation straight cement mixtures by the 56-day tests.
- Average surface resistivity results of optimized aggregate gradation mixtures with fly ash out had similar resistivities than the companion non-optimized aggregate gradation straight cement mixtures by the 56-day tests (4.5 percent lower) and had higher average surface resistivities by the 90-day tests (7.2 percent higher).

5.2 Recommendations for Future Work

This section is to provide recommendations for future work, with a particular focus on gaining a better understanding of the results of electrical tests (surface resistivity tests) for concrete mixtures with optimized aggregate gradations, which provide a greater volume of coarse aggregate than is typical.

In this study, optimized aggregate gradation straight cement and 20 percent fly ash mixtures with reduced cementitious content and paste volume showed a negligible difference from companion non-optimized aggregate gradation concrete mixtures for the mechanical property tests. Optimized aggregate gradation mixtures with 30 percent fly ash showed negligible difference from companion non-optimized mixtures for mechanical property tests at later ages. The economic and environmental benefits of using concrete mixtures have been demonstrated. However, studies completed by Oklahoma State University and Rowan University corroborate the findings of this research study, indicating that the additional aggregates in optimized aggregate gradation concrete mixtures may influence the measurements made in electrical tests. This could be due to the increased volume of the interfacial transition zone, the increased aggregate volume, or other factors.

Future research projects to support NCDOT's PEM initiatives may want to explore the impacts of increased aggregate volumes, SCMs, and chemical admixtures on electrical tests for concrete and identify targets for these tests that will accurately predict their field durability. Suggested work includes examining permeability of optimized aggregate gradation mixtures using non-electrical permeability tests such as the Germann Water Permeation Test (GWT). The GWT is a non-electrical test that measures the permeation of water into the surface under an applied pressure. By comparing concrete mixtures with a varying volume of coarse aggregates electrical test results with a non-electrical test, more accurate targets for surface resistivity test results could be developed.

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Appendix: Supplemental Information for Chapter 3

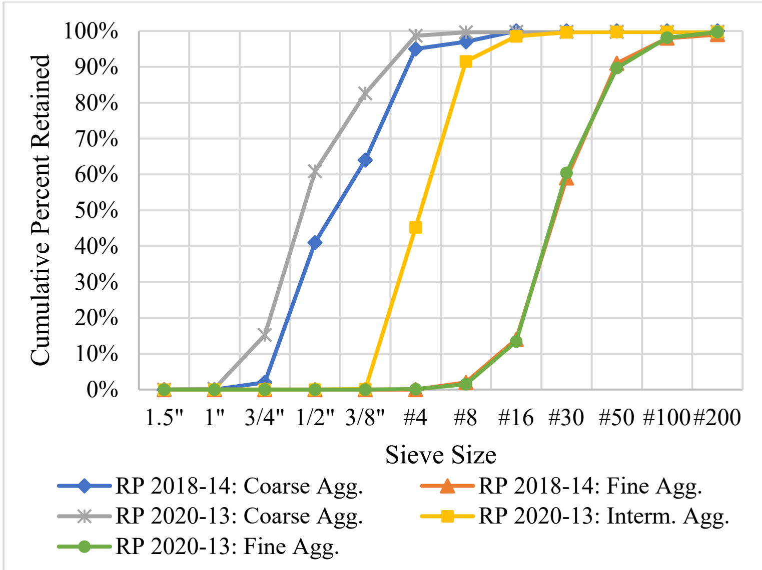

Figure A.1: Cumulative Percent Retained

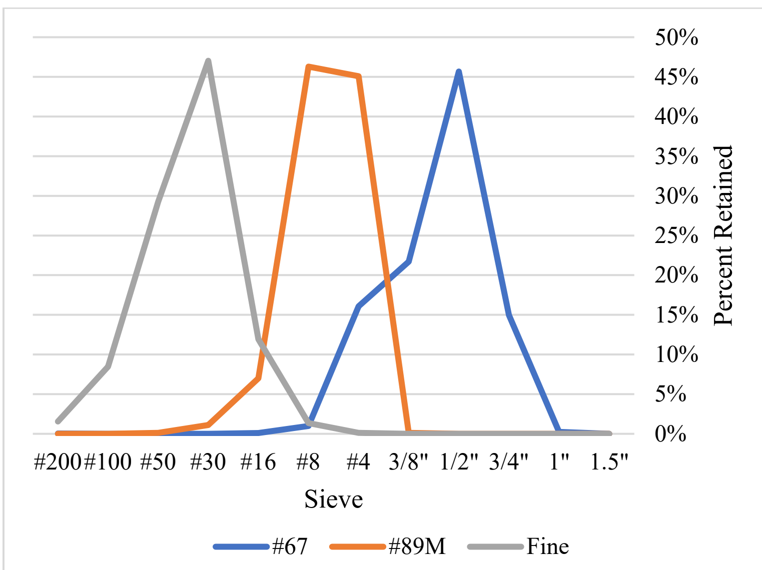

Figure A.2: #67, #89M, and fine aggregate percent retained

Table A.1: Aggregates percent retained

Percent Retained			
Sieve Number	#67	#89M	Fine
1.5"	0%	0%	0%
1"	0%	0%	0%
3/4"	15%	0%	0%
1/2"	46%	0%	0%
3/8"	22%	0%	0%
#4	16%	45%	0%
#8	1%	46%	1%
#16	0%	7%	12%
#30	0%	1%	47%
#50	0%	0%	29%
#100	0%	0%	8%
#200	0%	0%	2%

Table A.2: Coarse aggregate calculated properties

Property	Sample 1	Sample 2	Sample 3	Average
Bulk SG	2.64	2.62	2.65	2.64
Bulk SG (SSD)	2.66	2.68	2.68	2.67
Absorption (%)	0.72%	2.29%	1.21%	1.41%

Table A.3: Intermediate aggregate calculated properties

Property	Sample 1	Sample 2	Sample 3	Average
Bulk SG	2.60	2.67	2.70	2.66
Bulk SG (SSD)	2.68	2.74	2.77	2.73
Absorption (%)	2.90%	2.70%	2.68%	2.76%

Table A.4: Fine aggregate calculated properties

Property	Sample 1	Sample 2	Sample 3	Average
Bulk SG	2.64	2.61	2.62	2.62
Bulk SG (SSD)	2.68	2.64	2.65	2.66
Absorption (%)	1.81%	1.46%	1.18%	1.48%

Report of Results for Product Testing *June 4, 2020*
Product Name: Roxboro DS 11/23-12/11 (2015)
TEC Services Project #: 20-1612
TEC Laboratory #: 20-461

Table 1 – Results of the Chemical Analysis

Oxides	Results Weight (%)
Silicon Dioxide (SiO_2)	**52.7**
Aluminum Oxide (Al_2O_3)	**26.7**
Iron Oxide (Fe_2O_3)	**11.12**
Sum ($SiO_2 + Al_2O_3 + Fe_2O_3$)	**90.5**
Calcium Oxide (CaO)	**2.1**
Magnesium Oxide (MgO)	**1.1**
Sodium Oxide (Na_2O)	**0.34**
Potassium Oxide (K_2O)	**2.24**
Equivalent Alkalies (Na_2O+0.658 K_2O)	**1.81**
Titanium Dioxide (TiO_2)	**1.42**
Manganic Oxide (Mn_2O_3)	**0.026**
Phosphorus Pentoxide (P_2O_5)	**0.21**
Sulfur Trioxide (SO_3)	**0.75**
Loss on Ignition	**1.9**
Moisture Content	**0.38**

Figure A.3: Belews Creek fly ash chemical analysis