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FINAL REPORT

VIABILITY OF CONCRETE PERFORMANCE-BASED SPECIFICATION FOR GEORGIA DOT PROJECTS

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Final Report

VIABILITY OF CONCRETE PERFORMANCE-BASED SPECIFICATION FOR GEORGIA DOT PROJECTS

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16. Abstract:

Performance-based specifications provide an alternative means of designing concrete to meet specific performance-related goals. Instead of specifying materials, means, and proportions, as current prescriptive specifications do, a performance-based specification instead indicates the functional requirements for concrete in the plastic and hardened states and allows the concrete producer to design a mixture that meets those requirements. Research conducted in this project has shown that by focusing on achieving specific performance objectives, it is possible to produce more durable concrete mixtures with longer service lives and lower costs than mixtures made under current prescriptive requirements. Based on the results of this research effort, recommendations are proposed for future introduction of performance-based options into the GDOT Section 500 – Concrete Structures Standard Specification. It is recommended that performance criteria for permeability and dimensional stability be included in future GDOT specifications as optional requirements to supplement the existing prescriptive requirements for concrete structures.

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TABLE OF CONTENTS

Page

LIST OF TABLES

LIST OF FIGURES

EXECUTIVE SUMMARY

Research conducted in this project has shown that performance-based concrete specifications can lead to tailor-made concrete mixtures that focus on specific performance objectives, potentially leading to more durable concrete with longer service lives and lower costs than mixtures made under current prescriptive requirements.

A review of current performance-based specifications used in the United States and throughout the world was conducted to identify the "best practices" for future GDOT specifications. It was found that the most successful approach in regions with varying geography and climate has been to adopt an exposure class system that details performance requirements based on the type and severity of environmental exposures, with requirements typically specified for exposures to chloride ions, sulfates, and freezing and thawing.

Based on the review of best practices, it was determined that permeability, strength, and dimensional stability were the three most important criteria to examine in this experimental study. Twelve initial concrete mixtures were prepared for permeability testing, using both the AASHTO T277/ASTM C1202 Rapid Chloride Permeability Test and the AASHTO TP95 Surface Resistivity Test. The results show that low-permeability concrete with longer service lives and lower life cycle costs could be tailor-made using binary and ternary blends containing supplementary cementitious materials (fly ash and metakaolin) and interground limestone powder.

The effect of regional variations in geology of aggregates was also considered as a potential influence on concrete performance. Four different aggregate pairings were selected to represent concrete produced both above and below the state's fall line. For each aggregate combination, a prescriptive concrete mixture meeting the requirements of GDOT Section 500 Standard Specifications for Class AA concrete was compared to a performance-based concrete mixture consisting of limestone and fly ash. It was found that for each aggregate combination, the performance-based mixtures designed to achieve low permeability exhibited better performance in the surface resistivity and Rapid Chloride Permeability tests, but the prescriptive mixtures had better performance in the compressive strength and drying shrinkage tests. The specific aggregate source was found to play a significant role in the compressive strength and permeability measured for each concrete mixture, but had less of an effect on the 28-day drying shrinkage.

Based on the results of this research effort, recommendations are proposed for future introduction of performance-based options into the GDOT Section 500 – Concrete Structures Standard Specification. It is recommended that performance criteria for permeability and dimensional stability be included in future GDOT specifications as optional requirements to supplement the existing prescriptive requirements for concrete structures.

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1. INTRODUCTION

1.1 Purpose of Research

Currently, GDOT's concrete specification (Section 500 – Concrete Structures) is primarily a prescriptive specification. That is, limits are provided for the quantities of the various mix components (e.g., minimum cementitious materials content, maximum supplementary cementitious material content), as well as their proportions (e.g., maximum water-to-cement ratios). The overall goal of such specifications is to achieve specific performance criteria, through control of materials selection and mixture proportioning, rather than by specifying limits on performance directly.

While such specifications are common, there has been a gradual industry-wide shift from prescriptive specifications toward performance-based specifications ("P2P") because of increasing belief that the prescriptive specifications in practice limit innovation. For example, using only prescriptive specifications does not allow for the use of some new materials and approaches to design which may be more cost effective, either initially or in the long-term through increased durability, while still achieving the same minimums in performance measured for comparable mixes under current specifications. The use of performance-based or end-result specifications may increase quality and sustainability while lowering project costs through innovations in materials selection and proportioning [1].

The difficulty with performance-based specifications is identifying the measurable performance requirements and how the performances are measured.

Currently, while no national performance-based specification for concrete exists in the United States, the National Ready Mixed Concrete Association [2] has published a "guide to improving concrete specifications" which includes commentary on performance, and some states have moved to provide performance-based requirements for all or some concrete specifications. In Canada, performance-based specifications are included as an annex to the most recent specification for concrete materials and methods for concrete construction [3]. Performance-based specifications for concrete have also been developed and are being used in Europe, Australia, and the Middle East [4]. Approaches found to be successful in other countries, as well as experience gained by states developing performance-based specifications, can be examined for their relevance to GDOT needs and applicability to regional materials and practices.

1.2 Objectives

The objectives of this research effort include:

(1) to provide a review of current "best practices" for the development and implementation of performance-based specifications for concrete structures for transportation projects,

(2) to perform a detailed analysis comparing initial and long-term costs associated with prescriptive and performance-based designs, and

(3) to generate guidelines for incorporating performance-based specification as an option in Section 500 – Concrete Structures.

The results of this research will be used to generate draft performance-based specifications as a potential alternative to Georgia's existing Section 500 specification.

The performance-based specifications will be drafted through examination of other countries' and states' experiences with performance-based specification and with understanding of the implications of these specifications on concrete quality and cost. The adoption of a performance-based option follows current trends in the construction industry and is anticipated to allow Georgia DOT to obtain concrete of better quality and at lower cost, while promoting innovation in the industry.

1.3 Organization of Report

Chapter 2 of this report reviews existing performance-based specifications used worldwide and within the United States and identifies the "best practices" for development and implementation of performance-based specifications for concrete structures. Requirements for strength and other functional properties are reviewed, and field and laboratory methods for assessment of these properties are compared. Interviews conducted with DOT personnel, materials suppliers, concrete producers, and contractors are also included to consider their opinions on potential impacts of performance-based specifications on concrete quality, performance, and cost.

Chapters 3 and 4 compare prescriptive and performance-based concrete designs to better understand the implications of performance-based specifications on initial cost and long-term cost (e.g., cost savings achieved via extended service life vs. costs for maintenance and repair). Test methods identified as best practices in Chapter 2 are employed to assess the permeability, mechanical properties, and dimensional stability of the concrete mixtures. Chapter 3 primarily focuses on the effect of variations in binder composition on concrete properties, while Chapter 4 instead considers regional variations in aggregate types and sources.

Chapter 5 summarizes the conclusions of the study, and Chapter 6 provides recommendations for incorporating a performance-based specification option into the Section 500 – Concrete Structures specification.

2. LITERATURE REVIEW

Most concrete specifications currently in place are prescriptive in nature: "recipes" that specify in mandatory language the processes, materials, proportions, and methods that must be used to achieve a desired product. These specifications are based on past experience and primarily rely upon empirical or implied relationships between the specified materials and processes and the final in-place concrete performance. Prescriptive specifications tend to be conservative, often requiring higher cement contents and lower water-to-cementitious materials ratios (w/cm) than are actually needed to obtain the required performance characteristics. As a result, prescriptive mixes may be more expensive, as more cement must be used than may be necessary, and potentially less durable, as adherence to a prescriptive specification does not necessarily guarantee good long-term performance. Furthermore, because prescriptive specifications typically only address minimum compressive strength as an indicator of concrete's performance, other considerations such as chloride penetration resistance and dimensional stability or crack resistance are often ignored. Given that there is a growing demand for more durable structures with service lives exceeding 75 to 100 years, a shift from prescriptive specifications to more durability-minded performance specifications is in order [5]. This shift to performance specifications is already underway in many countries worldwide, including Australia, Canada, and South Africa, and is slowly gaining acceptance in the United States as federal and state agencies begin to adopt performance-based alternatives to their current concrete specifications.

The Canadian Standards Association (CSA) defines a performance specification as “a method of specifying a construction product in which the final outcome is given in mandatory language, in a manner that the performance requirements can be measured by accepted industry standards and methods. The processes, materials, or activities used by the contractors, subcontractors, manufacturers, and materials suppliers are then left to their discretion” [3]. Unlike a prescriptive specification, where limits are placed on the types and quantities of cementing materials, aggregates, and admixtures used, a performance specification simply specifies the desired outcome and allows the concrete supplier and contractor to work together to design a mixture that conforms to specified performance requirements. The flexibility in design allows for the use of unique materials, local materials, and combinations of materials currently not allowed under prescriptive specifications, and it may lead to more economical and innovative designs with improved long-term durability [6].

This is not to say, however, that performance-based specifications are the “best” or the only means of designing durable concrete. In many instances, prescriptive specifications are actually favorable in terms of production, cost, and performance. For example, a prescriptive approach may be reasonable for small projects where performance testing may be expensive or impractical or for projects where the relationships between the specified materials, means, and methods and the required outcomes are already well-established [7]. Rather than completely replacing prescriptive specifications with performance specifications, specifiers should seek a balance between prescriptive and performance specifications, providing performance-based alternatives where properties other than strength, such as long-term durability, are of concern.

2.1 Features of a performance specification

At its core, a performance specification contains the following [6,7]:

1. States in mandatory language the functional requirements of the hardened concrete,
2. Indicates test methods and limits relevant to the appropriate performance requirements,
3. Provides a clearly defined procedure for the qualification and acceptance of concrete in both the fresh and hardened states, and
4. Defines actions to take in the event of non-compliant performance.

Accompanying each functional requirement should be a standard test procedure and limits within which the concrete product must fall. For example, a low-permeability concrete may be specified with a requirement that the charge passed during an ASTM C1202 Rapid Chloride Permeability Test (RCPT) be below 1000 coulombs, while a freeze-thaw resistant concrete may be specified with a durability factor of at least 90 when tested using ASTM C666, Method A. While the goal is to evaluate the concrete's suitability using direct performance indicators, reliable tests may not always be available or economically viable, and other durability issues may not be apparent or easy to define at the time the specifications are formulated. At times, therefore, it may be more cost-effective to impose certain prescriptive requirements, such as limiting chloride content rather than directly measuring corrosion rate [8], or to allow the use of surrogate testing, where a relationship between one easily measured property, such as strength, can be correlated to a desired performance indicator, such as permeability [7].

One of the biggest challenges facing performance testing, however, is the long lead times that are often needed to obtain accurate results. RCPT, for instance, requires 28 to 56 days of curing before the concrete's performance may be evaluated, and freeze-thaw testing requires a full 90 days of evaluation; other indicators, such as alkali-aggregate reactivity, may require even longer lead times, up to one year (or more) in some cases. To accommodate the long lead times these tests may require, a performance specification should also include provisions for the prequalification of mixtures on the basis of historical records of performance or based on the results of laboratory performance testing. For example, a potentially reactive aggregate may be used in combination with supplementary cementitious materials (SCMs) if it can be shown that concrete prism expansions per ASTM C1293 are within the acceptable limits. Similarly, a previously approved mixture that resulted in satisfactory performance in the plastic and hardened states may be approved for use on a future project requiring the same durability criteria without the need for additional testing as long as it is shown that all current materials and construction practices are equivalent.

Once the concrete is delivered to the construction site, identity testing may be performed to verify that the fresh properties of the delivered concrete (e.g., water content, density, air content, and workability) are consistent with those measured during prequalification. Mixtures whose properties do not conform to those of the prequalified mixture would be rejected at the point of delivery prior to placement. Acceptance testing conducted on samples prepared at the time of placement and on cores of the in-place concrete may also be used to verify the in-place properties of the hardened concrete, and any instances of non-compliance may be addressed as they arise [7]. Although identity and acceptance testing alone cannot guarantee the long-term performance of the concrete product, they can serve as indicators that the delivered concrete is consistent with the prequalified mixture and will perform as required. Instances of non-compliance are addressed as they arise.

2.2 Current international practices

Performance-based specifications have been adopted in a number of industrialized nations worldwide, including Australia, Canada, South Africa, and nations of the European Union. Nearly every specification includes a permeability requirement, most often citing a chloride permeability test (e.g., ASTM C1202) or a surface resistivity test (e.g., AASHTO TP95) as an indicator of satisfactory permeability for a specified service environment. Limiting the transport of ionic species through concrete is vital to ensuring adequate durability and long service lives [9], which is why this is the most frequently specified performance criterion worldwide. Most specifications also include a required minimum compressive strength, although it has been argued that this provision is unnecessary since mixes that satisfy the permeability requirements typically will also have sufficient strength [6, 7, 10]. Other frequently cited criteria are air void system parameters (air content and spacing factor), alkali-aggregate reaction (AAR) resistance, shrinkage limits, and abrasion resistance [6].

In many instances, the specifications adopt an “exposure class” approach, in which performance requirements are based on specific environmental exposures to which the structure may be subjected. The most frequently cited classes are exposures to chlorides, sulfates, freezing and thawing, and aggressive chemicals, but other classes and combinations of classes may also be specified. The three countries discussed below all adopt an exposure class approach, but implement different rating scales and requirements for each subclass.

2.2.1 Australia

Australia was one of the first countries in the world to adopt performance-based specifications [12]. Sorptivity limits began to be specified for specialized projects in the 1990s to limit the permeability of concrete, but specifications otherwise remained prescriptive in nature. In 2000, the Australian standards committee amended Australian Standard (AS) 1379, “Specification and Supply of Concrete”, to consider both prescriptive and performance-based options. Two grades of concrete are defined [6]:

1. *Normal grade concrete*, a prescriptive class that is primarily specified by its compressive strength, slump, maximum size aggregate (MSA), placement method, and air-entrainment requirements.
2. *Special grade concrete*, a prescriptive *or* performance-based class requiring characteristics that differ from those of normal grade concrete. If the performance-based option is selected, quality and volume must be also specified, and the concrete producer has the right to refuse to accept an order that is performance-based.

In all cases, quality assessment must be performed to ensure that statistical strength requirements are met, and producers must submit documentation every six months to indicate that their mixes satisfy performance-based shrinkage requirements and prescriptive chloride and sulfate content limits [12].

The AS 3600 standard, “Concrete Structures”, also specifies exposure classes based on climate (e.g., tropical), geography (e.g., coastal), and environmental exposure (e.g., above ground). First, the concrete is categorized based on environmental exposure [6]:

1. Concrete in contact with the ground,
2. Concrete in interior environments,
3. Concrete above ground,
4. Concrete in contact with water, and
5. Concrete in other environments.

Within each environmental exposure, the concrete is further subdivided based into categories based on geography and climate. The “above ground” exposure, for example, includes subcategories for [11]:

1. Structures within 1 km (0.6 mi) of coastline,
2. Structures within 1 to 50 km (0.6 to 31 mi) of coastline, and
3. Structures more than 50 km (31 mi) from coastline:

   3a. within 3 km (1.9 mi) of an industrial polluting area,

   3b. in a tropical zone,

   3c. in a temperate zone, or

   3d. in an arid zone.

The subcategories then direct the specifier to an overall exposure class for the concrete (U, A1, A2, B1, B2, or C), for which a combination of performance and prescriptive requirements, including strength, cover, chemical content, freeze-thaw resistance, and curing practices is specified [6, 11]. For concretes with marine exposures, additional practices are recommended to guard against corrosion of the reinforcement, including prequalification of mixtures on the basis of sorptivity, permeability, and/or chloride diffusion testing and additional prescriptive requirements for strength, w/cm, and binder content [6, 12].

2.2.2 South Africa

In South Africa, performance specifications adopt a “durability index” approach, which assesses the quality of concrete based on the expected time to initiate corrosion [13]. Exposure classes are once again specified based on environmental conditions, but these classes only consider corrosion as an indicator of durability. Exposure classifications are [14]:

1. Concrete exposed to airborne salts,
2. Permanently submerged structures,
3. Structures permanently submerged on one side,
4. Concrete in tidal splash and spray zones, and
5. Concrete subject to chloride induced corrosion.

These environmental conditions are then combined with knowledge of the binder composition, cover depth, and required service life to determine a required durability index. The in-place concrete’s compliance with the required durability indices is evaluated based on oxygen permeability, water sorptivity, and chloride conductivity testing, with results ranging from “excellent” to “very poor” [14].

2.2.3 Canada

In 2004, the Canadian Standards Association (CSA) revised their CSA A23.1/A23.2 standard to offer two options for the specification of concrete. The owner may specify either:

1. A *performance* requirement, in which "the owner requires the concrete supplier to assume responsibility for performance of the concrete as delivered and the contractor to assume responsibility for the concrete in place," or
2. A *prescriptive* requirement, in which "the owner assumes responsibility for the concrete" [3].

Regardless of the option selected, the designer must specify the severity of exposure for each of five exposure classifications:

1. Chloride exposure (C),
2. Freeze/thaw exposure (F),
3. Neither chloride nor freeze/thaw exposure (N),
4. Chemical exposure (A), and
5. Sulfate exposure (S).

Each exposure is accompanied by prescriptive requirements for the concrete including minimum w/cm ratio, minimum compressive strength, air content, curing type, and permeability limits. Additional performance requirements for sulfate exposure are shown in Table 1. The standard states that while each environmental exposure should be considered when designing a performance-based specification, it is ultimately the responsibility of the design professional to define the appropriate performance limits and to address any additional exposures to which the structure may be subject [3].

Table 1. CSA performance requirements for sulfate exposure, Class S [3].

Class	Description	Water-soluble sulfate in soil, % by weight	Sulfate in groundwater, ppm	Water-soluble sulfate in recycled aggregate, % by weight	Maximum CSA A3004-C8 Expansion
S-1	Very severe	$SO_4 > 2.00$	$SO_4 > 10{,}000$	$SO_4 > 2.00$	0.05% at 6 mo. 0.10% at 12 mo.
S-2	Severe	$0.20 \leq SO_4 \leq 2.00$	$1500 \leq SO_4 \leq 10{,}000$	$0.60 \leq SO_4 \leq 2.00$	0.05% at 6 mo. 0.10% at 12 mo.
S-3	Moderate	$0.10 \leq SO_4 < 0.20$	$150 \leq SO_4 < 1500$	$0.20 \leq SO_4 < 0.60$	0.10% at 6 mo.

2.3 Current practice in the United States

At the present time, most concrete specifications in the United States are prescriptive in nature. Recent initiatives by the Federal Highway Administration (FHWA), the American Concrete Institute (ACI), and the National Ready Mixed Concrete Association (NRMCA) have encouraged the adoption of performance-based specifications at the state and federal levels, and several states have adopted or are in the process of adopting such alternative specifications.

<u>2.3.1 FHWA</u>

An initiative by FHWA from 2004 to 2008 sought to make performance specifications the new standard for federal highway projects [5]. These specifications are termed “performance-related specifications” (PRS) and are defined by FHWA as “specifications for key materials and construction quality characteristics (M&C factors) that have been demonstrated to correlate significantly with long-term performance of the finished work” [15]. The PRS typically specified for FHWA projects include requirements for 28-day compressive strength, slab thickness, air content, and pavement roughness, from which computer models may be used to determine expected service lives. In regions where chloride ingress is of concern, for example in a coastal region, permeability limits may also be specified.

Unlike true performance specifications, PRS typically also contain prescriptive requirements for minimum cementitious materials, maximum w/cm, aggregate gradations, and limits on supplementary cementitious materials (SCMs). In this sense they are *hybrid* specifications, containing both performance and prescriptive components. Nevertheless, PRS have been successfully implemented on federal highway projects in Florida, Indiana, Tennessee, and Wisconsin, with only minor criticisms relating to the pay adjustments for satisfactory performance of the finished product [16-22].

2.3.2 NRMCA and ACI

In January 2006, the National Ready Mix Concrete Association (NRMCA) issued a state-of-the-art review of international performance-based specifications in what is commonly referred to as the Prescriptive-to-Performance (P2P) Initiative Phase I Report [6]. The P2P Phase I report surveys performance-based criteria used in more than 30 countries worldwide, discusses the available test methods for performance evaluation, and outlines a multi-step plan of action to transition the United States from prescriptive to performance specifications. As part of this initiative, the NRMCA made a series of recommendations to the American Concrete Institute (ACI) to amend its ACI 318 “Building Code Requirements for Structural Concrete” and ACI 301 “Specifications for Structural Concrete” to include a clearer system of exposure classes and to expand performance-based options and provisions.

In 2008, exposure classes were introduced to Chapter 4, "Durability Requirements," of the ACI 318 Building Code. Like the Canadian classification system, the new Chapter 4 requires that design professionals specify durability on the basis of exposure to freezing and thawing cycles (F), sulfates (S), and chlorides (C). A fourth class, P, may also be specified when class F, S, and C exposures do not apply but when low permeability to water is still desired. The prescriptive requirements specified in the ACI 318-11 Building Code are provided in Table 2 through Table 5. Exposure classes S1-S3 additionally limit the types and amounts of cementitious materials that may be used, but the Code permits the use of alternative combinations of cementitious materials provided that ASTM C1012 sulfate expansions are within the limits specified in Table 6 [23].

Table 2. ACI 318-11 requirements for concrete exposed to freezing and thawing [23].

Class	Description	Maximum w/cm	Minimum f'_c[1], psi	Minimum air content[2], %
F0	Not applicable	N/A	2500	N/A
F1	Moderate	0.45	4500	3.5 – 6
F2	Severe	0.45	4500	4.5 – 7
F3	Very severe	0.45	4500	4.5 – 7

[1]Measured at 28 days or as specified.
[2]Varies with maximum size aggregate.

Table 3. ACI 318-11 requirements for concrete exposed to sulfates [23].

Class	Description	Maximum w/cm	Minimum f'_c[1], psi
S0	Not applicable	N/A	2500
S1	Moderate	0.50	4000
S2	Severe	0.45	4500
S3	Very severe	0.45	4500

[1]Measured at 28 days or as specified.

Table 4. ACI 318-11 requirements for concrete requiring low permeability to water [23].

Class	Description	Maximum w/cm	Minimum f'_c[1], psi
P0	Not applicable	N/A	2500
P1	Required	0.50	4000

[1]Measured at 28 days or as specified.

Table 5. ACI 318-11 requirements for concrete exposed to chlorides [23].

Class	Description	Maximum w/cm	Minimum f'_c[1], psi	Maximum water-soluble chloride content in concrete, % by weight of cement	
				Reinforced concrete	Prestressed concrete
C0	Not applicable	N/A	2500	1.00	0.06
C1	Moderate	N/A	2500	0.30	0.06
C2	Severe	0.40	5000	0.15	0.06

[1]Measured at 28 days or as specified.

Table 6. ACI 318-11 optional performance requirements for concrete exposed to sulfates [23].

Class	Description	Maximum ASTM C1012 Expansion
S0	Not applicable	N/A
S1	Moderate	0.05% @ 6 months
S2	Severe	0.05% @ 6 months or 0.10% @ 12 months
S3	Very severe	0.10% @ 18 months

With the exception of the sulfate exposure class, the current ACI durability requirements are still predominantly prescriptive specifications. While the adoption of exposure classes is certainly a push in the right direction when it comes to specifying concrete for durability, the current prescriptive nature of the ACI Code may still limit the compositions and ultimately the service lives of the concrete mixtures produced.

In 2008, the NRMCA issued a “Guide to Specifying Concrete Performance,” commonly referred to as the P2P Phase II Report [24]. The report is written in the format of a performance specification and is based upon the ACI 318-08 and ACI 301-05 durability requirements previously discussed. The exposure class definitions remain the same, but instead of specifying the minimum compressive strength and maximum w/cm as is done in the Code, the Phase II report provides alternative performance requirements based upon the maximum chloride penetrability as measured by the ASTM C1202 test. Additional performance-based alternatives are specified for freeze-thaw durability and sulfate resistance, as shown in Table 7 through Table 10 [24].

Although the P2P Phase II report provides a good model for future performance-based specifications, one criticism of the proposed alternatives is that the 56 day permeability limits are all within the “moderate” range of chloride penetration resistance for the ASTM C1202 test, regardless of the severity of exposure to chlorides or other fluids. In order to ensure long service lives, concrete requiring low permeability should be specified to achieve lower chloride penetration resistance than what is currently proposed. Concrete with severe chloride exposures (Class C2), for example, should be required to have a chloride resistance below 1000 coulombs at 56 days, while concrete subject to P1 permeability restrictions should have chloride resistances below 2000 coulombs at 56 days.

Table 7. NRMCA performance alternatives for freeze-thaw durability [24].

Class	Description	ASTM C1202 Chloride Resistance (Coulomb)	Air Content
F0	Not applicable	N/A	N/A
F1	Moderate	2000 @ 28 days[1] 2500 @ 56 days	ASTM C666 durability factor ≥ 80% ASTM C672 mass loss ≤ 1.0 kg/m^2 ASTM C457 spacing factor ≤ 0.008 in and air content ≥ 3.0%
F2	Severe	2000 @ 28 days[1] 2500 @ 56 days	ASTM C666 durability factor ≥ 85% ASTM C672 mass loss ≤ 1.0 kg/m^2 ASTM C457 spacing factor ≤ 0.008 in and air content ≥ 3.0%
F3	Very severe	2000 @ 28 days[1] 2500 @ 56 days	ASTM C666 durability factor ≥ 90% ASTM C672 mass loss ≤ 1.0 kg/m^2 ASTM C457 spacing factor ≤ 0.008 in and air content ≥ 3.0%

[1]Accelerated cure.

Table 8. NRMCA performance alternatives for sulfate resistance [24].

Class	Description	ASTM C1202 Chloride Resistance (Coulomb)	Maximum ASTM C1012 Expansion
S0	Not applicable	N/A	N/A
S1	Moderate	2500 @ 28 days[1] 3000 @ 56 days	0.05% @ 6 months, PQ[2]
S2	Severe	2000 @ 28 days[1] 2500 @ 56 days	0.05% @ 6 months OR 0.10% @ 12 months, PQ
S3	Very severe	2000 @ 28 days[1] 2500 @ 56 days	0.10% @ 18 months, PQ

[1]Accelerated cure.
[2]PQ indicates that the test is to be performed as a pre-qualifier for the mix, due to its long lead time.

Table 9. NRMCA performance alternatives for low-permeability concretes [24].

Class	Description	ASTM C1202 Chloride Resistance (Coulomb)
P0	Not applicable	N/A
P1	Required	2500 @ 28 days[1] 3000 @ 56 days

[1]Accelerated cure.

Table 10. NRMCA performance alternatives for chloride resistance [24].

Class	Description	ASTM C1202 Chloride Resistance (Coulomb)
C0	Not applicable	N/A
C1	Moderate	1500 @ 28 days[1] 2000 @ 56 days
C2	Severe	1500 @ 28 days[1] 2000 @ 56 days

[1]Accelerated cure.

2.3.3 High Performance Concrete

Many states, including Alabama, Colorado, Georgia, Louisiana, Nebraska, New Hampshire, New Mexico, North Carolina, Ohio, South Dakota, Tennessee, Texas, Virginia, and Washington [24], have experience with High Performance Concrete (HPC), defined by ACI as "concrete which meets special performance and uniformity requirements that cannot always be achieved routinely by using only conventional materials and normal mixing, placing, and curing practices" [26]. By definition, these concretes must adhere to specific performance requirements prior to acceptance, and may require different techniques for mixing, placing, and curing – techniques that do not necessarily conform to current prescriptive specifications for concrete.

HPC mixtures may be specified to achieve high compressive strength, low permeability, resistance to cracking and scaling, good workability and finishability, or other criteria relating to the performance of the mixtures in the plastic or hardened states [24]. These criteria are quite similar to those specified by performance-based specifications, suggesting that HPC mixtures may provide a reasonable framework upon which performance-based concrete standards may be based. While HPC mix specifications are still primarily prescriptive in nature, requiring certain minimum percentages of supplementary cementitious materials (SCMs), for example, the durability requirements and the test acceptance limits proposed by existing HPC specifications can form a basis for future durability-based performance specifications.

To illustrate, the FHWA performance grade system for high performance concrete adopts an exposure classification approach similar to those found in the Australian and Canadian performance specifications. Performance grades for HPC mixes range from 1 to 3, with higher numbers used for more severe exposures. Functional requirements are considered for freeze-thaw durability, scaling resistance, abrasion resistance, and chloride penetration, as shown in Table 11 and Table 12 [24, 27]. Requirements to limit alkali-silica reactivity and to improve sulfate resistance have also been proposed [27]. Taken together, these definitions and requirements could be easily adapted into a performance specification, so that a bridge deck in coastal Georgia, for example, may be specified to achieve Grade 1 Freeze-Thaw Durability and Grade 3 Chloride Penetration Resistance. Additional performance characteristics that may be specified but are not related to exposure conditions are also provided in Table 13. These may similarly be adopted into functional requirements for performance specifications.

Table 11. FHWA Performance Grade classifications for HPC [24,27].

Exposure Condition	Standard Measurement, x	FHWA HPC Performance Grade			
		N/A	**1**	**2**	**3**
Freeze-Thaw Durability	Cycles per year	$x < 3$	$3 \leq x < 50$	$50 \leq x$	--
Scaling Resistance	Applied salt, tons/lane-mile-year	$x < 5.0$	$5.0 \leq x$	--	--
Abrasion Resistance	Average daily traffic, vehicles	no studs/ chains	$x \leq 50{,}000$	$50{,}000 < x < 100{,}000$	$100{,}000 \leq x$
Chloride Penetration	Applied salt, tons/lane-mile-year	$x < 1$	$1.0 \leq x < 3.0$	$3.0 \leq x < 6.0$	$6.0 \leq x$
Alkali-Silica Reactivity[3]	Expansion at 14d per ASTM C1260, percent	$x < 0.1$	$0.1 \leq x < 0.2$	$0.2 \leq x < 0.4$	$0.4 \leq x$
Sulfate Resistance[3]	Sulfates, ppm	$x = 0$	$0 \;\; x \leq 150$	$150 \;\; x \leq 1500$	$1500 < x$

[3]Proposed but currently not included.

Table 12. FHWA functional requirements for HPC durability [24,27].

Performance Characteristic	Standard Test Method	Standard Measurement, x	FHWA HPC Performance Grade		
			1	2	3
Freeze-Thaw Durability	AASHTO T161 ASTM C666 (Proc. A)	Relative dynamic modulus of elasticity after 300 cycles	60% ≤ x < 80%	80% ≤ x	--
Scaling Resistance	ASTM C672	Visual rating of surface after 50 cycles	x = 4, 5	x = 2, 3	x = 0, 1
Abrasion Resistance	ASTM C944	Avg. depth of wear, mm	2.0 > x ≥ 1.0	1.0 > x ≥ 0.5	0.5 > x
Chloride Penetration	AASHTO T277 ASTM C1202	Coulombs	3000 ≥ x > 2000	2000 ≥ x > 800	800 ≥ x
Alkali-Silica Reactivity[4]	ASTM C441	Expansion at 56d, percent	0.20 ≥ x > 0.15	0.15 ≥ x > 0.10	0.10 ≥ x
Sulfate Resistance[4]	ASTM C1012	Expansion, percent	x ≤ 0.10 at 6 mo.	x ≤ 0.10 at 12 mo.	x ≤ 0.10 at 18 mo.

[4]Proposed but currently not included.

Table 13. FHWA functional requirements for HPC mechanical properties [24,27].

Performance Characteristic	Standard Test Method	Standard Measurement, x	FHWA HPC Performance Grade			
			1	2	3	4
Strength	AASHTO T2 ASTM C39	Compressive strength, ksi	6 ≤ x < 8	8 ≤ x < 10	10 ≤ x < 14	x ≥ 14
Elasticity	ASTM C469	Elastic modulus, ksi	4000 ≤ x < 6000	6000 ≤ x < 7500	x ≥ 7500	--
Shrinkage	ASTM C157	Microstrain	800 > x ≥ 600	600 > x ≥ 400	400 > x	--
Creep	ASTM C512	Microstrain/ pressure, psi^{-1}	0.52 ≥ x > 0.41	0.41 ≥ x > 0.31	0.31 ≥ x > 0.21	0.21 ≥ x
Flowability[5]	AASHTO T119 ASTM C143	Slump, in	x > 7.5	--	--	--
		Slump flow, in	x < 20	20 ≤ x ≤ 24	24 < x	--

[5]Proposed but currently not included.

2.3.4 New Mexico

The New Mexico Department of Transportation (NMDOT) was one of the first state DOTs to adopt performance-based specifications for concrete structures. Local New Mexico aggregates are extremely reactive, and under certain conditions, have the potential to cause alkali-silica reaction (ASR) when used in concrete. A project initially intended to revise their prescriptive concrete specifications to address ASR mitigation in the late 1990s resulted in a complete overhaul of all of the concrete specifications in the state [28].

A review of the then in-place prescriptive specifications indicated that the prescriptive specifications tended to result in mixtures that were difficult to mix, place, and finish. Additionally, it was believed that signs of early-age cracking, shrinkage, and segregation that were apparent in several bridge deck and concrete structures could be attributed, at least in part, to the prescriptive requirements of the existing specifications. Most importantly, however, it was realized that the prescriptive specifications did not effectively address the long-term durability of the concrete produced throughout the state, and problems relating to ASR, freezing and thawing, and salt-related damages were common [28].

The revised standards were issued on January 1, 1999. The standards removed any reference to minimum cement content and maximum w/cm, the first of its kind in the United States to do so. The new performance-based specifications are summarized below [29]:

1. ASR: Aggregates must be tested for potential ASR reactivity using AASHTO T303, ASTM C1260, or ASTM C1293. If the aggregates are considered "potentially

reactive" or "reactive", the producer is to add SCMs or use ASR-mitigating agents such as lithium nitrate and repeat the test. If 14-day ASTM C1260 (AASHTO T303) mortar bar expansions are less than 0.10%, then the mixture is considered satisfactory.

2. Freeze-thaw: Risk zones are specified based on geography and minimum air contents are indicated for each zone. Freeze-thaw durability is assessed using a modified ASTM C666, Method A, procedure and minimum durability factors are assigned to each risk zone (85, 90, and 95 for low-, medium-, and high-risk zones, respectively). Additional characterization of the hardened air void system via ASTM C457 is also specified as a conservative requirement to ensure adequate freeze-thaw durability [28].
3. Permeability: 28-day ASTM C1202 (RCPT) chloride ion permeability threshold values are established for high- (2000 coulombs), medium- (2500 coulombs), and low-risk (3000 coulombs) zones.
4. Strength gain: The 7-day strength must be no greater than 75% of the 28-day strength, and the 56-day strength must be at least 108% of the 28-day strength. This ensures that the concrete continues to gain strength after the standard measurements have been completed [28].
5. Drying Shrinkage: 56-day drying shrinkage is limited to less than 0.05%, when tested in accordance with AASHTO T160.

The long lead times for many of these durability tests necessitated a program of pre-qualifications and intermediate approvals. If 7-day and 28-day test results for strength, air-void system, RCPT results, and ASR mitigation are deemed acceptable, a temporary approval is issued pending the results of the 56-day tests.

Simons [28] mentions that in the four years following the implementation of the performance-based specifications, there has been a dramatic reduction in concrete-related problems reported throughout the state. Additionally, he reports that contractors have also indicated that the mixtures have become much easier to use, place and finish and that their overall performance has become more uniform. As these structures continue to age, they will continue to provide useful information regarding the benefits (or potentially, the hidden consequences) of performance-based specifications.

2.3.5 Virginia

Another pioneer in the adoption of performance specifications is the Virginia Department of Transportation (VDOT). Over the last 20 years, VDOT has adopted special provisional "end-result specifications" (ERS) on several highway projects throughout the commonwealth. Like FHWA's performance-related specifications, these standards are concerned with the quality of both the materials and the construction practices and how they relate to the long-term durability of the finished product; however, unlike true performance-based requirements, the design mix proportions must still be approved prior to use.

The special provisions provide a minimum 28-day compressive strength and 28-day permeability for each class of concrete, although smoothness, cover, and thickness requirements may also be specified for highway projects. Extensive quality control measures are implemented to ensure that the concrete provided meets the standards of the specification, and pay adjustments are made based on the percent within limits (PWL) of

28-day compressive strength and the results of a modified ASTM C1202 RCP test. The pay factors include both bonuses and penalties, with 100% pay being awarded for 90% PWL and bonuses applied for concretes exceeding that level of quality [30]. One criticism noted by several producers and acknowledged by VDOT as a potential challenge for the future is that, although VDOT has yet to enforce the pay adjustment factors, they worry that bonuses will go to the contractors while the penalties will go to the producers [30-32].

In an interview with VDOT personnel, it was additionally mentioned that a shift to performance-based or end-result specifications does not mean that the existing prescriptive specifications are inferior or should be discarded. Virginia adopted a hybrid specification type specifically because there are instances when performance-based specifications are *not* a viable option on their own. For example, there are certain performance criteria that are difficult to define or assess in the field and so a performance-based specification may be difficult to develop or enforce. Additional reasons for adopting a hybrid-type specification include better management of legal issues, in that retaining prescriptive requirements and reviewing mixture proportioning and testing for performance-based designs provides the owner (VDOT) with more information about how the mixtures should perform prior to acceptance. Furthermore, prescriptive requirements must be retained in order to protect the business interests of smaller ready-mix plants that may lack the resources to create innovative mixtures or to perform the required performance testing necessary to compete with larger companies for projects under performance-based specifications [33].

2.3.6 Minnesota

The state of Minnesota has been working toward performance-based specifications since the early 1990s. Since 1992, the Minnesota Department of Transportation (MnDOT) has allowed contractors to develop mix designs for special contract provisions in selected projects [6]. Along with these special provisions, MnDOT also developed short courses intended to train and certify personnel who adopt the new approach. While current specifications are still predominantly prescriptive in nature, the MnDOT 2301 *Standard Specification for Concrete Pavements* [34] supports innovations in design by awarding incentives to concrete producers who optimize their mixtures on the basis of aggregate gradation and w/cm ratio; well-graded aggregates (defined by MnDOT as aggregates with 8-18 or 7-18 gradations) and low w/cm ratios receive payment bonuses, while poor gradations and high w/cm ratios incur penalties.

2.3.7 Port Authority of New York/New Jersey

The Port Authority of New York/New Jersey (PANYNJ) has been using performance-based concrete specifications for more than a decade. Each application has its own individual specification, with its own unique performance criteria. Criteria may include compressive and flexural strength, bond strength, permeability, and shrinkage, among others [35, 36]. As with other agencies, bonuses are paid for concrete meeting performance criteria, while a penalty is assessed to concrete that fails to meet the specified quality indices. PANYNJ personnel have noted an improvement in the quality of the concrete delivered as a result of these specifications [36].

2.3.8 Pennsylvania

A 2004 initiative by the Pennsylvania DOT (PennDOT), FHWA, and industry partners sought to develop optimal design parameters for HPC mixes. The mixes were subject to a combination of prescriptive and performance-based requirements that serve as a good model for future hybrid specifications. All mixes were to contain 564 to 611 lb./yd.^{3} of cementitious material with a maximum w/cm of 0.43, and to demonstrate the following performance characteristics [37]:

1. 28-day shrinkage per ASTM C157 must be less than 500 microstrain,
2. 56-day RCPT conductivity per AASHTO T277 must be less than 1500 coulombs,
3. At least 60% reduction in ASR expansion must be obtained when the design mixture is compared to the control mixture specified in ASTM C441,
4. Compressive strength at 56 days must be greater than 4000 psi, and
5. Plastic air content must be between 6.0 ± 1.5% and hardened air content between 4.5 and 8.0%, with a 0.008 in spacing factor per ASTM C457.

More than 150 mixtures were tested, each containing various combinations and amounts of SCMs. It was found that the HPC mixtures, all containing binary and ternary combinations of SCMs, typically resulted in a 40 to 75% reduction in chloride permeability, with a smaller average pore diameter, higher resistivity, and lower cracking potential than the standard bridge deck concrete prescribed in Pennsylvania for the past 30 years [37]. All of these features combine to dramatically increase the service lives of the in-place concrete, indicating one of the primary benefits of performance-based specifications.

2.3.9 Colorado

The Colorado DOT (CDOT) currently employs a hybrid type of specification, requiring certain minimum cementitious materials contents, maximum w/cm ranges, and aggregate contents, but also requiring that certain performance indicators be demonstrated. In particular, Class H and HT concretes, high-performance mixtures specified for use in concrete bridge decks, require [38]:

1. A w/cm ratio between 0.38 and 0.42,
2. A ternary mixture of cementitious materials, containing 450 to 500 lb./yd.^{3} of hydraulic cement, 90 to 125 lb./yd.^{3} of fly ash, and 20 to 30 lb./yd.^{3} of silica fume,
3. A minimum percentage of coarse aggregate in specific gradations (dependent upon the class of concrete specified),
4. 56-day RCPT permeability of less than 2000 coulombs, and
5. No cracking before 15 days in the AASHTO T334 cracking tendency test.

While the first three of these requirements are clearly prescriptive in nature, the final two relate to the long-term durability of the concrete. Additional performance-based requirements for ASR mitigation and sulfate resistance are also provided.

A recent study conducted by the CDOT Research and Innovation Branch examined the durability of the two Class H and HT concretes, with particular emphasis on early-age cracking (addressed in the present specification by item #5 above). The report recommended increasing the maximum allowable w/cm, increasing the allowable quantities of SCMs, permitting the use of additional SCMs such as ground-granulated blast furnace slag, and reducing the cementitious content in the absence of SCMs [39]. In essence, they concluded that changing the prescriptive requirements would lead to improved performance. But changing prescriptive requirements is only an indirect means of obtaining satisfactory performance. As such, efforts to develop fully performance-based alternatives to these largely prescriptive specifications are ongoing, and it is believed that such alternative mixtures will alleviate many of the performance-related issues CDOT has experienced [40].

2.4 Summary and Conclusions

Performance-based specifications provide an alternative means of designing concrete for durability. Rather than specifying materials, means, and methods, as current prescriptive specifications do, a performance specification instead indicates the functional requirements for concrete in the plastic and hardened states and grants the concrete producer the freedom to design a mixture that meets those requirements. The most commonly specified parameters include low permeability, per the ASTM C1202 Rapid Chloride Permeability Test or the AASHTO TP95 surface resistivity test; good sulfate resistance, per the ASTM C1012 sulfate expansion test; and adequate freeze-thaw durability and/or air void parameters, per ASTM C666 and/or ASTM C457, respectively. Often these requirements take the form of “exposure classes,” such as those specified by ACI 318 or CSA A23.1/A23.2. Given that these classification systems have been particularly successful in states and countries where climate varies considerably with geography, this same approach may prove particularly useful in Georgia, where environmental exposures vary tremendously between coastal regions in the southern part of the state and the mountainous regions in the north.

One of the primary benefits of a performance-based specification is that it allows the concrete producer to use innovative mixture designs and processes currently not permitted under prescriptive specifications. In many cases, innovations have resulted in more durable concretes with predicted service lives exceeding 75 years, lower economic costs to the owner, and decreased environmental impacts due to the increased use of recycled materials. However, the successful implementation and execution of performance specifications can only come about when the specifier properly identifies behaviors and characteristics necessary to ensure long-term durability and when the contractor and producer understand how to produce mixtures that meet those requirements. Additional instruction may be required to provide the designers and construction team with the knowledge required to develop the most effective specifications and mixes. Similarly, further research is required to develop fast and accurate testing procedures for many desired characteristics, including permeability and shrinkage. Nevertheless, it is evident from past experience that performance specifications have the potential to improve the durability and longevity of concrete mixtures produced worldwide, and performance-based alternatives should be incorporated into standard concrete specifications.

3. EFFECT OF BINDER COMPOSITION

3.1 Introduction

Based upon the outcomes of the state-of-the-art review, it was determined that permeability is the most often cited performance criterion in a performance-based specification. Permeability controls many aspects of concrete durability, from resistance to chloride and sulfate ion penetration to the rate of ingress of water, which has implications for freeze-thaw damage and alkali-silica reaction, among others. Therefore, it was desirable to better understand factors that affect the permeability of concrete, paying particular attention to variations in binder composition.

The most commonly specified test method used to assess the permeability of concrete is the Rapid Chloride Permeability Test (RCPT), specified in the AASHTO T277 [41] and ASTM C1202 [42] standard test methods. In this test, a 2 in. thick disk of concrete is placed in a test cell with a 0.3 N sodium hydroxide (NaOH) solution on one side and a 3 wt. % sodium chloride (NaCl) solution on the other. A 60 V voltage is applied to the test cell for 6 hours and the total charge passed between the two cells is recorded. The total charge passed is related to the electrical conductivity of the concrete, which, in turn, is related to the diffusivity coefficient of the concrete by the Nernst-Einstein equation. Because of this relationship, the total charge passed during RCPT can be used to qualitatively assess the permeability of the concrete, using the classifications shown in Table 14.

A newer test, described in a provisional standard AASHTO TP95 [43], instead uses the surface resistivity (SR) of the concrete as an indicator of its permeability. In this test, a four-probed resistivity meter called a Wenner array is applied to the surface of a 4 in. by 8 in. or 6 in. by 12 in. concrete cylinder. Current flows through the two outer probes and the voltage between the two inner probes is measured. Using Ohm's Law, the resistance of the concrete can then be computed as the ratio between the voltage and the current. Then, knowing the geometry of the concrete sample and the spacing between the probes, the surface resistivity of the concrete can be determined. As with RCPT, the surface resistivity can then be used to qualitatively assess the permeability of the concrete, using the classifications shown in Table 14.

Table 14. Chloride ion permeability limits for concrete tested at 28 days according to AASHTO T277 and AASHTO TP95 [41-43].

Classification	**RCP Limits (Coulombs Passed)**	**SR Limits (kOhm-cm)**	
		4"x8" cylinder	**6"x12" cylinder**
High	> 4000	< 12	< 9.5
Moderate	2000 – 4000	12 – 21	9.5 – 16.5
Low	1000 – 2000	21 – 37	16.5 – 29
Very Low	100 – 1000	37 – 254	29 – 199
Negligible	< 100	> 254	> 199

One advantage of surface resistivity testing is that it is relatively fast and simple to perform compared to RCPT and other standard tests for diffusion and permeability; measurements can be made within minutes using an off-the-shelf device available from several commercial producers. Furthermore, surface resistivity testing is non-destructive, meaning that resistivity measurements can be made on the same concrete test specimens at different ages, effectively allowing the permeability of the concrete to be monitored over time.

The objective of this phase of the project is to determine which binder compositions offer the greatest improvements in concrete permeability. Twelve different binder compositions were examined, considering variations in water-to-cementitious materials ratios (w/cm), the use of supplementary cementitious materials (SCMs), and alternative cement compositions. Permeability was assessed at 56 days using RCPT and periodically throughout the 56 day curing period using surface resistivity. It was expected that by monitoring surface resistivity over time, the effects of each binder component could be isolated, leading to greater insights into which combination of materials will lead to better performance.

3.2 Materials

The permeability characteristics of twelve initial mix designs were evaluated over a period of 56 days. The mixes were selected to represent a variety of mixtures that might be considered for a future structural application, such as a bridge deck subjected to moderate chloride exposures. Two "prescriptive" mixtures were selected to conform to GDOT Section 500 Specifications for Class AA concrete, while the remaining 10 "performance-based" mixtures were selected to better understand the effect of varying water-to-cementitious materials ratio (w/cm), supplementary cementitious materials (SCM) content, and cement composition on concrete permeability. A brief summary of each mixture is provided in Table 15, and the mix designs are shown in Table 16. The two Class AA prescriptive mixtures are shown in italics.

Table 15. Concrete mixture summary. The two italicized mixtures conform to the GDOT Section 500 Specifications for Class AA concrete.

Mixture	Cement Type	w/cm	Class F Fly Ash, % wt. cement	Metakaolin, % wt. cement
OPC 0.40	*ASTM C150 Type I/II*	*0.40*	-	-
OPC 0.50	ASTM C150 Type I/II	0.50	-	-
OPC 0.60	ASTM C150 Type I/II	0.60	-	-
15F	*ASTM C150 Type I/II*	*0.40*	*15*	-
25F	ASTM C150 Type I/II	0.40	25	-
25F+5MK	ASTM C150 Type I/II	0.40	25	5
10LS	ASTM C595 Type IL – 10% LS	0.40	-	-
10LS+15F	ASTM C595 Type IL – 10% LS	0.40	15	-
10LS+25F	ASTM C595 Type IL – 10% LS	0.40	25	-
12LS	ASTM C1157 Type GUL – 12% LS	0.40	-	-
12LS+15F	ASTM C1157 Type GUL – 12% LS	0.40	15	-
12LS+25F	ASTM C1157 Type GUL – 12% LS	0.40	25	-

Table 16. Concrete mix designs. The two italicized mixtures conform to the GDOT Section 500 Specifications for Class AA concrete.

Mix	Water (lb/yd^3)	Cement (lb/yd^3)	Fine Aggregate (lb/yd^3)	Coarse Aggregate (lb/yd^3)	Class F Fly Ash (lb/yd^3)	Metakaolin (lb/yd^3)
OPC 0.40	*354*	*850*	*1000*	*1738*	*0*	*0*
OPC 0.50	401	803	1000	1738	0	0
OPC 0.60	451	752	1000	1738	0	0
15F	*354*	*723*	*1000*	*1738*	*127*	*0*
25F	354	637	1000	1738	212	0
25F+5MK	354	595	1000	1738	212	43
10LS/12LS	354	850	1000	1738	0	0
10LS/12LS+15F	354	723	1000	1738	127	0
10LS/12LS+25F	354	637	1000	1738	212	0

The first six mixtures were designed using ASTM C150 Type I/II ordinary portland cement (Argos). Three of the mixtures were selected to examine the effect of varying the w/cm (OPC 0.40, OPC 0.50, and OPC 0.60), while the remaining mixtures were selected to examine the effects of SCM additions at a constant w/cm of 0.40. Class F fly ash (Boral) was used in two binary mixes at cement replacement rates of 15% and 25% by mass, and a ternary mix containing fly ash and metakaolin (Thiele) at cement replacement rates of 25% and 5%, respectively (15F, 25F, and 25F+5MK, respectively).

The remaining six concrete mixtures were made at a w/cm of 0.40, using locally-produced (Lehigh; Leeds, AL) portland limestone cements (PLC) containing either 10% or 12% interground fine limestone powder (LS), by weight. Because it has been reported in the literature that there exists a synergy between PLCs and fly ash that improves the overall permeability and strength characteristics of the concrete [44, 45], mixtures containing limestone powders and fly ash were also examined. Class F fly ash was used as a partial replacement for the cement at replacement levels of 0%, 15%, and 25% by mass.

The chemical compositions of the raw cements and SCMs are provided in Table 17 and Table 18, respectively. The particle size distributions are shown in Figure 1. The data indicate that the PLC with 10% limestone has a higher C_3S content than the Type I/II OPC but a comparable fineness, while the PLC with 12% limestone has a comparable chemical composition to the Type I/II cement but a finer particle size distribution. The metakaolin is finer than all three cements, while the fly ash is coarser; both SCMs are primarily composed of silicates and aluminates.

Table 17. Chemical composition (QXRD) for cement samples.

Phase	Type I/II OPC (%)	PLC – 10% LS (%)	PLC – 12% LS (%)
C_3S	53.36	61.93	56.74
C_2S	22.81	7.29	11.80
C_3A	2.98	4.20	2.87
C_4AF	11.12	10.65	11.35
Calcite	2.72	7.10	9.04
Dolomite	-	-	4.37

Table 18. Oxide analysis for fly ash and metakaolin samples.

Component	Class F Fly Ash (%)	Metakaolin (%)
SiO_2	55.95	51.28
Al_2O_3	29.39	44.27
Fe_2O_3	4.91	0.40
CaO	1.05	0.08
MgO	0.86	0.17
SO_3	0.29	0.13
LOI	2.69	0.96
Na_2O	0.29	0.41
K_2O	2.16	0.11
TiO_2	1.72	1.85
P_2O_5	0.48	0.29
MnO	0.02	0.01
SrO	0.13	0.01

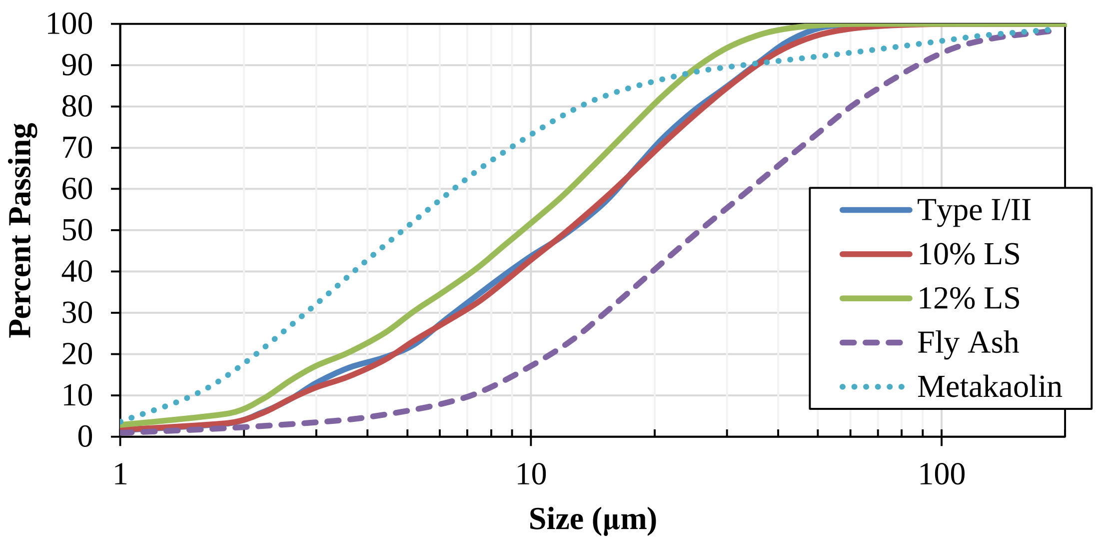


Figure 1. Particle size distribution for raw materials.

A crushed granite coarse aggregate (#67 stone with specific gravity = 2.65 and unit weight = 98 pcf) and natural sand fine aggregate (specific gravity = 2.63 and fineness modulus = 2.4) were also used. The aggregate proportions were identical for all twelve mixtures so that only the relative effects of varying binder composition would be observed. It should be noted that although the volume of concrete produced for this phase of the project was not large enough to conduct a standard slump test, it was observed during mixing that the OPC 0.50 and OPC 0.60 mixtures were highly fluid (suggesting a slump significantly higher than the prescribed 2-4 in.) due to the decision to use constant aggregate proportions for all 12 mixtures. Thus, the results for the OPC 0.50 and OPC 0.60 mixtures may not necessarily be indicative of the performance of true concrete mixtures made at w/cm of 0.50 and 0.60; however, they can still provide useful information about the effect of varying binder composition on the performance of concrete mixtures.

3.3 Methods

Three 4 in. by 8 in. cylinders of concrete were cast for each mixture. After 24 hours, the cylinders were stripped from their molds, and an initial surface resistivity measurement was made on each concrete sample using a four-probed Wenner array, in accordance with AASTHO TP95-11 (Figure 2). Measurements were made along lines drawn lengthwise at quarter-points around the circumference of the cylinder, with eight measurements being made for each cylinder (two measurements per line).

The samples were then stored in a saturated calcium hydroxide (limewater) solution at room temperature for 56 days, during which time the surface resistivity of each cylinder was periodically measured. The surface resistivity test is non-destructive, so measurements were able to be conducted on the same set of concrete specimens over all 56 days. Measurements were made once daily for the first 7 days, every other day for the next 7 days, and twice each week for the remainder of the 56 day period. At 28 days and 56 days of age, an additional bulk resistivity measurement was made on each cylinder, by placing the cylinder between two parallel plate electrodes attached to the surface resistivity meter [46].

After completing the final set of resistivity measurements on day 56, a 2 in. disk was cut from the center of two of the three cylinders cast for each mixture. The disks were conditioned overnight under vacuum pressure (Figure 3) and the Rapid Chloride Permeability (RCP) test was performed the following day in accordance with AASHTO T277/ASTM C1202 (Figure 4).

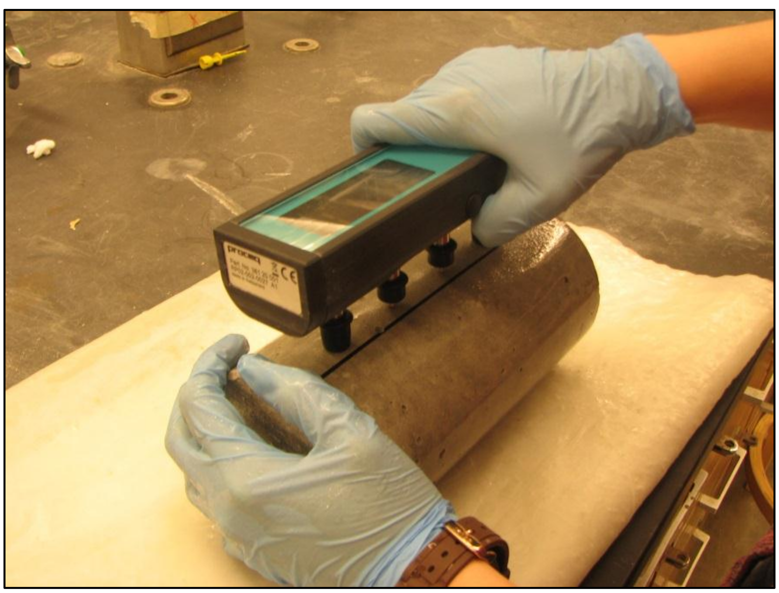


Figure 2. Surface resistivity test performed using four-probed Wenner array (AASHTO TP95-11).

Figure 3. Conditioning of concrete disks for RCP test (AASHTO T277).

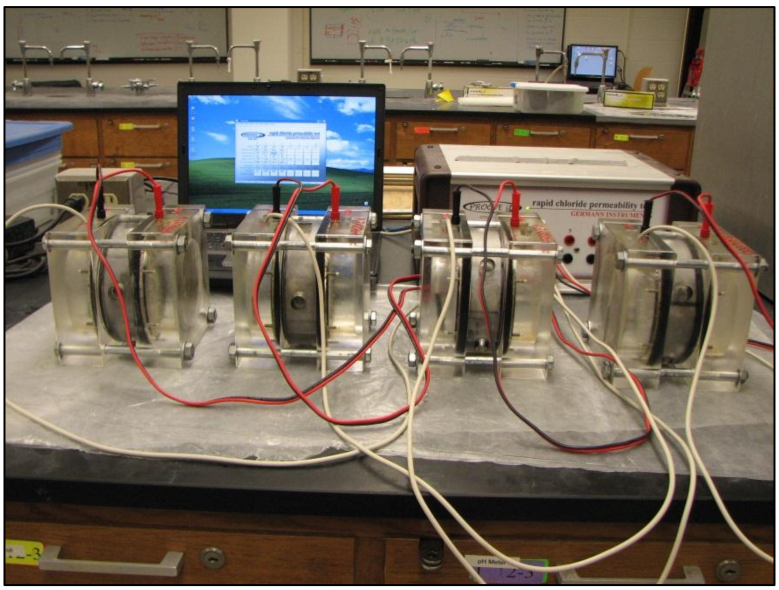


Figure 4. RCP test conducted in accordance with AASHTO T277.

3.4 Results and Discussion

3.4.1 Electrical Resistivity

Plots of average surface resistivity versus time are shown in Figure 5 and Figure 6 for the ordinary portland cement and limestone cement concrete mixtures, respectively. To aid in the interpretation of the results, horizontal dashed lines are shown in both figures to indicate each permeability classification range, according to the AASHTO TP95 standard. The AASHTO standard recommends that the resistivity values of limewater-cured concrete be multiplied by a factor of 1.1 to obtain an "equivalent" fog-cured concrete resistivity, but this was not done for the results presented in this report, since all twelve mixtures were cured in limewater and direct comparison between mixtures is possible without scaling. Instead, the 28 day resistivity limits provided in the AAHSTO standard were divided by a factor of 1.1 to obtain the values shown in Table 19 and indicated by the dashed lines in the figures.

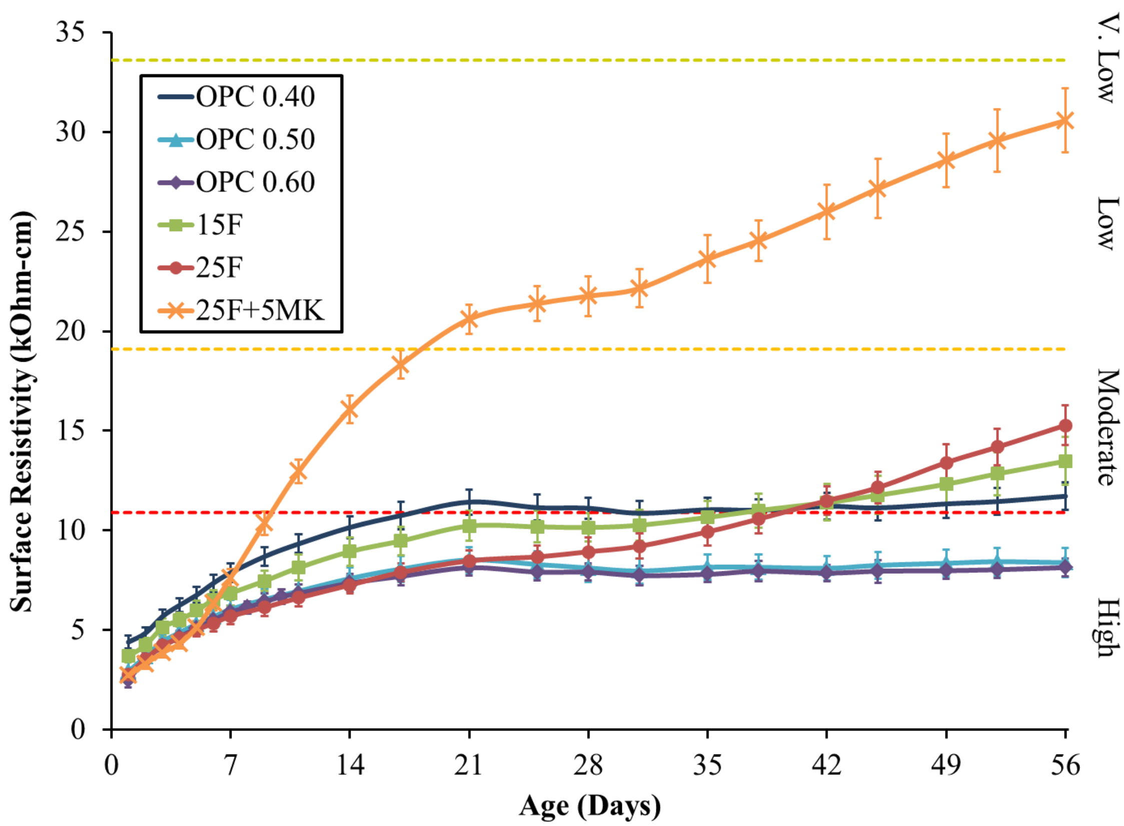


Figure 5. Surface resistivity development over time for OPC mixtures. Horizontal dashed lines indicate permeability classification limits specified in AASHTO TP95 for 4"x8" concrete cylinders at 28 days. Limits have been divided by a factor of 1.1 to account for limewater curing.

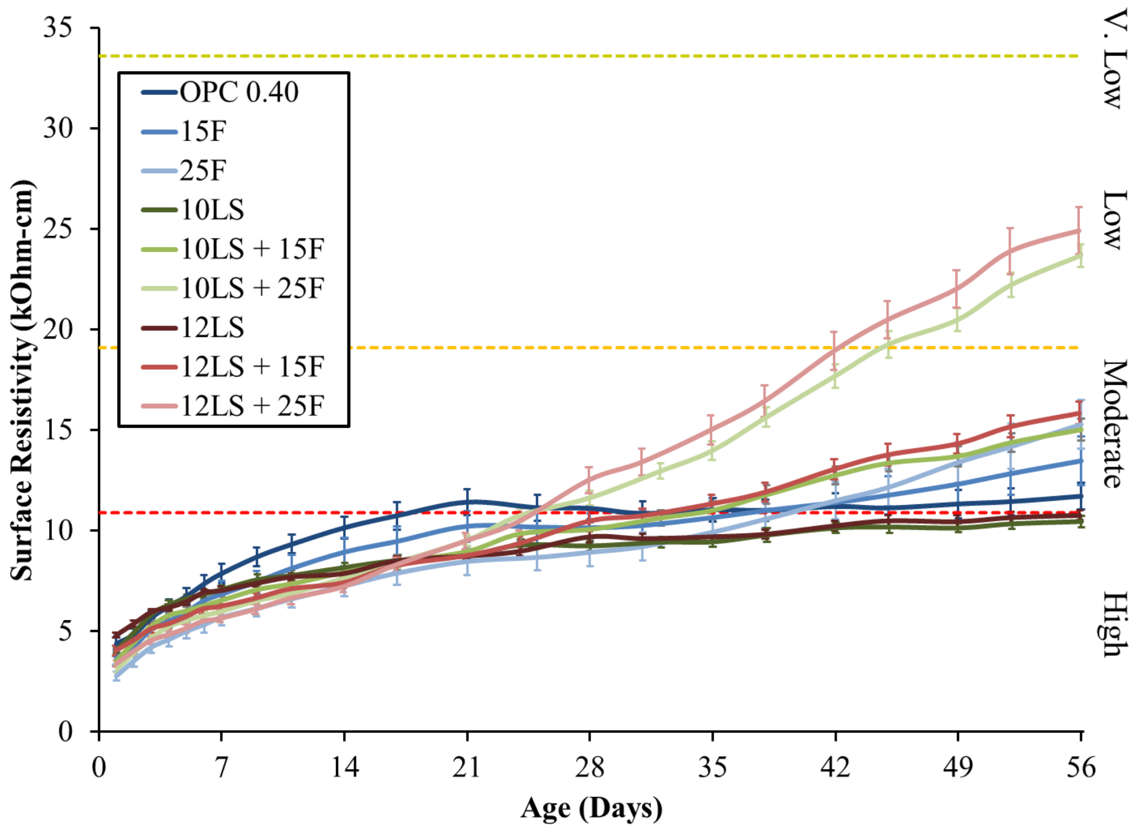


Figure 6. Surface resistivity development for limestone cement mixtures. OPC control mixtures are shown in blue for comparison. Horizontal dashed lines indicate permeability classification limits specified in AASHTO TP95 for 4"x8" concrete cylinders at 28 days. Limits have been divided by a factor of 1.1 to account for limewater curing.

Table 19. Adjusted AASHTO TP95 permeability limits for limewater cure.

Permeability Classification	Surface Resistivity Limits (kOhm-cm)	
	Moist cure	**Limewater cure**
High	< 12	< 10.9
Moderate	12 – 21	10.9 – 19.1
Low	21 – 37	19.1 – 33.6
Very Low	37 – 254	33.6 – 230.9
Negligible	> 254	> 230.9

All six OPC mixtures (Figure 5) show initial increases in surface resistivity as the hydration of the cement leads to a decrease in permeability and a subsequent increase in electrical resistivity. The relative heights of the six curves is initially directly related to the amount of cement in the mixes: the 25F + 5MK mixture contains the least cement and has the lowest initial resistivity, while the OPC 0.40 mixture has the highest cement content and the highest initial resistivity.

As each SCM begins to react, additional increases in surface resistivity can be observed. After 4 days, for example, the high reactivity of the metakaolin is made apparent by a steep increase in resistivity values of the 25F + 5MK mixture that persists for the next two weeks. By 28 days, the slower rate of reaction of the fly ash begins to become apparent, as the three SCM mixtures show slightly higher increases in resistivity over the three plain concrete mixtures. While the three OPC mixtures essentially level off by 21 days of curing (due to a lack of secondary SCM reactions), the three SCM mixtures continue to show steady increases in surface resistivity through 56 days.

By monitoring surface resistivity of concrete over time, it can also be seen that at 28 days, the fly ash does not react sufficiently to show any benefit to using fly ash over the control mixture; the surface resistivities of the 25F and 15F mixtures are only 80% and 90%, respectively, of the surface resistivity of the control OPC 0.40 mixture. By 56 days, however, the later reaction of the fly ash becomes apparent, as the surface resistivities of the 25F and 15F mixtures are 30% and 15% higher, respectively, than the resistivity of the OPC 0.40 control. This observation highlights the importance of testing SCM-containing mixes at later ages, in order to allow the SCMs to react before accepting or rejecting performance-based mix designs.

Similar trends can be noted for the limestone cement mixtures shown in Figure 6. The two PLC mixtures (10LS and 12LS) behave like the plain OPC mixture, as expected. Again, the largest gains in surface resistivity occur in the first 28 days, followed by a leveling off to approximately constant levels by 56 days. One notable difference between the two PLC mixtures and their OPC 0.40 counterpart is the downward shift in values as the limestone content increases. This downward shift occurs as a result of the “dilution effect.” When a portion of portland cement is replaced with an inert filler material such as ground limestone, there is less cement available to react during hydration; with a smaller volume of hydration products, the concrete is consequently more porous and more permeable, leading to decreases in surface resistivity values. Essentially, the limestone “filler” serves to decrease the effective cement content of the mix, resulting in a similar downward shift as was observed for the OPC 0.50 and OPC 0.60 concrete mixtures.

When the Class F fly ash is added to the limestone cement concrete mixtures, the concretes do not exhibit the same leveling-off behavior seen for the OPC-fly ash mixtures. Instead, the surface resistivity of the four fly ash-containing mixtures continues to increase steadily over the entire 56 day observation period. This results in significantly higher resistivity (lower permeability) by 56 days, especially for the mixtures containing 25% fly ash, where surface resistivity increases of more than 50% are observed when compared to the equivalent OPC concrete. This observation supports the synergy between fly ash and limestone cements previously reported by De Weerdt, et al. [44, 45], and suggests that these alternative concrete mixtures, not allowed under current Section 500 guidelines, may be effectively used for applications requiring low permeability. Assessment of the mechanical properties of these mixtures are considered in Chapter 4.

The 28- and 56-day bulk resistivity measurements are plotted against the corresponding surface resistivity measurements for all 12 concrete mixtures in Figure 7. At both ages, the bulk resistivity is approximately double the surface resistivity (R^2 =

0.985), minus a constant likely related to the resistivity of the sponges used to ensure adequate contact between the electrodes and the concrete surface during the bulk resistivity test. This is consistent with the results found by Hooton and Shahroodi [47]; however, their research notes that the relationship between the two measurements depends upon the devices used to make the measurements. Because more research is needed in this area, for the remainder of the study, only surface resistivity will be considered.

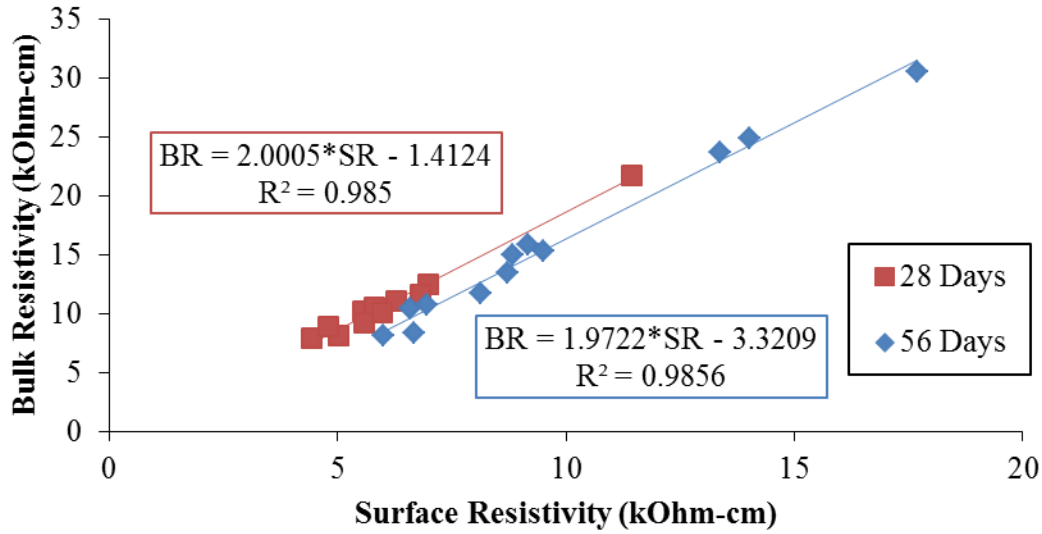


Figure 7. Relationship between bulk resistivity and surface resistivity.

<u>3.4.2 RCPT</u>

The relationship between surface resistivity and RCPT results at 56 days is shown in Figure 8, and corresponding values are provided in Table 20. A very strong correlation (R^2 = 0.979) between surface resistivity and RCPT is observed, as has been previously noted in the literature [48, 49]. The strong correlation suggests that resistivity testing can be considered as an alternative to RCPT for evaluating the permeability of concrete mixtures. One caveat, however, is that despite the strong correlation, the overall permeability classifications for the two tests do not always agree; as shown in Table 20, in certain cases, surface resistivity testing results in a lower permeability classification than RCPT. Furthermore, the line of best fit obtained for the twelve mixtures studied here is markedly different from the one found by Chini, et al. [49], on which the AASHTO provisional standard is based. Chini, et al., found that the relationship between surface resistivity (SR) and the charge passed during RCPT at 28 days is approximately

$$SR = 10442(RCPT)^{-0.819}$$

In this study, the relationship between surface resistivity and charged passed during RCPT at 56 days was found to be

$$SR = 8754.5(RCPT)^{-0.781}$$

In addition to age of testing being a primary source of discrepancy between these two studies, other possible sources of discrepancy may include differences in aggregate sources (Georgia granitic gneiss vs. Florida pleistocene limestone), curing conditions (limewater cure vs. fog cure), and cement composition (Type IL vs. Type I/II). Future work should be conducted to address these discrepancies before surface resistivity testing is accepted as an alternative to RCPT; however the results, in general, are consistent with one another to the extent that surface resistivity testing can be used to gain a better understanding of the permeability of concrete mixtures.

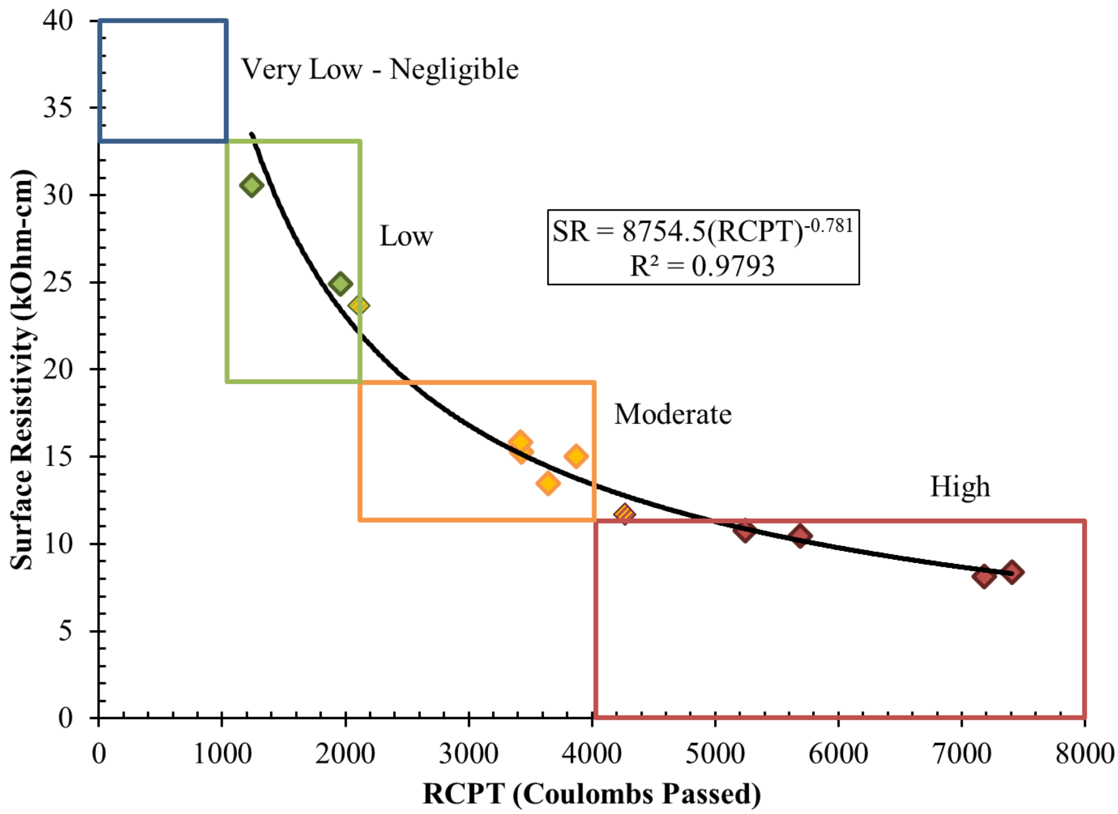


Figure 8. Relationship between surface resistivity and RCPT results at 56 days.

Table 20. 56 day results for RCPT and surface resistivity (SR).

Mix	**56 Day RCPT (Coulombs Passed)**	**56 Day SR (kOhm-cm)**	**Permeability Classification (RCPT/SR)**
OPC 0.40	4262	11.7	High/Moderate
OPC 0.50	6094	8.4	High/High
OPC 0.60	5792	8.1	High/High
15F	3645	13.5	Moderate/Moderate
25F	3428	15.3	Moderate/Moderate
25F + 5MK	1240	30.6	Low/Low
10LS	5689	10.5	High/High
10LS + 15F	3873	15.0	Moderate/Moderate
10LS + 25F	2106	23.7	Moderate/Low
12LS	5244	10.8	High/High
12LS + 15F	3420	15.8	Moderate/Moderate
12LS + 25F	1960	24.9	Low/Low

3.4.3 Chloride Ion Diffusivity and Service Life Modeling

From the results of the rapid chloride permeability test provided in the previous section, it is also possible to predict the chloride ion diffusivity of the individual concrete mixtures using the methods of Barde, et al. [50]. In their report, Barde, et al., employ the Nernst-Einstein equation to relate the diffusivity D of a charged species i to the electrical conductivity of that species through the concrete, σ_i. A simplified version of the Nernst-Einstein equation is shown below,

$$D_i = K_i \times \sigma_i$$

where the constant K_i is dependent upon the temperature of the sample, as well as the charge and concentration of the species considered. For the diffusivity of chloride ions through concrete at room temperature (70°F), the proportionality constant K_i takes on a value of 2.75 E-4 lb·in^4/C^2 (5.10 E-10 J·m^3/C^2).

Barde, et al., then related the electrical conductivity of the concrete to the total charge passed during RCPT, Q_t (in Coulombs), using the definition of electrical conductivity. A correction factor of 0.75 was applied based on experimental observations to relate the total charge passed to the initial current flowing through the concrete:

$$\sigma_i = 0.75 \cdot \frac{L \cdot Q_t}{V \cdot A \cdot t}$$

In this equation, L is the length of the sample (2 inches or 50 mm), V is the voltage applied to the specimen (60 V), A is the cross-sectional area of the specimen (12.19 in^2 or 7865 mm^2), and t is the duration of the test (6 hrs).

The two equations can then be combined to estimate the chloride ion diffusivity of the concrete. For the twelve mixtures considered in this study, the chloride ion diffusion coefficients were calculated from the 56-day RCPT values, as shown in Table 21.

Table 21. Chloride ion diffusion coefficients predicted at 56 days.

Mix ID	56 Day RCPT (Coulombs Passed)	Predicted Diffusion Coefficient (m^2/s)	Predicted Diffusion Coefficient (in^2/s)
OPC 0.40	4262	7.99E-12	1.24E-08
OPC 0.50	7408	1.39E-11	2.15E-08
OPC 0.60	7182	1.35E-11	2.09E-08
15F	3645	6.83E-12	1.06E-08
25F	3428	6.43E-12	9.96E-09
25F+5MK	1240	2.32E-12	3.60E-09
10LS	5689	1.07E-11	1.65E-08
10LS+15F	3873	7.26E-12	1.13E-08
10LS+25F	2106	3.95E-12	6.12E-09
12LS	5244	9.83E-12	1.52E-08
12LS+15F	3420	6.41E-12	9.94E-09
12LS+25F	1960	3.67E-12	5.70E-09

The chloride ion diffusion coefficients are useful in predicting the service life of concrete exposed to chlorides. The predicted 56-day diffusion coefficients in Table 21 were used as inputs into the Life 365 [51] software model to predict the service life of a hypothetical, *uncracked* 8-inch thick concrete slab with a 2-inch cover undergoing one-dimensional chloride ion diffusion. Corrosion of the reinforcing steel initiates when the concentration of chloride ions at the level of the reinforcement reaches a critical concentration threshold; for this model, a threshold of 0.05 wt. % was assumed. The corrosion is allowed to propagate for 6 years before the structure reaches its assumed service life.

Two exposure conditions were modeled for the concrete mixtures considered in this study. The first exposure condition assumes that the concrete belongs to a bridge deck located in urban Atlanta, with low exposure to chloride ions. The second exposure condition assumes that the concrete belongs to a bridge deck located in coastal Savannah, in a marine spray zone subject to extreme levels of chloride exposures. The predicted service lives of the twelve concrete mixtures for both exposure conditions are shown in Figure 9. In general, it was observed that the mixtures containing SCMs had significantly longer service lives than the mixtures containing only portland or limestone cements, which is consistent with the lower permeabilities that were observed for those mixtures. For example, the three mixtures with SR values indicative of “low” permeability (25F+5MK, 10LS+25F, and 12LS+25F) all contained 25% fly ash and had predicted service lives in excess of 120 years under normal exposure and 30 years under extreme exposures, while the four mixtures with SR values indicating “high” permeability (OPC 0.50, OPC 0.60, 10LS, and 12LS) all had service lives of approximately 50 and 10 years under normal and extreme exposures, respectively, and no SCM additions.

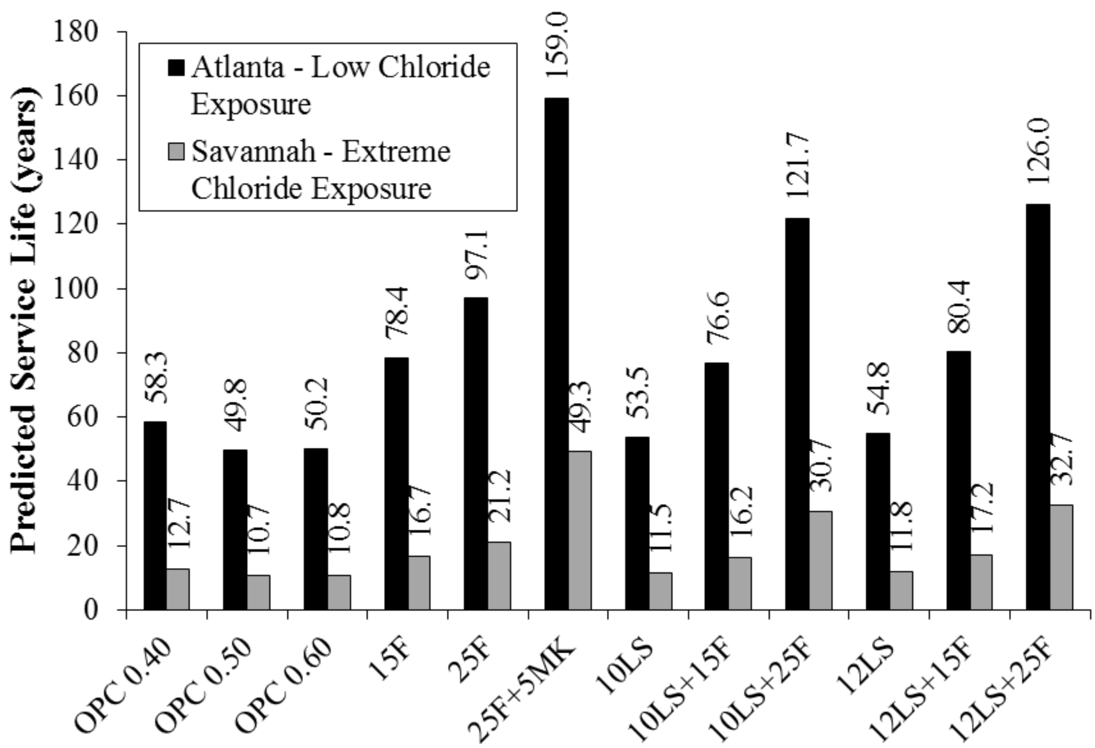


Figure 9. Service life predictions for low and extreme chloride exposures.

Two modifications were made to the Life 365 model for the concrete mixtures studied in this section, but neither is expected to have a significant effect on the service lives predicted. First, the Life 365 model is designed to take the 28-day diffusion coefficients as input, rather than the 56-day coefficients computed here. Life 365 decreases the diffusion coefficients as a function of time as the cement and SCMs in the concrete continue to hydrate and lead to denser porosity, so by assuming that the 28-day diffusion coefficient takes on the values computed at 56 days, the service life of the concrete is slightly overestimated. Second, the Life 365 software is designed to consider only concrete blends containing ordinary portland cement, Class F fly ash, slag, and silica fume and cannot accurately account for mixtures containing limestone cements or metakaolin. Therefore, for the six mixtures containing limestone cements, service lives were computed assuming ordinary portland cement, since the SR curves in Figure 6 suggest that the diffusion coefficients of the limestone cements and portland cements change at approximately equal rates. For the mixture containing both fly ash and metakaolin, the metakaolin was substituted by silica fume, since both additions are fast-reacting and not expected to have significant effects on diffusivity beyond the 28 day starting period. Thus, these two assumptions are not expected to have significant effects on the service lives predicted for the twelve mixtures and the assessments of concrete durability discussed above are still valid.

3.4.4 Economic Assessment

The cost of each concrete mixture was calculated on a per-ton basis, assuming materials costs of $110/ton for ordinary portland cement, $50/ton for Class F fly ash, $30/ton for both fine and coarse aggregates, and $500/ton for metakaolin. Because producers of portland limestone cements aim to produce limestone cements with the same performance as ordinary portland cements, it is expected that limestone cements will also have the same pricing as the ordinary portland cements, and consequently a cost of $110/ton was also assumed.

The cost per cubic yard of concrete produced for each mixture is shown in Table 22. Also shown in the table are the costs normalized by the predicted service life of the concrete under extreme chloride exposures (Savannah exposure). The results of the economic analysis indicate that although the ternary mixture containing 25% fly ash and 5% metakaolin was initially 2% more expensive than the base OPC 0.40 mixture, the dramatic improvements in permeability and service life that were achieved when using the two SCMs in combination actually made the mixture the most economical option when considering its 50-year service life under extreme chloride exposures. Furthermore, the three least expensive mixtures – all containing 25% fly ash combined with either an ordinary portland cement or a portland limestone cement – also had three of the longest expected service lives, each in excess of 20 years under extreme chloride exposures and 100 years under low chloride exposure conditions. It is therefore observed that improvements in both economic cost and durability are possible when designing concrete according to performance-based criteria.

Table 22. Economic analysis of twelve concrete mixtures.

Mix ID	Cost per yd^3 Concrete	Improvement in Cost per yd^3 Versus OPC 0.40 Control	Cost per Year Service Life: Extreme Chloride Exposures	Improvement in Cost per Year Versus OPC 0.40 Control
OPC 0.40	$87.82	--	$6.91	--
OPC 0.50	$85.24	**3%**	$7.97	**-15%**
OPC 0.60	$82.43	**6%**	$7.63	**-10%**
15F	$84.01	**4%**	$5.03	**27%**
25F	$81.41	**7%**	$3.84	**44%**
25F+5MK	$89.85	**-2%**	$1.82	**74%**
10LS	$87.82	0%	$7.64	**-11%**
10LS+15F	$84.01	**4%**	$5.19	**25%**
10LS+25F	$81.41	**7%**	$2.65	**62%**
12LS	$87.82	0%	$7.44	**-8%**
12LS+15F	$84.01	**4%**	$4.88	**29%**
12LS+25F	$81.41	**7%**	$2.49	**64%**

3.5 Conclusions

The permeability testing performed as part of this research effort suggests that surface resistivity can be used to quickly compare the performance of several concrete mixtures, which is especially useful for developing performance-based mix designs. The influence of SCM content, cement type, and w/cm were clearly observed in the surface resistivity development curves as differences in slope and height, where steeper slopes indicate faster rates of reaction and higher values indicate greater reductions in permeability.

The results additionally show that surface resistivity testing per AASHTO TP95 is a viable alternative to the currently specified AASHTO T277/ASTM C1202 Rapid Chloride Permeability Test, but future work should be conducted to address discrepancies between the overall permeability classifications provided by the two tests. As with RCPT, surface resistivity testing should be performend at 56 days of age, at the earliest, for concrete mixes containing SCMs; however, it is most effective to test at several ages so that trends in permeability may be observed. Both tests should be considered for use in a performance-based specification if permeability is selected as a primary performance criterion.

Finally, it has been shown that more durable concretes achieving lower permeabilities (according to surface resistivity or RCPT assessment) and longer service lives can be achieved by using alternative cements and SCM addition rates currently not allowed under GDOT Section 500 guidelines. Since permeability is of great concern when designing durable concrete with long service lives, mixes specially-tailored for reduced permeability can provide high performance at a relatively low cost, when designed under a performance-based specification.

4. EFFECT OF AGGREGATE TYPE

4.1 Introduction

Many of the performance-based concrete specifications discussed in Chapter 2 consider regional variations in climate and geography. In Georgia, regional variations in geology may also have significant impacts on the design and measurements of performance-based concrete mixtures. In this chapter, aggregates sourced throughout Georgia were incorporated into representative concrete mixtures that might be produced under future performance-based specifications. The mixtures were tested for permeability strength, and dimensional stability. Dimensional stability was evaluated using a modified shrinkage test based on the AASHTO T160 [52] and ASTM C157 [53] standards, and compared to requirements recently adopted by the Alabama Department of Transportation [54].

The objective of this part of the project was to assess how regional variations in aggregate sources affect the performance indicators used to assess concrete mixtures, with particular focus on permeability, mechanical properties, and dimensional stability. A prescriptive mixture designed to meet GDOT Section 500 requirements for Class AA concrete is compared to a performance-based mixture consisting of limestone cement and fly ash, based on the findings of Chapter 3. Based on the experiment design, it was expected that each aggregate type would produce concrete mixtures with comparable permeabilities, but that each might result in different strengths and shrinkage.

Additionally, it was anticipated that the performance-based mixture, which was tailored to achieve low permeability at 56 days, would also have higher strength when compared to the prescriptive mixtures, but potentially higher shrinkage as well, due to the influence of the fine limestone and fly ash additions.

4.2 Materials

Four aggregate pairings (Table 23) were selected to represent the regional variations in geology across Georgia. Each pair was recommended by the Georgia state geologist as a representative aggregate combination for concrete produced in different regions of the state: aggregate pairs A and B are typically used in concrete produced above the fall line, while pairs C and D are typically used in concrete produced below the fall line. It should be noted that the piedmont region fine aggregate used in pair B contains organic matter, and has been reported to have higher ASTM C1260 alkali activity than most other northern sands. All coarse aggregates have a nominal #57 gradation with a maximum size aggregate of 1 inch.

Table 23. Aggregate pairings.

Pairing	Coarse Aggregate		Fine Aggregate	
	Type	Source	Type	Source
A	Dolomitic Limestone (unit weight = 95 pcf)	Adairsville, GA	Coastal Plain Natural Sand (fineness modulus [FM] = 2.7)	Lambert Shorter, AL
B	Piedmont Crushed Granite (unit weight = 100 pcf)	Hanson Gainesville, GA	Piedmont Natural Sand (FM = 2.5)	Redland Greene Co., GA
C	Banded Crushed Granite (unit weight = 102 pcf)	Martin Marietta Ruby, GA	Cretaceous Mid-coastal Natural Sand (FM = 2.4)	Atlanta Sand Roberta, GA
D	Crushed Granite (unit weight = 102 pcf)	Vulcan Macon, GA	Cretaceous Mid-coastal Natural Sand (FM = 2.4)	Atlanta Sand Roberta, GA

Prescriptive mixtures were made using a Type I/II cement (National) with the minimum cement factor (635 lb/yd^3) and maximum w/c (0.445) allowed for Class AA concrete under current GDOT Section 500 specifications. The oxide analysis and particle size distribution for the cement are shown in Table 24 and Figure 10, respectively. Aggregates were proportioned using the PCA mix design method to achieve a 2” – 4” slump for each pairing. An actual slump of 1.5” was obtained for all four prescriptive mixes.

Table 24. Oxide analysis and Bogue composition for Type I/II cement.

Oxide Composition	Type I/II Cement (%)
SiO_2	20.51
Al_2O_3	4.65
Fe_2O_3	3.35
CaO	62.6
MgO	2.81
SO_3	2.99
LOI	1.85
Na_2O	0.07
K_2O	0.75
TiO_2	0.28
P_2O_5	0.04
MnO	0.05
SrO	< 0.01
C_3S	54
C_2S	18
C_3A	6.7
C_4AF	10
Gypsum	6.4

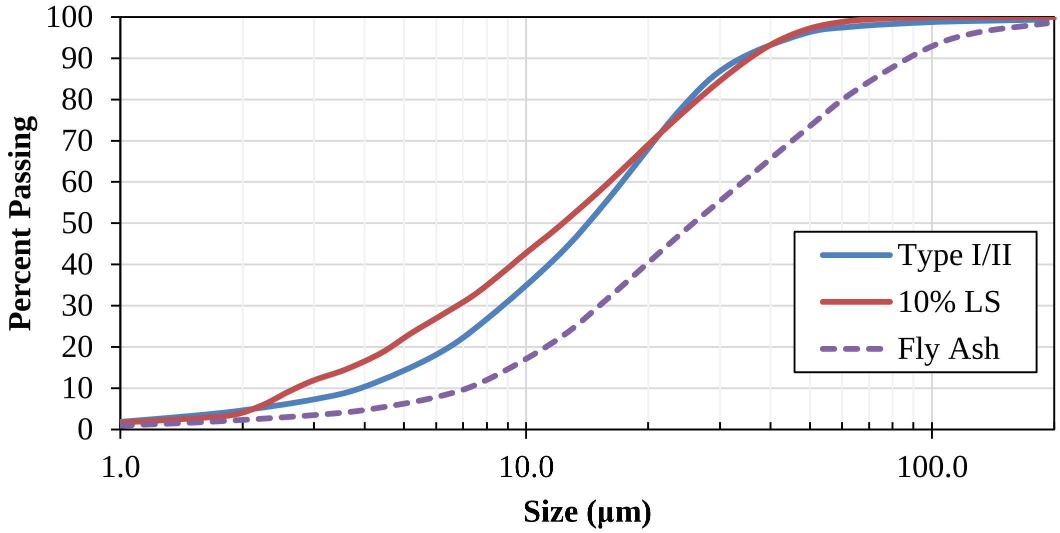


Figure 10. Particle size distribution for raw materials used in performance and prescriptive mixtures.

Performance mixtures were made using a Type IL portland limestone cement (Lehigh; Leeds, AL) containing 10% interground limestone. 15% of the cement was replaced by volume with Class F fly ash (Boral), for an effective mass replacement of 20.9%. The same cements and supplementary cementitious materials described in Chapter 3 were used for these mixtures. Like the prescriptive mixtures, the minimum cement factor (635 lb/yd^3) and maximum w/cm (0.445) were selected, and aggregates were proportioned to achieve a 2" – 4" slump. An actual slump of 1.5" was obtained for all four performance mixes, consistent with their prescriptive counterparts.

The eight concrete mix designs are given in Table 25. Mixes labeled "PRES" are the prescriptive mixes designed with the Type I portland cement, while mixes labeled "PERF" are the performance mixes designed with the Type IL limestone cement and Class F fly ash.

Table 25. Prescriptive and performance mix designs for each aggregate type.

Mix ID	Cement (lb/yd^3)	Water (lb/yd^3)	Coarse Aggregate (lb/yd^3)	Fine Aggregate (lb/yd^3)	Class F Fly Ash (lb/yd^3)
A-PRES	635	283	1886	1106	0
A-PERF	502	283	1886	1106	133
B-PRES	635	283	1796	1366	0
B-PERF	502	283	1796	1366	133
C-PRES	635	283	1955	1153	0
C-PERF	502	283	1955	1153	133
D-PRES	635	283	1917	1187	0
D-PERF	502	283	1917	1187	133

4.3 Methods

Eight 4 in. by 8 in. cylinders of concrete were cast for each mixture: 3 cylinders each for compressive strength testing at 28 and 56 days, and 2 cylinders for RCPT at 56 days. After 24 hours, the cylinders were stripped from their molds and an initial surface resistivity measurement was made on three of the eight concrete samples (labeled 1, 2, and 3) in accordance with AASTHO TP95-11. The samples were then stored in a fog room at 100% relative humidity and 70ºF for 56 days, during which time the surface resistivity of samples 1, 2, and 3 for each mixture were periodically measured. Measurements were made once daily for the first 7 days, every other day for the next 7 days, and 1 to 2 times per week for the remainder of the 56 day period.

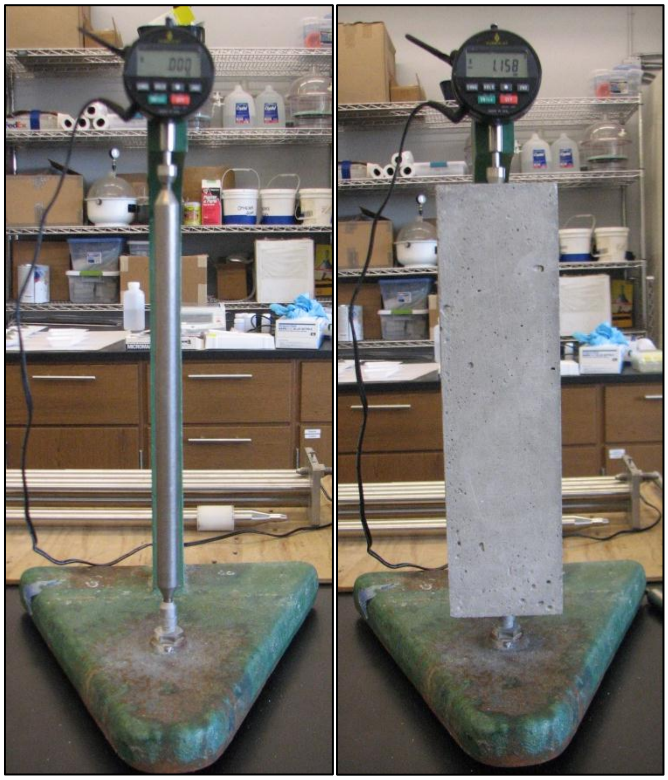
Compressive strength was measured on samples 4, 5, and 6 for each mixture after 28 days of curing, and on samples 1, 2, and 3, after the final resistivity measurement had been made on day 56. Surface resistivity was measured on samples 4-6 prior to testing to ensure that the mixture properties were consistent with those of samples 1-3.

After 56 days of curing, a 2 in. disk was cut from the center of the final two cylinders (7 and 8) cast for each mixture. The disks were conditioned overnight under vacuum pressure and the Rapid Chloride Permeability Test (RCPT) was performed the following day in accordance with AASHTO T277/ASTM C1202. Once again, surface resistivity was measured just prior to cutting to ensure consistency with the other 6 samples for each mixture.

Three 3 in. by 3 in. by 11.25 in. concrete prisms were also cast for each mixture for a modified AASHTO T160/ASTM C157 shrinkage test based on the current Alabama DOT Specifications [54]. Prisms were not cast for the B-PERF mix due to a shortage of materials. After 24 hours of sealed curing at room temperature, the prisms were removed from their molds and an initial comparator reading was made for each sample, as shown in Figure 11. The samples were then placed in a saturated limewater bath to cure for 7 days. At the end of the curing period, the prisms were removed from the water and allowed to equilibrate with the ambient conditions for 30 minutes. A second comparator reading was then made for each sample at 7 days ± 0.5 hours of age. The prisms remained exposed to ambient conditions (73 ± 3ºF and 50 ± 4% relative humidity) for the remainder of the 28 day testing period, with length changes measured 1, 4, 7, 14, and 21 days after removal from water. The length change of the prism was calculated as the difference between the comparator reading (CRD) at a given age and the initial comparator reading, divided by an assumed gage length of 250 mm (10 in.).

$$\text{Length change} = \frac{\text{CRD} - \text{initial CRD}}{\text{Gage Length}} \times 100\%$$

At the end of the testing period, the 28-day drying shrinkage was measured and compared for each mixture. A 28-day shrinkage limit of 0.04% was selected as a maximum allowable drying shrinkage based on recent changes to the Alabama DOT standard concrete specifications [54].



(a) (b)

Figure 11. Drying shrinkage test set-up. (a) Calibrating the comparator using a zeroing bar. (b) Measuring the length change of the concrete prism.

4.4 Results and Discussion

4.4.1 Compressive Strength

The average compressive strengths measured for the four aggregate pairings after 28 and 56 days of curing are shown in Figure 12 and Table 26. Overall, the average 28-day strength was 4990 ± 670 psi for the four prescriptively designed mixes (“PRES”) and 3990 ± 540 psi for the four performance mixes (“PERF”). The average 56-day strength was 5630 ± 670 psi for the four prescriptive mixes and 4770 ± 540 psi for the four performance mixes. The performance mixes containing limestone and fly ash were noted during mixing to have a smoother, more uniform consistency when compared to the prescriptive mixes, which may account for the smaller variations in 28- and 56-day strength measurements. Since the only differences between mixes A, B, C and D were the aggregate sources, the variability suggested by the 670 psi and 540 psi standard deviations can be attributed primarily to the influence of the aggregates on the concrete properties.

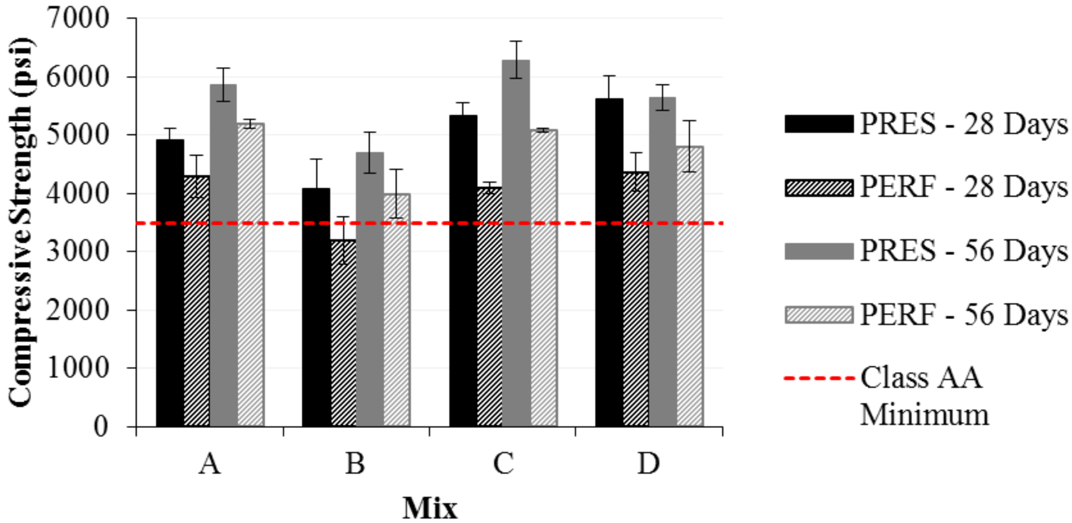


Figure 12. Compressive strength comparison for prescriptive (PRES) and performance (PERF) mixes with different combinations of aggregate, A-D.

Table 26. Average compressive strength measurements for prescriptive (PRES) and performance (PERF) mixes with different combinations of aggregate, A-D.

Mix ID	28 Days		56 Days	
	Avg. Strength (psi)	**Std. Dev. (psi)**	**Avg. Strength (psi)**	**Std. Dev. (psi)**
A-PRES	4919	191	5875	284
A-PERF	4301	365	5199	82
B-PRES	4089	496	4698	359
B-PERF	3198	403	3993	421
C-PRES	5329	238	6294	315
C-PERF	4092	102	5084	42
D-PRES	5630	393	5649	219
D-PERF	4365	332	4807	444

In general, it was observed that the strengths of performance mixes were approximately 20% lower than the prescriptive mixes after 28 days, and approximately 15% lower after 56 days. The lower strength of the performance mixes may be related to the substitution of approximately 10% of the cement clinker with fine limestone powder, which in this case can be considered largely inert, since the limestone does not react to produce any strength-giving phases. In other words, the reduction of the cement content by 10% may also reduce the strength by approximately 10%. Additional reductions in strength can be attributed to the replacement of 15% of the cement (by volume) with a slower reacting fly ash, which, like the limestone powder, initially means that there is less cement available to react. However, as the slow reaction of the fly ash with other cement hydration products proceeds, the concrete begins to gain additional strength, leading to more rapid increases in strength at later ages for the performance mixes than for the prescriptive mixes. As a result, the relative strengths of the performance mixes increase from approximately 80% of the prescriptive mix strengths at 28 days to more than 85% of the prescriptive mix strengths at 56 days.

With respect to the different aggregate sources, it was generally seen that concretes produced with the granite aggregates (Mixes C and D) had slightly higher strengths than concrete produced with the Georgia limestone aggregates (Mix A). Crushed granite tends to have slightly higher strength than Georgia crushed limestone, so it follows that concrete made with the granite aggregates will have slightly higher strengths than concrete made with the limestone aggregates. Additionally, the smoother surface of the dolomite may have further contributed to the formation of a slightly larger interfacial transition zone (ITZ) around the limestone aggregates when compared to the granite aggregates, which would also contribute to a slight reduction in strength. The ITZ is a region of higher porosity and microcracking about 10-50 μm in width localized around the coarse aggregates; when the aggregates have smoother surfaces, as is the case for the limestone aggregates, the ITZ tends to be larger and the concrete tends to have lower strengths [55].

While in general the granite concrete mixtures had higher strengths than the limestone aggregate mixtures, this was not the case for the two concrete mixes made with aggregate pair B. In addition to crushed granite coarse aggregate, Mix B also contained a fine aggregate sourced from river sediment and containing organic material. Organic material has the potential to affect the hydration reactions and reduce strength [55]. As a result, although all four prescriptive mixes and all four performance mixes were designed with identical binder proportions, the Mix B mixes had strengths approximately 1000 psi lower than the mixes with the other three aggregate pairings due to the influence of the organic impurities. If these aggregates were to be used in a GDOT project, a more thorough cleaning of the aggregates to remove the organic impurities would be required in order to achieve minimum required strengths (and to conform with the current requirements of GDOT Section 801 – Fine Aggregate [56]).

4.4.2 Dimensional Stability

The length change measurements over the 28-day drying shrinkage test period are shown in Figure 13, and the 28-day drying shrinkage results are summarized in Figure 14 and Table 27. As previously mentioned, shrinkage specimens for mix B-PERF could not be made due to a shortage of materials. Nonetheless, it is apparent from the results that the performance mixtures, containing fine limestone powder and fly ash, shrank approximately 0.005% more, on average, than their companion mixtures containing only portland cement, irrespective of aggregate source. Coarse aggregates in concrete provide restraint against drying shrinkage and hence it is the size and stiffness of the aggregate that are of significance [55]; since all four aggregate pairings had similarly graded aggregates (#57 gradation), the relative stiffnesses of the two aggregates are of primary interest. At the early ages that are of concern in the modified shrinkage test, both granite and limestone aggregates have been shown to have comparable degrees of drying shrinkage [55], so it is reasonable to expect drying shrinkage in this modified shrinkage test to remain relatively constant among different aggregate pairs. In fact, all three performance mixes had nearly identical length changes at all ages, while the prescriptive mixes A, C, and D were not markedly different after the first 7 days of air curing. One notable exception, again, is mix B, where the organic material in the fine aggregate may have altered the kinetics of the hydration reaction, potentially leading to increased drying shrinkage.

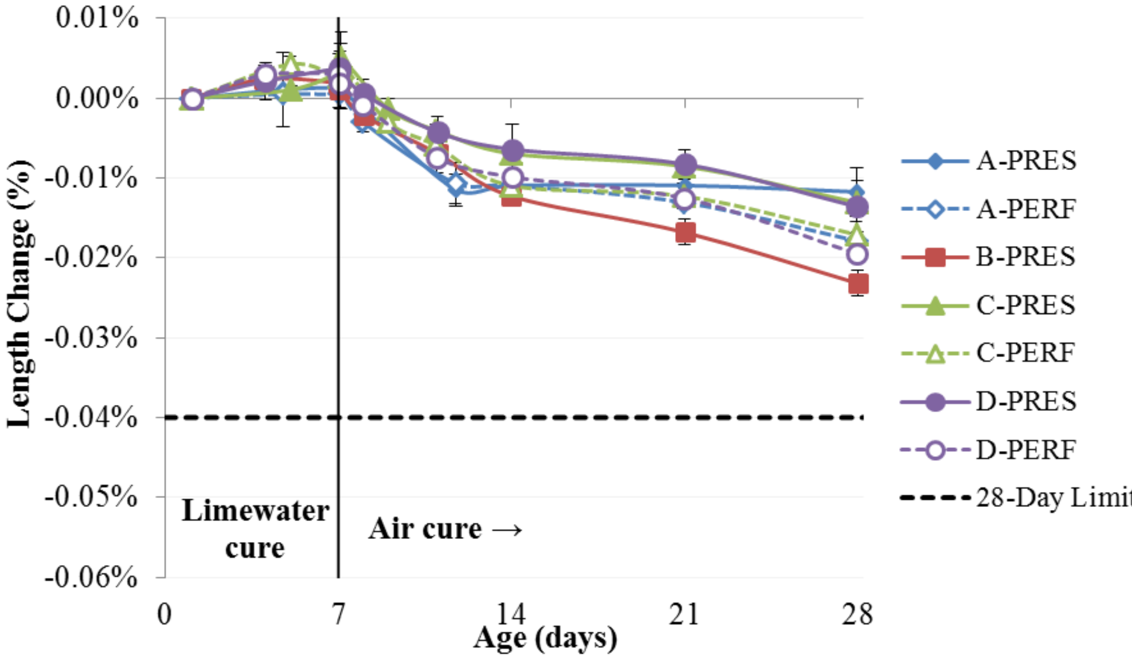


Figure 13. Length change over time for drying shrinkage specimens. The drying shrinkage specimens were removed from the limewater bath after 7 days.

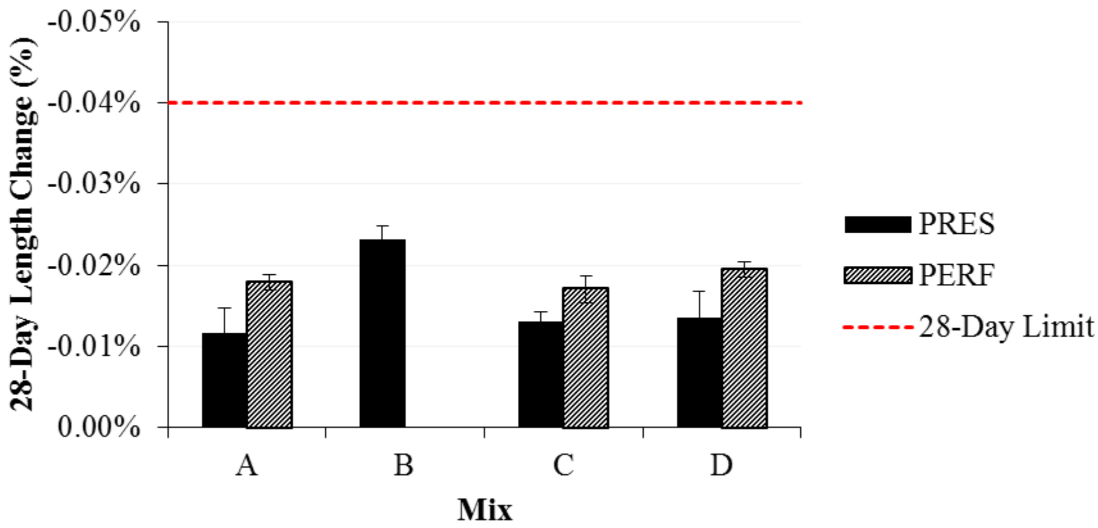


Figure 14. 28-day length change for prescriptive (PRES) and performance (PERF) mixes tested according to a modified AASHTO T160 drying shrinkage test. The 28-day limit shown is based on the current Alabama DOT standard concrete specifications.

Table 27. Average 28-Day drying shrinkage for prescriptive (PRES) and performance (PERF) mixes.

Mix ID	28-Day Shrinkage	Standard Deviation
A-PRES	-0.012%	0.003%
A-PERF	-0.018%	0.001%
B-PRES	-0.023%	0.002%
C-PRES	-0.013%	0.001%
C-PERF	-0.017%	0.002%
D-PRES	-0.014%	0.003%
D-PERF	-0.019%	0.001%

A more important factor affecting drying shrinkage at early ages is the binder composition. Binders containing finer particles like limestone powder and fly ash tend to exhibit increased drying shrinkage as a result of greater pore refinement during hydration [55]. Because roughly 30% of the cement in the performance mixtures was replaced by a finer combination of limestone powder and fly ash, significant increases in drying shrinkage would be expected. Accordingly, it was observed that the three performance mixtures all had slightly higher drying shrinkage than their companion prescriptive mixes, with an average shrinkage of 0.018% versus 0.013% for the performance and companion prescriptive mixes, respectively. Nevertheless, the 28-day drying shrinkage of all seven mixes examined fell well below the 0.04% limit proposed in the Alabama DOT standard concrete specifications [54], suggesting that although the combination of limestone cements and fly ash increased the total shrinkage of the concrete, the difference should not cause appreciable differences in the overall cracking potential of the concrete structure.

4.4.3 Permeability

The surface resistivity development trends for the eight mixtures are shown in Figure 15, and the 28- and 56-day resistivity values are compared in Figure 16. The surface resistivity development trends are consistent with those discussed in the first part of this study: the four mixes containing only portland cement (the prescriptive mixes, PRES) leveled off in resistivity after about 14-21 days, while the four limestone and fly ash mixes (the performance mixes, PERF) continued to increase in resistivity throughout the 56 day testing period. After 56 days, all four performance mixes had resistivity values indicative of "moderate" permeability, while only two of the four prescriptive mixes (C and D) did. Since the performance mixes were designed to achieve moderate permeability values after 56 days, they performed as expected.

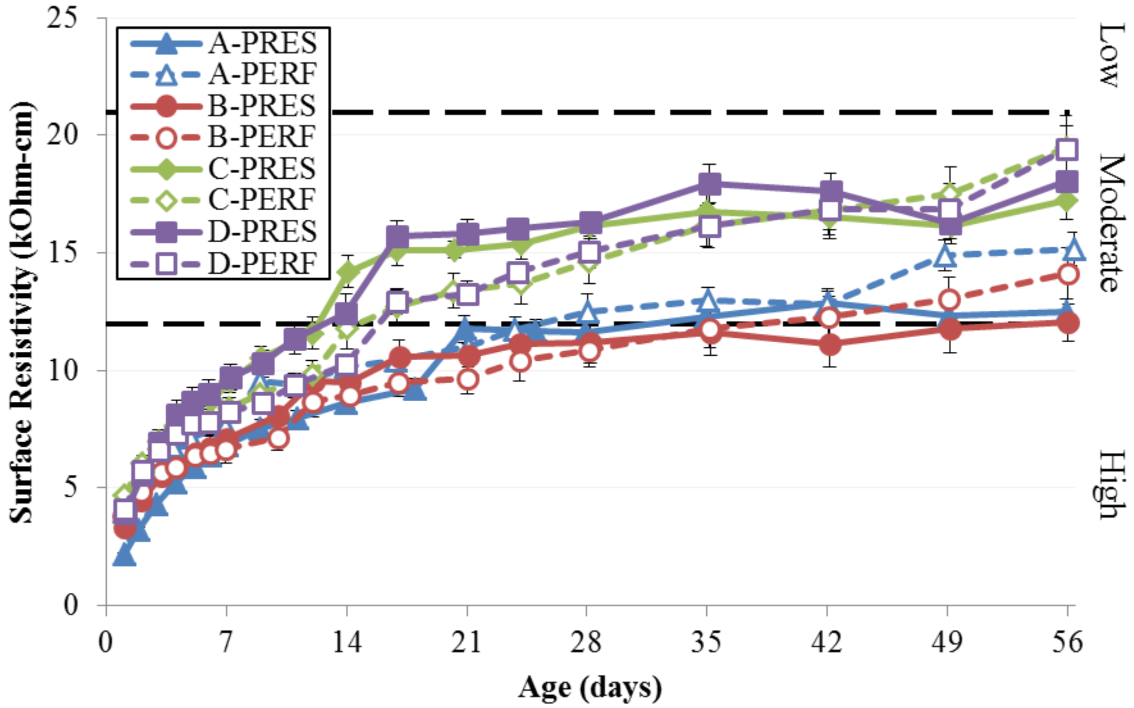


Figure 15. Surface resistivity development over time for prescriptive (PRES) and performance (PERF) mixes. Horizontal dashed lines indicate boundaries between high and moderate permeability and moderate and low permeability at 28 days, according to AASHTO TP 95-11.

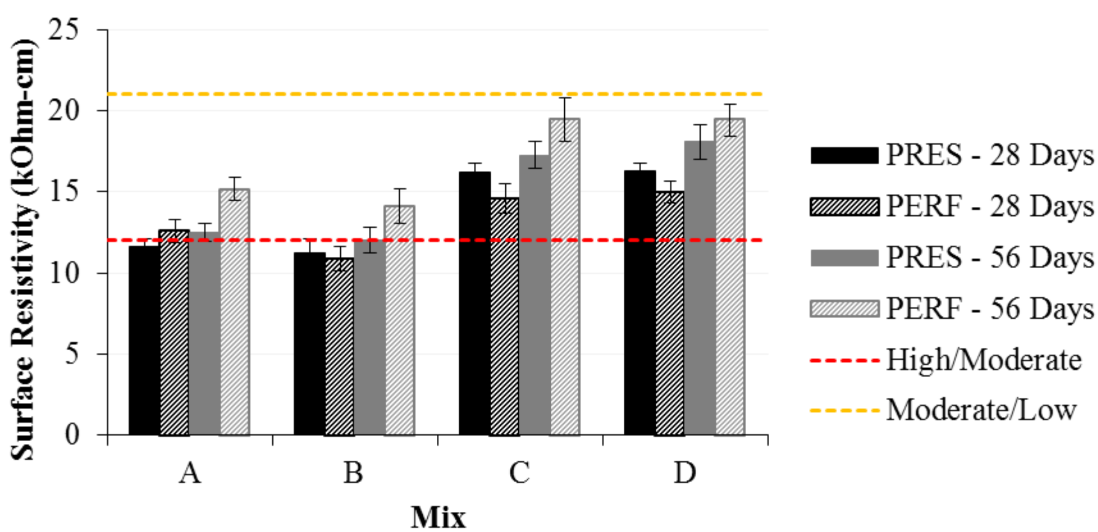


Figure 16. Surface resistivity measurements for prescriptive (PRES) and performance (PERF) mixes at 28 and 56 days.

There were, however, significant differences between how each aggregate pairing performed in the resistivity testing. Mixes C and D, for example, had significantly higher resistivity values than Mixes A and B. As discussed in the previous section, Mix B's fine aggregate contained organic material that altered its hydration and strength; it is not surprising, therefore, that the organic matter also appears to have altered the concrete's electrical resistivity. A more significant finding is that the dolomitic limestone aggregates in Mix A lead to resistivity values that were approximately 20-30% lower than the crushed granite aggregates in Mixes C and D. There are two primary explanations for this behavior. First, although the electrical resistivity of concrete is largely controlled by the resistivity of the pore solution and the cement paste matrix, the electrical resistivity of the aggregates can play a significant role in determining the overall resistivity of the concrete composite – particularly when mixtures containing different aggregate sources are compared to one another. It has been observed that dolomite is more electrically conductive than granite [57], and therefore, concretes containing dolomite aggregates will have lower electrical resistivity than concretes containing a similar amount of granite aggregates, as was observed.

A second possible explanation for the decrease in surface resistivity is that the surface characteristics of the limestone aggregates may have resulted in a larger ITZ in Mix A concretes than in the Mix C and D concretes. In addition to decreasing strength, a larger ITZ will also lead to higher permeability, since it creates a region within the concrete of high porosity and interconnected microcracks. Further research is needed to better understand whether the lower resistivity values observed for the dolomitic limestone aggregate concretes are due to an increase in the size of the ITZ, an increase in electrical conductivity of the concrete composite, or a combination of the two effects.

Similar trends were observed for the 56-day RCPT results (Figure 17) as were observed for the SR test. The RCP test uses electrical conductivity – the inverse of resistivity – as an indirect measure of the concrete's permeability, so factors that affect the electrical resistivity of the concrete should similarly affect its electrical conductivity. Again, the RCPT results suggest that Mix A is considerably more permeable than Mixes C and D, but as previously discussed, this result may have arisen from differences in the relative conductivities of the two aggregate sources, and not actually due to differences in overall permeability. (A similar discrepancy has also been reported for concrete containing silica fume, which similarly increases electrical conductivity while decreasing permeability [58]). This subtle disparity is important when it comes to placing limits on performance, as concretes containing limestone aggregates and granite aggregates, for example, may have similar permeabilities, but their electrical properties such as resistivity or conductivity may be significantly different from one another. An understanding of *how* particular test results may be influenced by factors such as aggregate source or environmental exposures is essential for setting performance requirements and interpreting test results in the context of a performance-based specification. Results of indirect tests such as surface resistivity and RCPT, in particular, should be interpreted with care in order to avoid rejecting good-quality mixtures or accepting poor-quality mixtures.

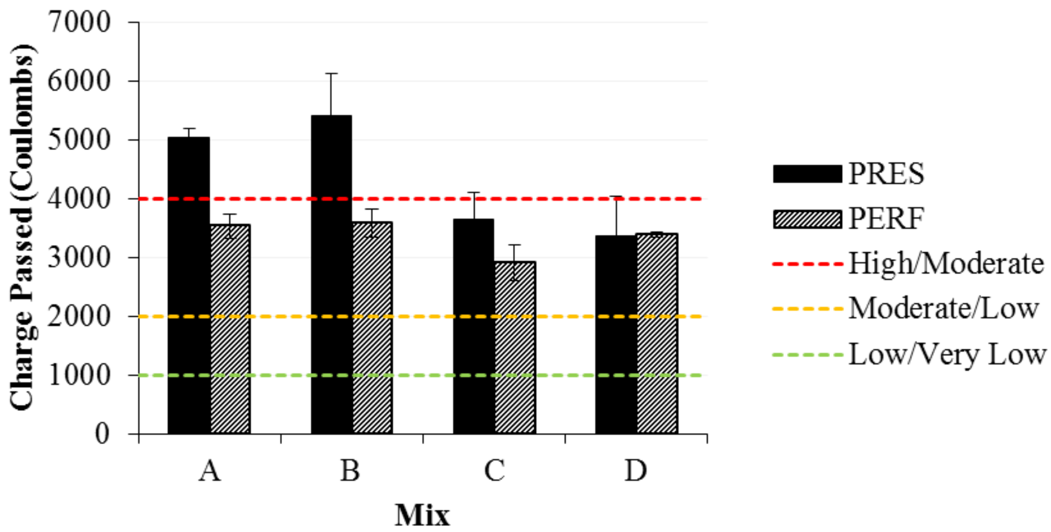


Figure 17. RCPT results for prescriptive and performance mixes. Higher values indicate more permeable concrete. Error bars indicate range of values obtained.

Finally, it should be noted that concrete mixtures with high surface resistivity or low charge passed during RCPT do not necessarily have the highest strengths; in other words, strength and permeability, while both largely controlled by the internal structure of the concrete, are not always directly related to one another. Figure 18 shows the relationship between the compressive strength and surface resistivity measurements for all individual concrete specimens tested at 28 and 56 days. While there is a general positive correlation between the two measured properties, mixes with high surface resistivities do not always have the highest strengths. In fact, one of the highest strength specimens measured after 56 days (denoted in Figure 18 by the arrow, with a strength of 6100 psi) had one of the lowest surface resistivity values measured after 56 days of curing (12.3 kOhm-cm).

The overall linear correlation coefficient between the measured compressive strength and measured surface resistivity is 0.55. Because the correlation coefficient can range from 0 (no linear correlation) to ±1 (perfect linear correlation), a coefficient of 0.55 indicates that the two properties are only moderately correlated (linearly) to one another. A modest linear correlation does not necessarily mean that the two properties are not related to one another by some other as yet unknown function, but it does mean that it cannot be concluded on the basis of a high strength measurement alone that a concrete mixture will definitively have low permeability or vice versa. If low permeability is a desired performance criterion, then it cannot be assumed that a high-strength concrete will necessarily also be a low-permeability concrete – both criteria must be examined independently of one another.

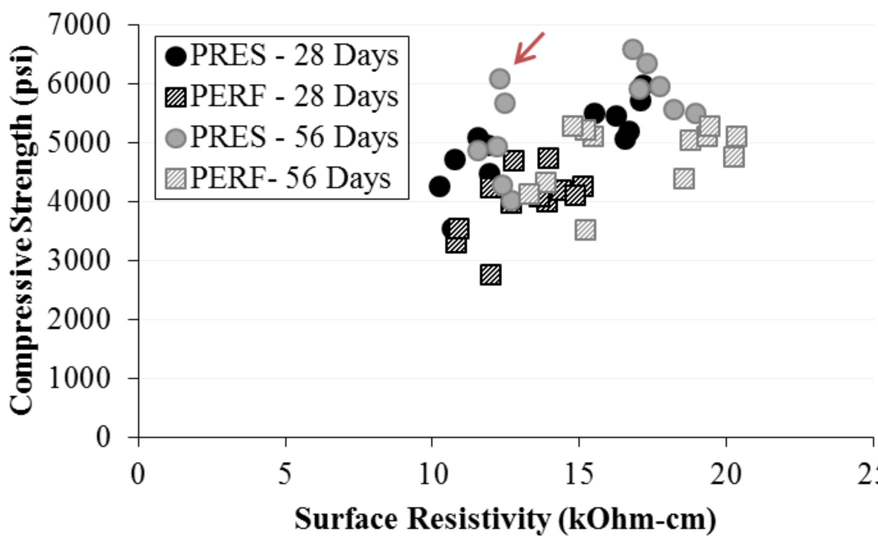


Figure 18. Relationship between surface resistivity and compressive strength for all performance (PRES) and prescriptive (PERF) test specimens at 28 and 56 days of age. The red arrow indicates a high-strength mixture with a low surface resistivity (potentially high permeability).

<u>4.4.4 Chloride Ion Diffusivity</u>

As in Section 3.4.3, the chloride ion diffusion coefficients for the eight mixtures were computed using the methods of Barde, et al. [50]. The results are shown in Table

28. The four prescriptive mixtures had an average predicted diffusion coefficient of 1.27 $\pm$ 0.29 $\times$ 10^{-8} in^2/s (8.18 $\pm$ 1.90 $\times$ 10^{-12} m^2/s), while the four performance mixtures had significantly lower diffusion coefficients at 0.975 $\pm$ 0.086 $\times$ 10^{-8} in^2/s (6.29 $\pm$ 0.57 $\times$ 10^{-12} m^2/s). These correspond to expected service lives of approximately 12.7 ± 1.0 years and 19.8 ± 1.2 years, respectively, when simulated under extreme chloride exposures (coastal Savannah) in Life 365, using the same assumptions discussed in Section 3.4.3. Under less severe chloride exposures (urban Atlanta), the average expected service lives were 58.5 ± 4.3 years and 91.3 ± 3.8 years, respectively. It can therefore be concluded that the performance-based mix designs considered in this study, while not currently satisfying the requirements of the GDOT Section 500 Standard Specifications for structural concrete, can nevertheless result in less permeable, more durable concrete with longer service lives and lower materials costs.

Table 28. Predicted 56-day chloride ion diffusivity coefficients and expected service lives for prescriptive (PRES) and performance (PERF) mixes.

Mix ID	**56 Day RCPT (Coulombs Passed)**	**Predicted Diffusion Coefficient (m^2/s)**	**Predicted Diffusion Coefficient (in^2/s)**	**Predicted Service Life: Extreme Exposure (years)**	**Predicted Service Life: Low Exposure (years)**
A-PRES	5039	9.45E-12	1.46E-08	12.0	55.4
A-PERF	3535	6.63E-12	1.03E-08	19.1	89.1
B-PRES	5401	1.01E-11	1.57E-08	11.7	54.3
B-PERF	3586	6.72E-12	1.04E-08	18.9	88.5
C-PRES	3640	6.82E-12	1.06E-08	13.4	61.4
C-PERF	2918	5.47E-12	8.48E-09	21.5	96.8
D-PRES	3377	6.33E-12	9.81E-09	13.8	62.9
D-PERF	3388	6.35E-12	9.84E-09	19.6	90.7

4.5 Conclusions

In this experimental program, it was demonstrated that the types of aggregates used for concrete mixtures may have a significant impact on the measured performance of the concrete. With respect to compressive strength, using a stronger aggregate like granite will naturally tend to produce slightly stronger concrete, as was demonstrated by Mixes C and D, but impurities in the fine aggregates may negate those improvements and instead significantly decrease the overall strength of the concrete, as was demonstrated by Mix B. Impurities in the aggregates may similarly lead to increases in permeability as measured by both surface resistivity and RCPT measurements; in addition, impurities may lead to increases in drying shrinkage, due to the alterations in the cement hydration reaction. Therefore, it is recommended that all aggregates used under a performance-based specification conform to the prescriptive requirements specified in GDOT Section 801 – Aggregates [56], unless it can be demonstrated that their use does not compromise the ability of the concrete to perform as required.

With respect to permeability, it was further demonstrated that the electrical properties of the aggregates play a small but potentially significant role in the interpretation of surface resistivity and rapid chloride permeability test results. Dolomite, with a higher electrical conductivity, conducts electricity more easily than granite does, which may result in significantly higher charges passed during RCPT and lower surface resistivity measurements, even if the concrete itself is no more permeable than if granite had been used instead. On the other hand, the surface features of the aggregates, including a smoother texture, may have instead led to an increase in the size of the interfacial transition zone around the aggregate, which would have contributed to real increases in permeability and decreases in resistivity. Thus, it cannot be concluded on the basis of electrical measurements alone that the use of limestone aggregates makes the concrete any more permeable than if granite aggregates had been selected instead. In a performance-based specification, it is important to understand that the type and source of aggregates may lead to significant variability in the electrical properties of concrete, and that this variability may lead to incorrect interpretations of the permeability of the concrete examined. If a rapid electrical test is to be used to indirectly assess the permeability of concrete, the results should be interpreted with caution, taking into consideration the types of aggregates being examined. Further research may be required to determine whether the permeability assessment of the concrete under consideration is adversely affected by the electrical properties of the aggregates or if the differences observed are in fact due to a real increase in the concrete's permeability. If the former is the case, it may be necessary to adjust the resistivity or conductivity limits suggested in AASHTO TP95 and AASHTO T277/ASTM C1202, respectively, to account for differences in aggregate sources in a performance-based specification.

Unlike compressive strength and permeability, however, there was little difference observed in the drying shrinkage after 28 days for concrete made with granite aggregates versus concrete made with limestone aggregates. Because the coarse aggregates act as restraints to drying shrinkage, it is their size and stiffness that are important in controlling drying shrinkage. Since all four aggregate pairings used similar aggregate sizes with relatively similar stiffnesses, few differences were observed between pairings, aside from the influence of the impurities in Mix B previously discussed. A more significant influence to drying shrinkage was the composition of the binder itself, with finer additions such as limestone and fly ash contributing to greater shrinkage relative to conventional concrete mixtures due to the greater pore refinement during cement hydration.

In general the performance mixes, which were designed to achieve moderate-to-low permeability at 56 days, performed significantly better than the companion prescriptive mixtures in that regard. However, in every other criteria examined, the prescriptive mixes performed slightly better, having approximately 15-20% higher strength at 28 and 56 days and 0.005% less drying shrinkage at 28 days. Nevertheless, it was shown that if low permeability is the primary performance criterion for a structure, a concrete mixture that does not conform to current prescriptive standards could be designed to meet those requirements while also providing adequate compressive strength and drying shrinkage resistance.

5. CONCLUSIONS

Performance-based specifications provide an alternative means of designing concrete. Instead of specifying materials, means, and proportions, as current prescriptive specifications do, a performance-based specification instead indicates the functional requirements for concrete in the plastic and hardened states and allows the concrete producer to design a mixture that meets those requirements. The most commonly specified performance criteria include low permeability, good sulfate resistance, and adequate freeze-thaw durability, but other requirements such as dimensional stability and high strength may also be indicated. The specific manner of implementation of performance-based specifications varies from one agency to another, and several different approaches have been discussed. The "exposure class" system, in which performance criteria are specified based upon environmental exposure conditions, has been particularly successful in states and countries with varying climate and geography and is recommended for consideration in future GDOT specifications. Additionally, it has been seen in some states and countries that performance requirements for concrete can depend on the particular application for that concrete; that is, concrete designed for use in a bridge deck may have a different set of performance requirements than concrete designed for pavements or substructures, even under the same environmental exposures. Therefore, it is recommended that, if GDOT adopts a performance-based specification option, the functional requirements for the concrete should be considered in the context of their application and not simply based on their environmental exposures.

The most commonly specified performance criterion in the specifications reviewed for this study is low permeability; as a result, this criterion was selected as the primary focus for the investigation. Permeability testing conducted as part of this research effort indicated that surface resistivity testing can be used to quickly compare the permeability of several concrete mixtures, which is especially useful for performance-based mix design and assessment of concrete quality. The influence of SCM content, cement type, and w/cm can be clearly seen in the surface resistivity development curves, which provide insight into which combinations of materials could result in the desired permeability and performance characteristics.

The results show that surface resistivity testing per AASHTO TP95 is a viable alternative to the currently specified AASHTO T277/ASTM C1202 Rapid Chloride Permeability Test and that both tests should be considered for use in performance-based specification guidelines. However, before the surface resistivity test can be fully implemented, future work should be done to better understand the discrepancies between the overall permeability classifications provided by the two tests, as well as the effect of aggregate properties on the electrical properties of the concrete. As with RCPT, surface resistivity testing should be performed at the earliest at 56 days of age for concrete mixes containing SCMs; however, it would be most effective to test at several ages so that trends in resistivity may be observed, particularly for performance-based mixture designs which may contain SCMs with varying rates or ages of reaction.

Finally, it has been shown that lower concrete permeability can be achieved in performance-based concretes containing alternative cements currently not allowed under GDOT Section 500 guidelines, when compared to their prescriptively designed counterparts. Since permeability is of great concern when designing durable concretes with long service lives, mixes specially-tailored for reduced permeability can provide high performance at a relatively low cost, when designed under a performance-based specification. Although the compressive strength of the particular performance mixtures considered in this study was lower than for the conventional concrete mixtures and the drying shrinkage higher, the differences were not expected to play a significant role in decreasing the overall serviceability of the concrete as the properties were still well within the specified limits. If, however, strength and shrinkage were of additional concern for a particular structural application, a more suitable performance mixture could be designed to meet all three of the required criteria.

6. RECOMMENDATIONS

Based upon the results of this study, it is recommended that GDOT consider implementing performance criteria for Class A, AA, and AAA structural concretes. One possible method of doing so would be to add optional performance requirements to the current Section 500 – Concrete Structures Specification. Section 500 currently includes requirements for air entrainment and strength, but additional requirements for permeability, freeze-thaw and sulfate resistance, and drying shrinkage could also be included as an addition or substitute for structural applications for which such criteria may be significant. A proposed modification to Section 500 – Concrete Structures is provided in the appendix. The requirements for permeability and drying shrinkage are modeled after the Alabama DOT's Section 501 Specification for Structural Portland Cement Concrete, which requires that:

(1) the total charge passed in RCPT (AASHTO T277) at 56 days be less than 2000 Coulombs for any concrete used in a marine or seawater environment, within 10 miles of coastline, or exposed to brackish water; and

(2) the maximum 28 day drying shrinkage (AASHTO T160) for bridge superstructure concrete be less than 0.04%, when using the modified 7 day moist cure procedure discussed in Chapter 4 [54].

A surface resistivity-based permeability requirement may also be specified instead of or as an alternative to the proposed RCPT requirement, using the AASHTO TP95 permeability classification limits as guides. Although surface resistivity testing is currently not utilized by GDOT for quality control testing, equivalent surface resistivity values are also provided in the proposed modifications to the Section 500 specification.

Upon further research, requirements for additional environmental exposures such as sulfate resistance and freeze-thaw durability may also be added to the specification, eventually culminating in a full performance-based specification option to complement the existing prescriptive specifications. This could be modeled after the existing performance-based specification option in Canada's CSA 23.1 Standard Specification [3], or after the proposed specification provided in the National Ready-Mixed Concrete Association's "Guide to improving specifications for ready mixed concrete" [2].

In a performance-based specification option, it is important to understand that not every aspect of concrete performance can be quantified, nor can every performance-related issue a concrete structure faces be accounted for in a performance-based specification. Additionally, in states that have implemented performance-based options, it has been noted that smaller concrete producers tend to have more limited resources, making it more difficult for them to compete with the larger companies to produce performance-based mixtures. Therefore, it is recommended that while performance measures should be incorporated into future GDOT concrete specifications, the specifications themselves should not be entirely performance-based; that is, prescriptive requirements should, at the very least, remain an option for concrete suppliers to pursue.

Finally, the results of this research effort suggest that materials currently not allowed under current GDOT specifications can be effectively used to produce good quality concrete mixtures, and with potential cost savings. It is additionally recommended that GDOT develop an approval process whereby materials such as natural pozzolans (e.g, metakaolin), pozzolanic industrial by-products (e.g., silica fume), and alternative cements (e.g., ASTM C595's limestone cements) – which are currently not considered in GDOT standards – could be used for future concrete projects. Since the use of alternative binder compositions is one of the primary means by which producers can achieve optimal performance under a performance-based specification, developing such an approval process is an essential first step in allowing producers to design and produce innovative concrete mixtures that can achieve optimal performance characteristics in an economic and sustainable way.

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APPENDIX

PROPOSED MODIFICATION TO SECTION 500 – CONCRETE STRUCTURES

Two additions to Table 1 of Section 500 – Concrete Structures are proposed. The additions consider permeability limits based on AASHTO T 277 testing for concrete subject to chloride exposures, and drying shrinkage limits based on modified AASHTO T 160 testing for concrete used in bridge decks. The proposed modifications are shown on the following page in red text.

It should be noted that if surface resistivity testing based on AASHTO TP 95 were to be used as an alternative to the AASHTO T 277 permeability assessment, a maximum charge passed of 2,000 coulombs would correspond to a minimum surface resistivity of approximately 23 kOhm-cm for concrete specimens cured in lime-saturated water at 73ºF for 56 days, based on the results of this study.

Table 1 – Concrete Mix Table

English									
Class of Concrete	(2) Coarse Aggregate Size No.	(1 & 6) Minimum Cement Factor lbs/yd^3	Max Water/ Cement ratio lbs/lb	(5) Slump Acceptance Limits (in) Lower-Upper		(3 & 7) Entrained Air Acceptance Limits (%) Lower-Upper		Minimum Compressive Strength at 28 days (psi)	Durability Requirements
"AAA"	67,68	675	0.440	2	4	2.5	6.0	5000	(8 & 9)
"AA1"	67,68	675	0.440	2	4	2.5	6.0	4500	(8 & 9)
"AA"	56,57,67	635	0.445	2	4	3.5	7.0	3500	(8 & 9)
"A"	56,57,67	611	0.490	2	4	2.5 (3)	6.0	3000	(8 & 9)
"B"	56,57,67	470	0.660	2	4	0.0	6.0	2200	(8)
"CS"	56,57,67 Graded Agg.	280	1.400	-	3½	3.0	7.0	1000 (4)	(8)

Metric									
Class of Concrete	(2) Coarse Aggregate Size No.	(1 & 6) Minimum Cement Factor kg/m^3	Max Water/ Cement ratio kg/kg	(5) Slump Acceptance Limits (mm) Lower-Upper		(3 & 7) Entrained Air Acceptance Limits (%) Lower-Upper		Minimum Compressive Strength at 28 days (MPa)	Durability Requirements
"AAA"	67,68	400	0.440	50	100	2.5	6.0	35	(8 & 9)
"AA1"	67,68	400	0.440	50	100	2.5	6.0	30	(8 & 9)
"AA"	56,57,67	375	0.445	50	100	3.5	7.0	25	(8 & 9)
"A"	56,57,67	360	0.490	50	100	2.5 (3)	6.0	20	(8 & 9)
"B"	56,57,67	280	0.660	50	100	0.0	6.0	15	(8)
"CS"	56,57,67 Graded Agg.	165	1.400	-	90	3.0	7.0	7 (4)	(8)

Notes:

1. Portland cement may be partially replaced with fly ash as provided in Subsection 500.3.04.D.4 or with granulated iron blast furnace slag as provide for in Subsection 500.3.04.D.5.
2. Specific size of coarse aggregate may be specified.

3. Lower limit is waived when air entrained concrete is not required.
4. The mixture will be capable of demonstrating a laboratory compressive strength at 28 days of 1000 psi (7 MPa) + 0.18 R*. Compressive strength will be determined based upon result of six cylinders prepared and tested in accordance with AASHTO T 22 and T 126.

   * Where R = Difference between the largest observed value and the smallest observed value for all compressive strength specimens at 28 days for a given combination of materials and mix proportions prepared together.
5. Designed slump may be altered by the Office of Materials and Research when Type "F" water reducers are used.
6. Minimum cement factor shall be increased by 50 lbs/yd^3 (30 kg/m^3) when size No. 7 coarse aggregate is used.
7. When Class A is specified for bridge deck concrete, the entrained air acceptance limits shall be 3.5% to 7.0%.
8. Concrete mixtures used in marine environments, within 10 miles (16 kilometers) from coastline, completely or partially submerged in seawater, located within the tidal and splash zones, exposed to seawater spray, exposed to brackish water, or as shown on the plans shall have a maximum charge passed of 2,000 coulombs under AASHTO T 277. Permeability will be determined based upon result of concrete specimens prepared and tested in accordance with AASHTO T 277, using a lime-saturated water cure at 73ºF for a period of 56 days.
9. When Class A, AA, AA1, or AAA is specified for bridge deck or bridge superstructure concrete, the maximum drying shrinkage shall be 0.04% measured at 28 days. Drying shrinkage will be determined based upon result of three concrete prisms measuring 3 x 3 x 11.25 inches (75 x 75 x 286 mm), exposed to drying at a concrete age of 7 days and tested in accordance with AASHTO T 160.