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Development of High Performance Concrete for Use on Tennessee Bridge Decks and Overlays

Charles Herbert Hamblin Jr.
University of Tennessee - Knoxville

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To the Graduate Council:

I am submitting herewith a thesis written by Charles Herbert Hamblin Jr. entitled "Development of High Performance Concrete for Use on Tennessee Bridge Decks and Overlays." I have examined the final electronic copy of this thesis for form and content and recommend that it be accepted in partial fulfillment of the requirements for the degree of Master of Science, with a major in Civil Engineering.

Edwin G. Burdette, Major Professor

We have read this thesis and recommend its acceptance:

J. Hall Deatherage, Earl Ingram

Accepted for the Council:
Carolyn R. Hodges

Vice Provost and Dean of the Graduate School

(Original signatures are on file with official student records.)

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Anne Mayhew
Vice Chancellor and
Dean of Graduate Studies

(Original signatures are on file with official student records.)

DEVELOPMENT OF HIGH PERFORMANCE CONCRETE FOR USE ON TENNESSEE BRIDGE DECKS AND OVERLAYS

A Thesis
Presented for the
Master of Science
Degree
The University of Tennessee, Knoxville

Charles Herbert Hamblin, Jr.
August, 2004

ABSTRACT

The purpose of this study was to develop a High Performance Concrete mix design to be used on bridge decks and overlays in Tennessee. A total of eight mix designs were tested in this study. Both gap-graded and dense-graded aggregate combinations were used in the study.

Each mix was tested for fresh and hardened concrete properties. Fresh properties include slump, air content, unit weight, and temperature. Hardened properties include 7 and 28-day compressive strength, freeze-thaw durability, drying shrinkage, and chloride ion permeability.

Although further research is recommended, one promising mix was found as a result of this study. Mix 2F1 (dense-graded with 25% fly ash replacement) was found to meet all performance characteristics and was chosen because it has possible economic savings.

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CHAPTER 1

INTRODUCTION

1.1 Research Background

Currently, in the United States, many concrete pavements and bridge decks fail long before reaching their expected service life. A survey of the data showed that about 30% of the 583,000 bridges in the U.S. are rated substandard and have major problems associated with crack deterioration in their decks (1). However, these problems may be overcome in future construction projects by using high-performance concrete (HPC) mixes that provide an improved overall durability performance which would reduce the life-cycle costs of these bridge decks (2). These incentives promoted the Federal Highway Administration, in cooperation with state highway departments, to set forth programs to construct demonstration showcase bridges with HPC throughout the United States (3). Using this program, states can share challenges, knowledge, and benefits with each other. Several states in the U.S. have begun to use HPC in bridge construction. However, the four pioneering states of Nebraska, New Hampshire, Texas, and Virginia have had the most experience with HPC. Applications of HPC in bridge construction in these states have shown excellent results, specifically in the area of durability. This success has led many DOTs to establish HPC as the standard mix for their bridge decks instead of the conventional normal-strength concrete mixes. These applications have also led to the development of related HPC specifications.

1.2 Research Objectives

The overall objective of the research project approved by the Tennessee Department of Transportation (TDOT) was to develop optimum high-performance concrete mixes for use in Tennessee cast-in-place bridge decks using local aggregates and combinations of fly ash, slag, and silica fume. Gap and dense-graded HPC mixes were explored to produce the optimum mixes with high performance concrete properties that included good workability, decreased permeability, reduced cracking potential, adequate strength, and potential economical savings. The primary objective of the research presented in this thesis was to investigate the effect of adding an intermediate size coarse aggregate on the durability of the concrete mix.

While HPC almost by definition involves concrete with relatively high strength, the emphasis in the development of optimum mixes in this project is on durability. Durability is generally evaluated, and is being evaluated in this research, in terms of resistance to freeze-thaw cycles and resistance to chloride ion penetration. In addition to these two traditional measures of durability, the amount of volume change, or free shrinkage, is also being evaluated.

The present study was organized into the following three tasks.

Task 1. Development of Mix Proportions

A total of eight concrete mixes were prepared for this study with TDOT-Class D concrete mix being the control mix. Modifications of the control mix included mixes with the following ingredients:

I. Cementitious content of 570-lb/ yd^3 rather than 620-lb/ yd^3 in Class D.
II. Aggregate gradations: Gap-graded (2 aggregates) and Dense-graded (3 aggregates).
III. Four supplementary cementitious materials combinations (fly ash, fly ash and silica fume, slag, and slag and silica fume).

Task 2. Evaluation of Performance of HPC Mixes

Each mix was tested for the fresh and hardened concrete properties. The fresh concrete properties included slump, air content, unit weight, and temperature. The hardened concrete properties included compressive strength at 7 and 28 days of age, drying shrinkage, freeze-thaw durability, and chloride ion permeability.

Task 3. Selection of Optimum Mixes

The most promising mix(es) were identified and selected for further verification in phase II of this study. An optimum HPC mix would be expected to have a very low chloride permeability value (below 1000 Coulomb), a high freeze-thaw durability factor (above 80%), a low shrinkage value (between 400 and 600 microstrain), and a minimum compressive strength of 4000 psi.

CHAPTER 2

LITERATURE REVIEW

This chapter discusses the definitions of high performance concrete (HPC) proposed by other studies and how they pertain to this study. Findings from other studies have also been included to help understand what type and amount of materials were included to achieve the desirable HPC performance properties. The discussion also includes the HPC specifications set forth by the national leader states.

2.1 General Definition of HPC for Highway Structures

Many different definitions have been proposed for HPC. The Strategic Highway Research Program (SHRP), the American Concrete Institute (ACI), and the Federal Highway Administration (FHWA) each have a different definition for HPC. A brief description is given below.

2.1.1 SHRP Definition of HPC

SHRP (4) defined HPC as “any concrete which satisfies certain criteria proposed to overcome limitations of conventional concretes may be called High-Performance concrete.” Their design proportions criteria include a maximum water/cementitious material ratio of 0.35, and their performance criteria include a minimum relative dynamic modulus of elasticity (durability factor) of 80%, and a minimum strength meeting one of the criteria give in Table 2.1.

2.1.2 ACI Definition of HPC

The American Concrete Institute (ACI) defined HPC as "concrete which meets special performance and uniformity requirements that cannot always be achieved

Table 2.1 SHRP High-Performance Concrete Strength Criteria.

HPC Strength Rating	Strength, Age
Very Early Strength (VES)	3000 psi, 4 hours
High Early Strength (HES)	5000 psi, 24 hours
Very High Strength (VHS)	10000 psi, 28 days

routinely by using only conventional materials and normal mixing, placing, and curing practices" (5). The requirements may involve enhancements of placement and compaction without segregation, long-term mechanical properties, early-age strength, volume stability, or service life in severe environments. Concretes possessing many of these characteristics often achieve higher strength. Therefore HPC is often high strength, but high strength concrete may not necessarily be High-Performance.

2.1.3 Federal Highway Administration Definition of HPC

Goodspeed et al. (6) has broken down HPC into different grades. Table 2.2 shows each grade along with their characteristics.

2.1.4 Other Definitions of HPC

Forster (7) defined HPC as “a concrete made with appropriate materials combined according to a selected mix design and properly mixed, transported, placed, consolidated, and cured so that the resulting concrete will give excellent performance in the structure in which it will be exposed, and with the loads to which it will be subjected for its design life”.

2.2 HPC Mixtures Used by National Leaders

In 1994, the Federal Highway Administration (FHWA) started a national program to apply HPC into highway bridges, (8). To help implement HPC technology into

Table 2.2 FHWA Performance Grades in US Units (7).

Performance Characteristic	FHWA HPC Performance Grades			
	1	2	3	4
Freeze-thaw durability[1] (x = relative dynamic modulus of elasticity after 300 cycles)	$60\% \leq x < 80\%$	$80\% \leq x$		
Scaling Resistance[2] (x = visual rating of the surface after 50 cycles)	x = 4, 5	x = 2, 3	x = 0, 1	
Abrasion resistance[3] (x = avg. depth of wear in inches)	$2/25 > x \geq 1/25$	$1/25 > x \geq 1/50$	$1/50 > x$	
Chloride Penetration[4] (x = coulombs)	$3000 \geq x > 2000$	$2000 \geq x > 800$	$800 \geq x$	
Strength[5] (ksi) (x = compressive strength)	$6 \leq x < 8$	$8 \leq x < 10$	$10 \leq x < 14$	$x \geq 14$
Elasticity[6] (psi) (x = modulus of elasticity)	$4 \leq x < 6\ x10^6$	$6 \leq x < 7.5\ x10^6$	$x \geq 7.5\ x10^6$	
Free Shrinkage[7] (x = micro-strain)	$800 > x \geq 600$	$600 > x \geq 400$	$400 > x$	
Creep[8] (per psi) (x = micro-strain/pressure unit)	$0.52 \geq x > 0.41$	$0.41 \geq x > 0.31$	$0.31 \geq x > 0.21$	$0.21 \geq x$

Note: Information not specified if left blank.

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[1] *Test in accordance with AASHTO T 161 (ASTM C 666 Procedure A)*
[2] *Test in accordance with ASTM C 672*
[3] *Test in accordance with ASTM C 944*
[4] *Test in accordance with AASHTO T 277 (ASTM C 1202)*
[5] *Test in accordance with AASHTO T2 (ASTM C 39)*
[6] *Test in accordance with ASTM C 469*
[7] *Test in accordance with ASTM C 157*
[8] *Test in accordance with ASTM C 512*

highway design, the American Association of State Highway and Transportation Officials (AASHTO) Task Force on Strategic Highway Research Program (SHRP) teamed up with the FHWA to establish an HPC lead state team in 1996. Four major states led this effort to implement HPC technology into highway designs. They were Texas, New Hampshire, Virginia, and Nebraska. Eight other states (Alabama, Colorado, Georgia, Florida, New York, North Carolina, Ohio, and Washington) then joined the efforts shortly thereafter.

2.2.1 Texas Experience with HPC

Texas has used HPC on two different bridge projects. They are the Louetta Road overpass in Houston and the U.S. 67 Bridge in San Angelo. The executive director of the Texas Department of Transportation (TxDOT), Wes Heald, says there were several factors in using HPC in the design. Two main factors were quality control and quality assurance (QC/QA). Another factor he states is teamwork between the DOT and universities, contractors, fabricators, and researchers. Teamwork is vital to make sure each group is doing their job properly and the HPC meets specifications. Heald also states that HPC can be more durable than conventional concrete. Durable and impermeable bridge decks should not damage from scaling, freeze-thaw action, shrinkage cracking, or reinforcement deterioration. This resistance to damage is expected to increase the life of the bridge and reduce rehabilitation and maintenance costs over the life of the bridge (9).

2.2.2 New Hampshire Experience with HPC

New Hampshire’s first HPC structure is Route 104 stretching over the Newfound River. This bridge is located in Bristol, NH. The objective of using HPC in a bridge structure was to minimize maintenance and to prolong the life-span of the bridge. To accomplish their objective they wanted to design a bridge deck that was highly impermeable, freeze-thaw resistant, and free of cracks. To test their design, a trial pour of 5 yd^3 was placed to simulate the actual pour. Normal finishing and curing methods were used. This trial pour also helped in determining the workability of the actual mix, and made sure proper equipment was being used. This project was very successful and led to the construction of a bridge deck with no shrinkage or transverse cracking (10).

2.2.3 Florida Experience with HPC

Florida started its research of HPC in the 1970's due to road deterioration along the coastline. The Florida Department of Transportation (FDOT) has focused its research on the durability aspects of HPC. Their mix includes using a water/cement ratio of 0.44 with a fly ash replacement between 20% and 50% of total cementitious quantities. The design had compressive strengths of 5500 psi and chloride ion penetration resistance between 2000 and 3000 coulombs (11).

2.2.4 New York State Experience with HPC

In 1994, the New York State Department of Transportation (NYSDOT) set out to produce a mix design that would enhance the life of bridge decks. The main objective of this mix design was to produce a more durable concrete. They wanted less permeability, higher cracking resistance, and the mix to be easily placed and finished. To achieve this, they needed to use a lower cement content, use pozzolans, lower the water/cement ratio, and limit the use of chemical admixtures. To test their design, they tested for compressive strength, permeability, resistance to cracking and scaling, workability, and finishability.

New York's first application of HPC was for a bridge deck on New York Route 78. The compressive strength was tested at 7500 psi at 28 days and the permeability measured 600 coulombs. Also, there were no cracks visible twenty months after construction. New York has since required HPC for all bridge decks in New York. The use of HPC has resulted in stabilizing the bid prices of construction costs in New York (12).

2.2.5 Ohio Experience with HPC

Ohio built its first HPC structure on U.S. 22 near Cambridge, OH. It was to be a 116 ft, three-span, adjacent box girder bridge with 21-inch deep simply supported boxes. However, with the use of HPC, the design was changed to a single span with 42-inch deep box girders. This eliminated the need for two pier center supports. The use of HPC had immediate benefits including a reduction in construction costs, and long-term benefits including lower maintenance costs due to better corrosion resistance (13).

Table 2.3 provides each lead state's mix proportions, and Table 2.4 provides all ten state's compressive strength and permeability requirements.

2.3 Effects of Materials Selection on HPC

There have been many successful studies on HPC investigating various materials used in the mix proportions that affect the performance properties of the HPC. These materials include fly ash, slag, and silica fume.

Chang et al. (24) performed a study solely on the durability of concrete. They concluded that HPC containing pozzolanic materials such as fly ash and slag had lower water permeability, smaller voids, and thus improved durability, than conventional concrete mixes with no pozzolanic materials.

Table 2.3 HPC Mix Proportions for Four Lead States.

State	Cement (lb/yd^3)	Fly Ash (lb/yd^3)	Slag (lb/yd^3)	Coarse Aggregate (lb/yd^3)	Fine Aggregate (lb/yd^3)	Air Ent. (oz/yd^3)	WRA (oz/yd^3)	Retard (oz/yd^3)	HRWRA (oz/yd^3)	w/c ratio
Texas	383	148		1856	1243	2.6		45		0.43
	474	221		1810	1303			22	160	0.35
	492 (Type II)	211		1900	1216	3.9		28	204	0.31
	427 (Type II)	184		1856	1239	3.9		26		0.42
	610 (Type II)			1856	1243	3.9		26		0.42
New Hampshire	506			1388	910	5	20		158	0.38
Virginia	329		329	1774	1173					0.40
	329		329	1787	1158					0.38
Nebraska	754 (Type IP)	76		1400	1409			30	135	0.38

Note: Information not specified if left blank.

Table 2.4 Compressive Strength and Permeability Requirements of Top Ten Lead States.

State	28-day Compressive Strength (psi)	56-day Permeability (Coulombs)
Alabama[9]	6000	< 1800
Colorado[10]	5000	
Georgia[11]	7000	< 2000
Nebraska[12]	8000 (56-day)	< 1800
New Hampshire[13]	6000	< 1000
North Carolina[14]	6000	
Ohio[15]		< 1000
Texas[16]	4000 - 8000	
Virginia[17]	4000 - 6000	< 2500
Washington[18]	4000	< 1000

Note: Information not specified if left blank.

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[9] Alabama [14]
[10] Colorado [15]
[11] Georgia [16]
[12] Nebraska [17]
[13] New Hampshire [18]
[14] North Carolina [19]
[15] Ohio [20]
[16] Texas [21]
[17] Virginia [22]
[18] Washington [23]

Maslehuddin et al. (25) concluded that adding fly ash or slag increased the corrosion-resisting characteristics of concrete. They also found that concrete made with blast furnace slag lowered the corrosion rate of steel.

In comparing different types of fly ash, Ellis et al. (26) concluded that Class F fly ash is more effective than Class C fly ash in resisting chloride ion penetration. They also concluded that mixes containing Class F fly ash had comparable chloride penetration to those mixes that incorporated either silica fume or slag.

Naik et al. (27) studied the effects of blending Class C and Class F fly ash together on the durability and mechanical properties of concrete. They found that mixes that incorporated the blended fly ash performed better in chloride ion penetration than those mixes without fly ash.

Hooton et al. (28) studied the influence of silica fume on chloride resistance of concrete. Their findings confirmed that silica fume has a beneficial effect on chloride penetration resistance of concrete. They concluded that 7% silica fume provided a dramatic improvement in chloride penetration resistance.

Duval and Kadri (29) performed a study on the influence of silica fume on the workability and the compressive strength of high-performance concretes. They found that for concrete with w/c ratios varying from 0.25 to 0.45, up to 10% cement might be replaced by silica fume without harming the concrete workability. Also, the slump loss of silica fume concrete increased with the increased percentage of silica fume for low w/c ratios of 0.25, but for higher ratio (0.35), the loss was less important. They also found that compressive strength increased with the silica fume content up to 20% and reached a maximum (15% higher) for a 10 to 15% silica fume level.

Oh et al. (30) studied the effects of silica fume, fly ash, and blast-furnace slag on the development of chloride penetration resistance of high-performance concrete. The concrete containing silica fume showed the best performance among the specimens in the rapid chloride permeability test. Concrete containing fly ash also showed good performance in the rapid chloride permeability test. It was found that the addition of fly ash greatly decreased the permeability of concrete even though the strength of fly ash concrete at 28 days was not improved. The replacement of cement with blast-furnace slag also decreased the chloride permeability.

Li et al (31) performed an investigation of chloride diffusion for high-performance concrete containing fly ash, microsilica, and chemical admixtures. They concluded that the resistance to chloride diffusion of HPC improved significantly when fly ash was added to the mixture. An addition of 5-10 % micro silica with 25% fly ash further improved the concrete's resistance to chloride diffusion.

Sherman et al. (32) found that adding silica fume to a concrete with water-cement (w/c) ratio of 0.46 was very significant in reducing penetration of chloride ions. Bayasi and Zhou (33) also found that silica fume greatly reduces the permeability of concrete. Malhotra (34) studied the effects of silica fume (12%) with a 0.30 w/c ratio and found that chloride ion penetration values were below 300 coulombs.

CHAPTER 3

MATERIALS AND TESTING

This chapter discusses the materials used in the HPC mix designs, as well as the testing methods used to determine the performance properties of the HPC mixes. The testing methods described in this chapter are in accordance with the procedures endorsed by the American Association of State Highway and Transportation Officials (AASHTO) and the American Society for Testing and Materials (ASTM).

3.1 Materials

3.1.1 Portland Cement

Commercially available Type I Portland cement meeting ASTM C 150 (35) was used in this study. A specific gravity of 3.15 was used for the purpose of mix proportioning. Table A.1 in Appendix A gives the chemical composition of the Type I Portland cement used in the mixes.

3.1.2 Coarse Aggregates

There were two different types of coarse aggregates used in this study. The major coarse aggregate was #57 limestone with a nominal maximum size of 1-inch. This is the standard coarse aggregate for TDOT Class D mixes. This aggregate was used in both Group Mixes. Table 3.1 gives the general properties, while Table 3.2 provides the sieve analysis of the #57.

The second coarse aggregate used in this study was #7 limestone with a nominal maximum size of ¾-inch. This aggregate was used in Group Mixes 2. Table 3.3 gives the general properties and Table 3.4 provides the sieve analysis of the #7 limestone.

Table 3.1 General Properties of Coarse Aggregate (#57 Limestone).

Properties	Value
Nominal Size (in)	1
Absorption (%)	0.49
SSD Specific Gravity	2.79
Oven-Dry Specific Gravity	2.78
Apparent Specific Gravity	2.82

Table 3.2 Sieve Analysis of Coarse Aggregate (#57 Limestone).

	TDOT Specifications		Percent Passing
Sieve Size	Low	High	Vulcan Materials (#57)
1"	95	100	100.0
3/4"	--	--	87.7
1/2"	25	60	35.2
3/8"	--	--	14.5
#4	0	10	0.9

Table 3.3 General Properties of Coarse Aggregate (#7 Limestone).

Properties	Value
Nominal Size (in)	3/4
Absorption (%)	0.35
SSD Specific Gravity	2.84
Oven Dry Specific Gravity	2.83
Apparent Specific Gravity	2.85

Table 3.4 Sieve Analysis of Coarse Aggregate (#7 Limestone).

	Percent Passing
Sieve Size	Vulcan Materials (#7)
3/4"	100.0
1/2"	89.1
3/8"	54.6
#4	2.0
#8	2.6
#16	1.4
#30	2.3
#50	2.2
#100	2.0
#200	1.1

3.1.3 Fine Aggregate

The only fine aggregate used was Ohio River Valley natural sand. This aggregate was used in both Group Mixes. The properties of the natural sand are given in Table 3.5. The sieve analysis is given in Table 3.6.

3.1.4 Fly Ash

The Class C fly ash used in this study conformed to ASTM C 618 (36). The specific gravity of the fly ash used was taken as 2.3.

3.1.5 Slag

The ground granulated blast-furnace slag used in this study conformed to ASTM C 989 (37). The specific gravity of the slag was 2.93.

3.1.6 Silica Fume

The silica fume used in this study conformed to ASTM C 1240 (38). The specific gravity of the silica fume was taken as 2.2.

3.1.7 Chemical Admixtures

There were two chemical admixtures used in the mixes of this study. One was high range water reducing agent (HRWRA), the other being air entraining agent (AEA). The HRWRA used conformed to ASTM C 494 type F (39). The AEA used conformed to ASTM C 260 (40).

3.1.8 Mixing Water

The water used for mixing was tap water. The specific gravity of water was assumed to be 62.4 lbs/ft^3.

Table 3.5 General Properties of Fine Aggregate (Ohio River Valley Sand).

Properties	Value
Nominal Size (in)	3/8
Absorption (%)	0.62
SSD Specific Gravity	2.6
Oven Dry Specific Gravity	2.58
Apparent Specific Gravity	2.63

Table 3.6 Sieve Analysis of Fine Aggregate (Ohio River Valley Sand).

	Specifications		Percent Passing
Sieve Size	Low	High	Ingram (N.S.)
3/8"	100	100	100.0
#4	95	100	99.0
#8	--	--	92.0
#16	50	90	78.0
#30	--	--	62.0
#50	5	30	15.0
#100	0	10	1.5
#200	0	5	1.0

3.2 Mixing Procedure

All mixing was performed in a standard laboratory mixer with a 7 cubic feet capacity. A picture is provided in Figure 3.1. Before mixing, the water content of each aggregate was obtained for that particular mix. These values were used to adjust the amount of water used in the mix. Water was added for dry aggregates, while water was subtracted for wet aggregates.

The mixing procedure is listed as follows:

1. Coarse aggregates were added to the mixer.
2. Chemical admixtures (HRWRA and AEA) were mixed in with half of the water, while the fine aggregate was added simultaneously.
3. Aggregates were well mixed before moving to step 4.
4. Cementitious material was added. This included cement, fly ash or slag, and silica fume (if used). The other half of the water was added at the same time to reduce the amount of cementitious material escaping as dust particles.
5. Mixer was left rotating for 3-5 minutes, and then stopped for 1-2 minutes. Upon restarting the mixer, HRWRA was added if needed to control slump values.

Mixer was left rotating for another 3-5 minutes, and then fresh concrete properties (slump, temperature, air content, and unit weight) were taken within the relevant ASTM and AASHTO standards.

3.3 Test Specimen Preparation and Curing Conditions

Six cylinders measuring 6 in. diameter by 12 in. long were cast for compressive strength testing in accordance with ASTM C 192 (41). Beams measuring 3 in. x 4 in. x 16 in. long were cast for shrinkage (three beams) and freeze-thaw testing (two beams) in

Figure 3.1 Laboratory Mixer (7 cu. ft. capacity).

accordance with ASTM C 666 (42). Three cylinder specimens measuring 4 in. diameter by 8 in. long were cast for chloride ion permeability testing in accordance with ASTM C 1202 (43).

Specimens were covered with plastic immediately after casting, and then covered with wet burlap bags. Specimens were left to cure for 24 hours, and then de-molded. All specimens were then placed in a lime water bath. A picture of the specimens after casting is presented as Figure 3.2, while a picture of the lime water bath is given in Figure 3.3. The limewater bath was kept at 72^{o}F ± 2 ^{o}F.

Compressive strength specimens were kept in the bath for 7 and 28 days before testing. Shrinkage specimens were kept in the bath for seven days and air-dried at a temperature of 70^{o}F ±5 ^{o}F and a humidity of 40% ± 6%. Shrinkage measurements were taken immediately after the removal of the specimens from the lime bath. Freeze-thaw specimens were cured in the bath for 14 days until testing. Chloride permeability specimens were cured for 56 days until testing.

3.4 Testing Procedures

3.4.1 Testing of Fresh Concrete Mixes

Tests performed on HPC fresh concrete mixes included slump, air content, unit weight, and temperature.

3.4.1.1 Slump Test

The slump of the fresh concrete mixture was measured according to AASHTO T 119 (44). Figure 3.4 shows a picture of the slump test in progress. The typical slump value was about 3.5 inches.

Figure 3.2 Specimens Topped with Wet Burlap Sacks.

Figure 3.3 Curing Lime Bath for Test Specimens.

Figure 3.4 Slump Test.

3.4.1.2 Air Content Test

The air content was determined by the pressure method according to AASHTO T 152 (45). Figure 3.5 shows a picture of the air content apparatus.

3.4.1.3 Unit Weight Test

The unit weight of the concrete mixture was measured according to AASHTO T 121 (46).

3.4.1.4 Temperature

The temperature of concrete was measured with a standard thermometer, with an accuracy of ± 1 ^{o}F. The measurements were according to ASTM C 1064 (47).

3.4.2 Testing of Hardened Concrete Mixes

Tests performed on hardened HPC mixes included compressive strength, shrinkage, freeze-thaw durability, and chloride ion penetration.

Figure 3.5 Air Entraining Test Apparatus.

3.4.2.1 Compressive Strength Test

The compressive strength of each mix was tested according to AASHTO T 22 (48) using a 400,000 lb. capacity hydraulic loading compression machine. Each mix was tested for compressive strength at 7 and 28 days of age. Six specimens were tested from each mix (three at 7-day and three at 28-day). A picture of the compression machine after a test is given as Figure 3.6.

3.4.2.2 Freeze-Thaw Durability Test

The freeze-thaw durability test was performed on each of the mixes according to AASHTO T 161 (50). After curing, each specimen was weighed and tested for fundamental frequency prior to putting in the freeze-thaw machine. Each specimen was weighed and tested for fundamental frequency every 50 cycles up to 300 cycles. After 300 cycles, the test was terminated. Each cycle is noted as the following: the temperature is lowered from 40^{o}F to 0^{o}F in a 2 hour span, and then increased back to 40^{o}F also in a 2 hour span. Figures 3.7 and 3.8 are pictures of the freeze-thaw machine and the fundamental frequency machine, respectively.

3.4.2.3 Drying Shrinkage Test

The volume or length change (shrinkage) of each mix was tested according to AASHTO T 160 (49). Three specimens were tested for each mix. Each specimen was measured on three sides, giving a total of nine readings for each mix. Readings were taken at 1, 2, 3, 4, 5, 6, 7, 14, 21, 28, 56, 112, and 224 days after the seven-day curing process. A picture of the test is given in Figure 3.9.

Figure 3.6 Compression Machine after Testing.

Figure 3.7 Freeze-Thaw Testing Machine.

Figure 3.8 Fundamental Frequency Test Apparatus.

Figure 3.9 Drying Shrinkage Test.

3.4.2.4 Chloride Ion Permeability Test

The chloride ion permeability test was performed on each mix according to AASHTO T 277 (51). Each specimen was cut to 2 in. thick from a 4 in. diameter by 8 in. long specimen. Specimens were allowed to air dry for one hour. After air-drying, specimens were coated with an epoxy sealant and allowed to dry. Specimens were then subjected to 3 hours under vacuum before de-aerated water was added. Vacuum continued to run for 1 hour with specimens submerged in de-aerated water, and then specimens were opened to the atmosphere. Specimens soaked in water for 18 ± 2 hours. The vacuum apparatus is shown in Figure 3.10. The ends of each specimen were then sealed into a Plexiglas mold. A close-up view can be seen in Figure 3.12. One end of the specimen was submerged with a 3% sodium chloride solution, while the other was subject to a 0.3N sodium hydroxide solution. An electric current of 60.0 volts DC was applied to each specimen through copper screens attached to the Plexiglas molds. The current was applied for 6 hours with the apparatus taking current and Coulomb measurements every 30 minutes. The apparatus is shown in Figure 3.11. The Coulomb value after 6 hours is taken as a measure of the permeability of the concrete.

Figure 3.10 Chloride Ion Penetration Vacuum Set-up.

Figure 3.11 Chloride Ion Penetration Test Set-up.

Figure 3.12 Close-up of Chloride Ion Penetration Sample.

CHAPTER 4

TEST RESULTS AND DISCUSSION

This chapter presents the results, a discussion of various fresh and hardened HPC mix properties, and the development of the HPC mix designs. Gap and dense aggregate gradations were used in the development of the mix designs. Variations of cementitious materials were also used in the development of the mix designs. The fresh concrete properties tested included the slump, air content, unit weight, and temperature. The hardened concrete properties tested included the compressive strength at 7 and 28 days of age, freeze-thaw durability, drying shrinkage, and chloride ion permeability.

4.1 Development of HPC Mix Designs

Two groups of four mixes were developed to identify the optimum HPC mixes, giving a total of 8 mixes. The Group 1 mixes (1F1, 1F2, 1S1, and 1S2) are defined as follows:

- 1F1: Cement/Fly ash/Silica fume = 75/25/00 (by weight).
- 1F2: Cement/Fly ash/Silica fume = 75/20/05 (by weight).
- 1S1: Cement/Slag/Silica fume = 65/35/00 (by weight).
- 1S2: Cement/Slag/Silica fume = 60/35/05 (by weight).

TDOT specifications call for a minimum cement content of 620 lb/yd^3, a maximum water/cementitious (w/cm) ratio of 0.40, and a minimum 28-day compressive strength of 4000 psi for Class D concrete bridge deck mix. In order to compare results with the TDOT Class-D mix, a w/cm ratio of 0.40 was used in all eight mixes. However, the cementitious content was reduced to 570 lb/yd^3 in both mixes. This reduction in cementitious content was made for economic reasons and to improve the performance properties of the mixes. The aggregate gradations used in the Group 1 mixes were gap-graded aggregates using coarse aggregate No. 57 limestone and Ohio River Valley natural sand.

Group 2 also contained four mixes (2F1, 2F2, 2S1, and 2S2) with the same cement/fly ash/slag/silica fume combinations as the Group 1 mixes. The only difference between Group 1 and Group 2 mixes is the transition of gap-graded aggregates in Group 1 to dense-graded aggregates in Group 2. This transition occurred by adding an intermediate-size #7 aggregate to the gap-graded aggregates used in the Group 1. Therefore, Group 2 mixes have three aggregates (#57, #7, and natural sand).

Table 4.1 provides a summary of the two group mixes with various aggregate combinations and cementitious contents. Tables 4.2 and 4.3 show the mix proportions for both groups.

Table 4.1 Summary of Group Mixes with Various Aggregate Combinations and Cementitious Contents.

Group I.D.	Aggregate Combination, (Percentage)	Cementitious Content lb/yd^3	Group Mixes
1	#57, NS (60,40)	570	1F1*, 1F2*, 1S1*, 1S2*
2	#57,#7, NS (35,25,40)	570	2F1, 2F2, 2S1, 2S2

*F1 stands for Cement/Fly ash/Silica fume = 75/25/00 (by weight).
*F2 stands for Cement/Fly ash/Silica fume = 75/20/05.
*S1 stands for Cement/Slag/Silica fume = 65/35/00.
*S2 stands for Cement/Slag/Silica fume = 60/35/05.

Table 4.2 Mix Proportions for Group 1 Mixes.

Mix I.D.	1F1	1F2	1S1	1S2
Cement (1b/yd^3)	427.5	427.5	370.5	342.0
Fly ash (1b/yd^3) (% of Cement)	142.5 (25)	114.0 (20)	---	---
Slag (1b/yd^3) (% of Cement)	---	---	199.5 (35)	199.5 (35)
Silica Fume (1b/yd^3) (% of Cement)	---	28.5 (5)	---	28.5 (5)
Water (1b/yd^3)	228	228	228	228
Coarse Aggregate, #57 (1b/yd^3)	1950	1950	1950	1950
Natural Sand (1b/yd^3)	1194	1193	1225	1215
W/C Ratio	0.40	0.40	0.40	0.40
MBT AE 90 (oz/ yd^3)	15.0	23.0	15.0	15.0
HRWRA (oz/ yd^3)	98.0	122.0	122.0	143.0

Table 4.3 Mix Proportions for Group 2 Mixes.

Mix I.D.		2F1	2F2	2S1	2S2
Cement (1b/yd^3)		427.5	427.5	370.5	342.5
Fly ash (1b/yd^3) (% of Cement)		142.5 (25)	114 (20)	---	---
Slag (1b/yd^3) (% of Cement)		---	---	199.5 (35)	199.5 (35)
Silica Fume (1b/yd^3) (% of Cement)		---	28.5 (5)	---	28.5 (5)
Water (1b/yd^3)		228	228	228	228
Combined Aggregates (1b/yd^3)	#57	1131	1131	1143	1139
	#7	822	822	831	828
	NS	1204	1204	1217	1213
W/C Ratio		0.40	0.40	0.40	0.40
MBT AE 90 (oz/ yd^3)		8.0	9.0	8.0	8.0
HRWRA (oz/ yd^3)		61.0	69.0	61.0	61.0

Figure 4.1 shows the percentage of aggregates retained on each sieve for the two group mixes. As shown in the figure, the Group 1 mixes retained a significant portion of coarse aggregates on the ½”' sieve compared with those retained on the other sieves, indicating a gap-graded gradation. With the addition of the # 7 aggregate, the Group 2 mixes have significantly less aggregate retained on the ½” sieve than the Group 1 mixes. The gradation chart indicates that Group 2 has a dense-graded gradation. The ideal well-graded aggregate, which is produced using Haystack Technique as shown in Figure 4.2, would be very difficult to obtain unless the aggregates were sieved and then recombined to retain the desired percentage on each sieve. This technique was not followed for practical considerations.

4.2 HPC Fresh Concrete Mix Properties

Table 4.4 shows the fresh properties of the two groups. As shown in Table 4.4, the slump values ranged from 2 inches to 6 inches, and the air content values ranged from 4.0% to 5.75%. These values are within the range of the TDOT requirements for Class D concrete, which specifies 2 inches to 6 inches for the slump and 4.0% to 8.0% for the air content. The fresh unit weight of all of the mixes was comparable and within the narrow range of the normal concrete unit weight (145 lb/ft^3 to150 lb/ft^3). The temperature of the concrete mixes ranged from 64°F to 82°F, which is within the TDOT specifications of 40°F to 100°F.

The demand for HRWRA was generally higher for mixes that contained silica fume, as shown in Tables 4.2 and 4.3. This was expected because the silica fume particles are 100 times smaller than an average cement particle, thus giving it a larger surface area and a larger demand for water upon mixing. However, the amount of air

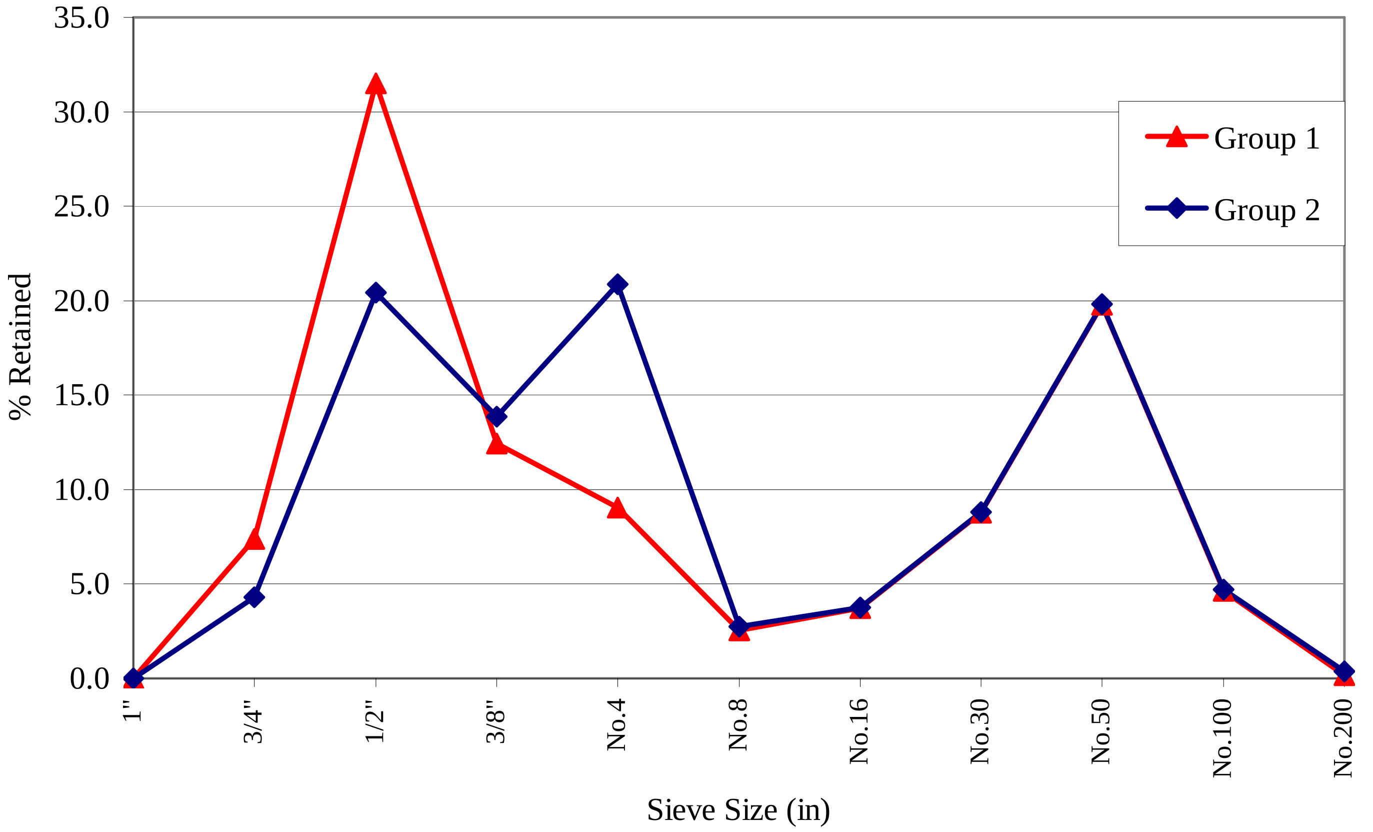

Figure 4.1 Haystack Gradation Chart for HPC Groups.

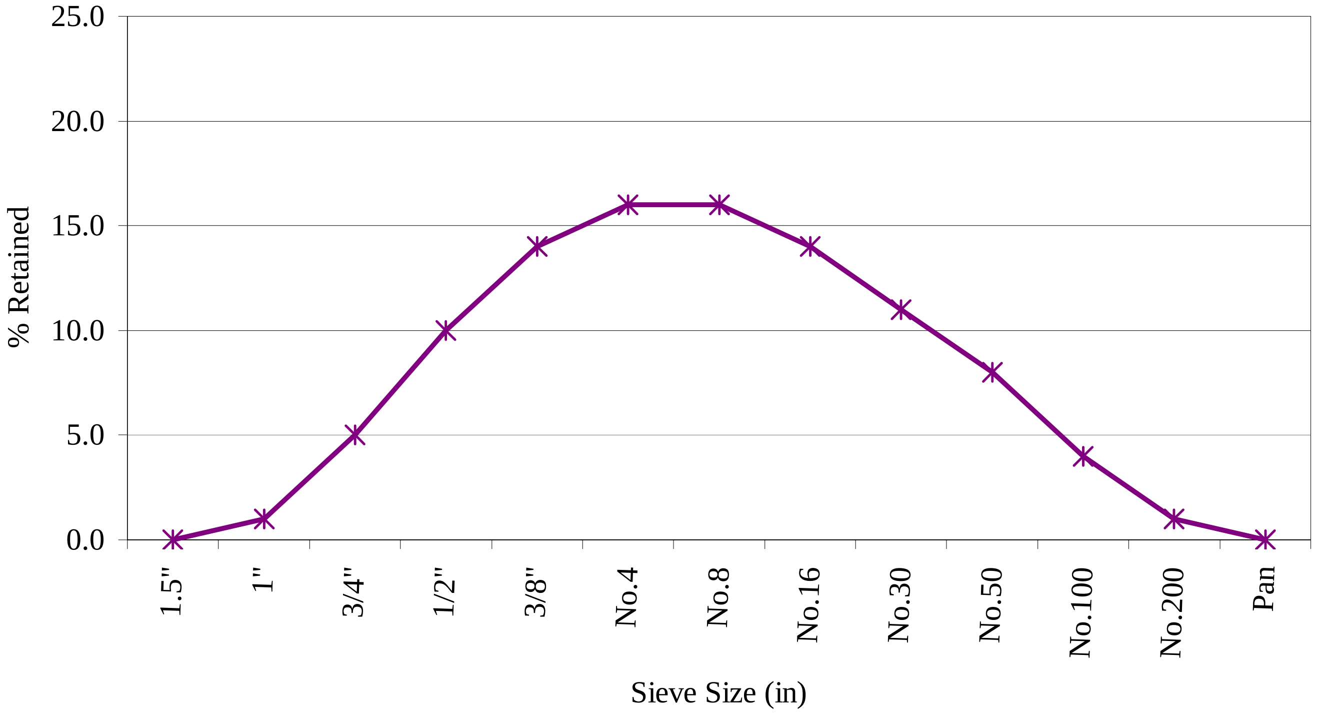

Figure 4.2 An Ideal Haystack Gradation Chart for Uniformly Graded Mixes.

Table 4.4 Fresh Concrete Properties for all HPC Mixes.

Mix ID	Slump (in)	Air Content %	Unit Weight (lb/ft^3)	Temperature (^{o}F)
1F1	6.00	4.50	150	77
1F2	3.50	5.00	150	77
1S1	2.25	4.00	152	82
1S2	2.50	4.50	150	77
2F1	5.75	5.00	152	70
2F2	4.00	5.75	146	64
2S1	2.00	5.30	150	73
2S2	3.75	5.25	150	77

entraining admixture required to obtain the desired air content was comparable for all the mixes.

The HRWRA dosage in the Group 1 mixes was almost double (120 oz/yd^3) that used in the Group 2 mixes (61 oz/yd^3). Since the two groups differ only in their gradation, it appears that the dense gradation mixes can achieve the required workability without the need for extra HRWRA.

4.3 Compressive Strength Results

Table 4.5 presents the average compressive strength values for all of eight HPC mixes at 7 and 28 days of age. When each group was examined separately, as shown in Figures 4.3 and 4.4, the mixes that contained slag as a cement replacement material generally had higher compressive strength at 28 days than the mixes that contained fly-ash. Also, the mixes that contained silica fume had higher compressive strength than those without silica fume. For example, for the Group 1 mixes, mix 1S1 (35% slag) had a compressive strength of 7398 psi, while mix 1F1 (25% fly ash) had a lower compressive strength of 6416 psi. Also within the same group, mixes 1F2 (20% fly ash

Table 4.5 Summary of Average Compressive Strengths for HPC Mixes at 7 and 28 Days of Age.

Mix I.D.	7-Day Compressive strength	28-Day Compressive Strength
1F1	5187	6416
1F2	4999	7193
1S1	6107	7398
1S2	6508	7806
2F1	5187	6157
2F2	5411	6515
2S1	5812	6393
2S2	6048	6864

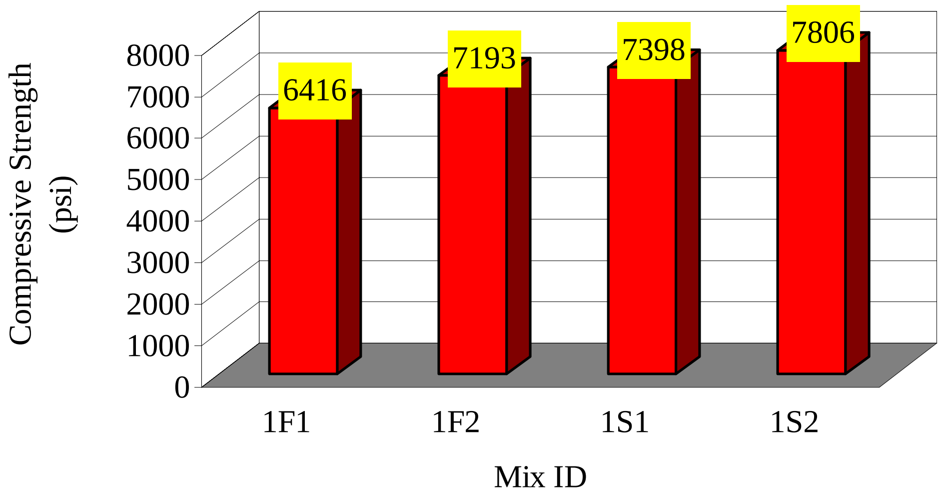

Figure 4.3: Compressive Strength for Group 1 Mixes.

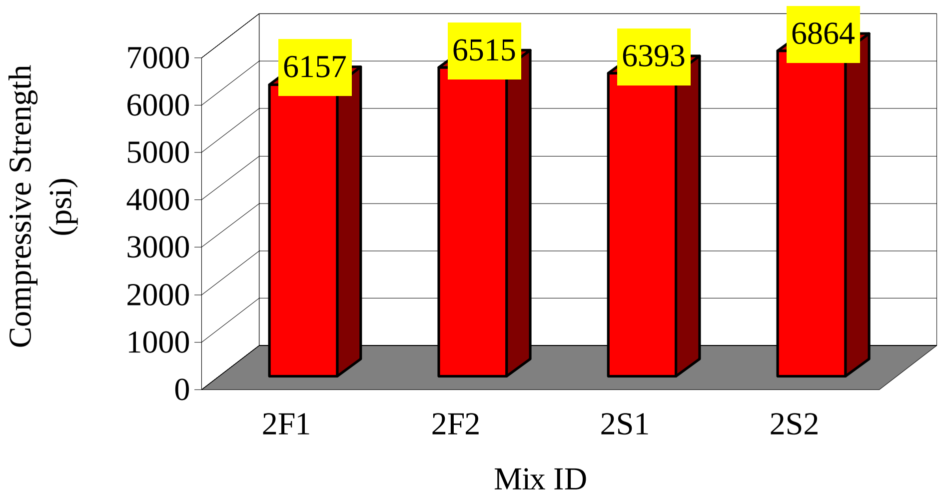

Figure 4.4: Compressive Strength for Group 2 Mixes.

and 5% silica fume) and 1S2 (35% slag and 5% silica fume) had higher compressive strengths of 7193 psi and 7806 psi, respectively, than mixes 1F1 and 1S1 (6416 psi and 7398 psi), respectively.

When comparing the two group mixes, as shown in Figure 4.5, the Group 2 mixes had consistently lower compressive strengths than the Group 1 mixes, which indicates that the dense-graded aggregate characteristics of the Group 2 mixes did not improve their compressive strength. However, it should be mentioned that the Group 1 mixes contained a higher percentage of coarse aggregate, 60%, compared with the 55% used in the Group 2 mixes. This may explain why the dense-graded aggregate nature of Group 2 failed to surpass the compressive strength of the Group 1 mixes.

The compressive strength of all of the mixes substantially exceeded the TDOT-specified compressive strength of 4000 psi at 28 days even though there were significant reductions in the cement content from the original Class D mix. The compressive strength of all eight mixes ranged from 6157 psi to 7806 psi with an average of 6843 psi. Reducing the cement content can also reduce the drying shrinkage.

4.4 Freeze and Thaw Durability Results

The freezing and thawing test was performed according to the standard method ASTM C666, Resistance of Concrete to rapid freezing and thawing, and AASHTO T 161. Appendix B gives detailed test data for mix 2F1.

The dynamic modulus of elasticity is generally used to determine the frost resistance of concrete because it is non-destructive to concrete specimens. The variation in the value of the modulus over the entire duration of freezing and thawing cycles provides a good indication of the variation in the strength of the concrete specimen.

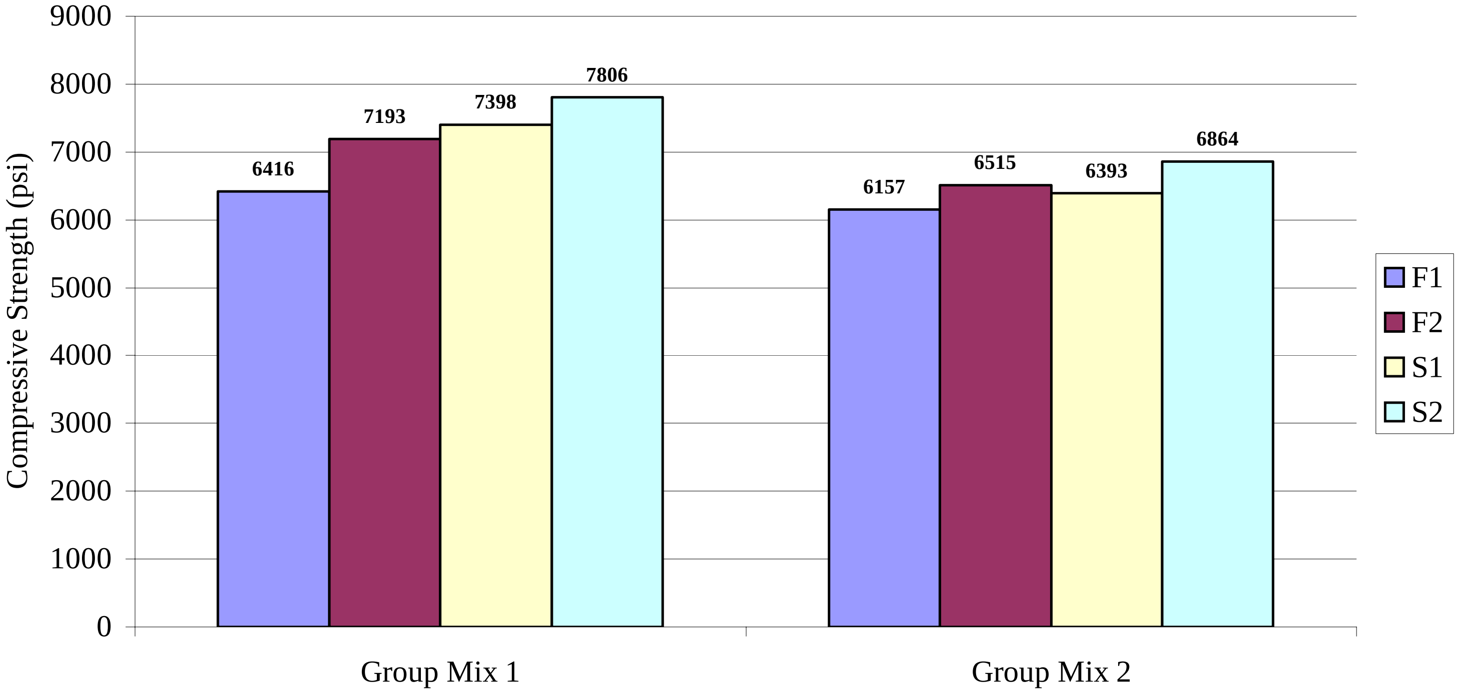

Figure 4.5: Summary of Compressive Strength at 28-Day.

The strength variation, in turn, reflects the degree of the freeze-thaw resistance. The relative dynamic modulus of elasticity (RDM) was calculated as follows:

$$\text{RDM} = \left(\frac{n_c^{\,2}}{n_o^{\,2}}\right) \times 100 \qquad (1)$$

Where:

RDM = relative dynamic modulus of elasticity after c cycles of freezing and thawing, percent.

c = number of cycles at time of testing.

n_c = fundamental transverse frequency after c cycles of freezing and thawing, in Hertz.

n_o = fundamental transverse frequency at zero cycles, in Hertz.

The RDM values for the mixes are shown in Figures 4.6 and 4.7. As shown in the figures, all the mixes survived more than 300 cycles of freezing and thawing without showing any significant loss in RDM. The TDOT recommendation is that the RDM value be greater than 80%.

The weight change of the concrete specimens over the freezing and thawing cycles is another indication of the deterioration of the concrete. Weight change can provide an idea of the amount of moisture absorbed due to the cracking of the specimen caused by the expansion of the cement paste. However, a significant amount of scaling was observed during testing, which could give a false reading of the weight change. For example, a specimen could have absorbed 0.1 lbs of water during a test cycle. However, the specimen could have scaled off 0.2 lbs of concrete. This leaves the tester thinking the specimen lost 0.1 lbs of water (+0.1 - 0.2 = -0.1).

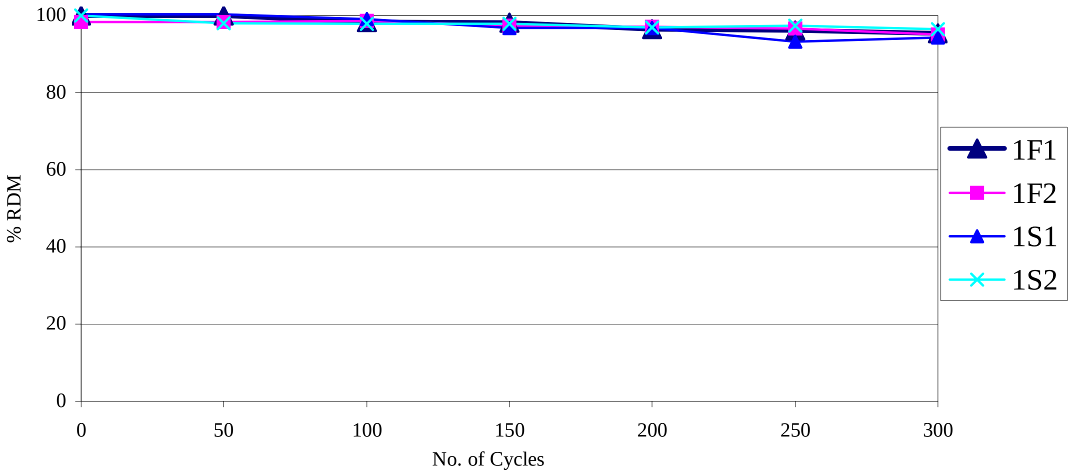

Figure 4.6: Relative Dynamic Modulus of Elasticity for Group 1 Mixes.

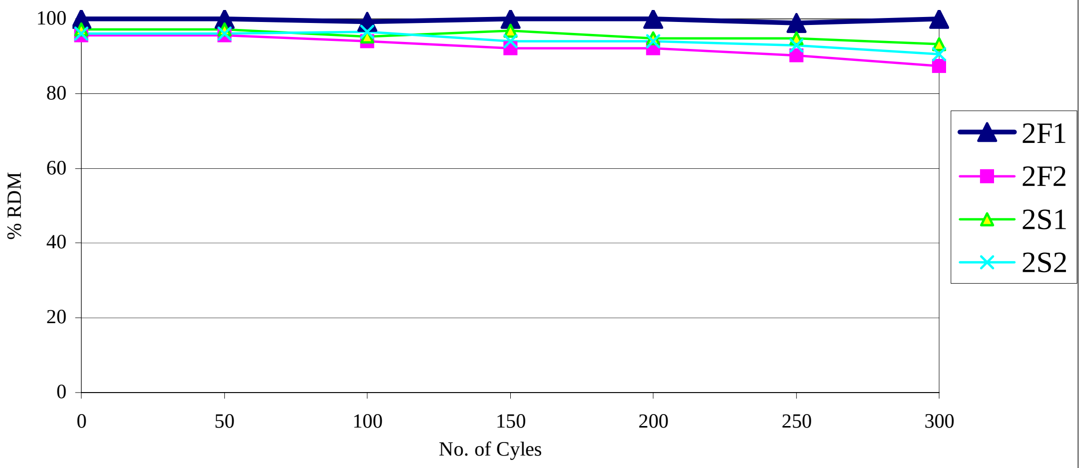

Figure 4.7: Relative Dynamic Modulus of Elasticity for Group 2 Mixes.

Water is not the only variable in the calculation of weight change; therefore, the calculation of weight change is not a sufficient measure of freeze-thaw durability.

The durability factor (DF) was determined as follows:

$$\mathrm{DF} = \left(\frac{RDM_{\max}}{RDM_{final}}\right) \times 100 \qquad (2)$$

Where:

DF = durability factor of test specimen.

$\mathrm{RDM}_{\mathrm{max}}$ = the maximum value of relative dynamic modulus of elasticity measured, percent.

$\mathrm{RDM}_{\mathrm{final}}$ = the value of relative dynamic modulus of elasticity at which exposure is terminated.

The DFs for the mixes can be seen in Figure 4.8. As shown in the figure, all mixes behaved extremely well with mix 2F1 having the highest frost resistance. The lowest DF is 87 (mix 2F2) which is still well above 80, which is required by TDOT.

4.5 Drying Shrinkage Results

The free drying shrinkage of concrete specimens was performed according to the standard test method ASTM C157, Length Change of Hardened Hydraulic-Cement Mortar and Concrete, and AASHTO T 160. After 7 days of moist curing in lime water at 73 ± 3 °F, the length change was measured at 1, 2, 3, 4, 5, 6, 7, 14, 21, 28, 56, 112 and 224 days. Drying shrinkage was measured at 40% ± 4% relative humidity and 70 ± 5 °F. The 7-days of moist curing was chosen to simulate the field condition of class D concrete in bridge decks, where the recommended minimum curing time is 7 days.

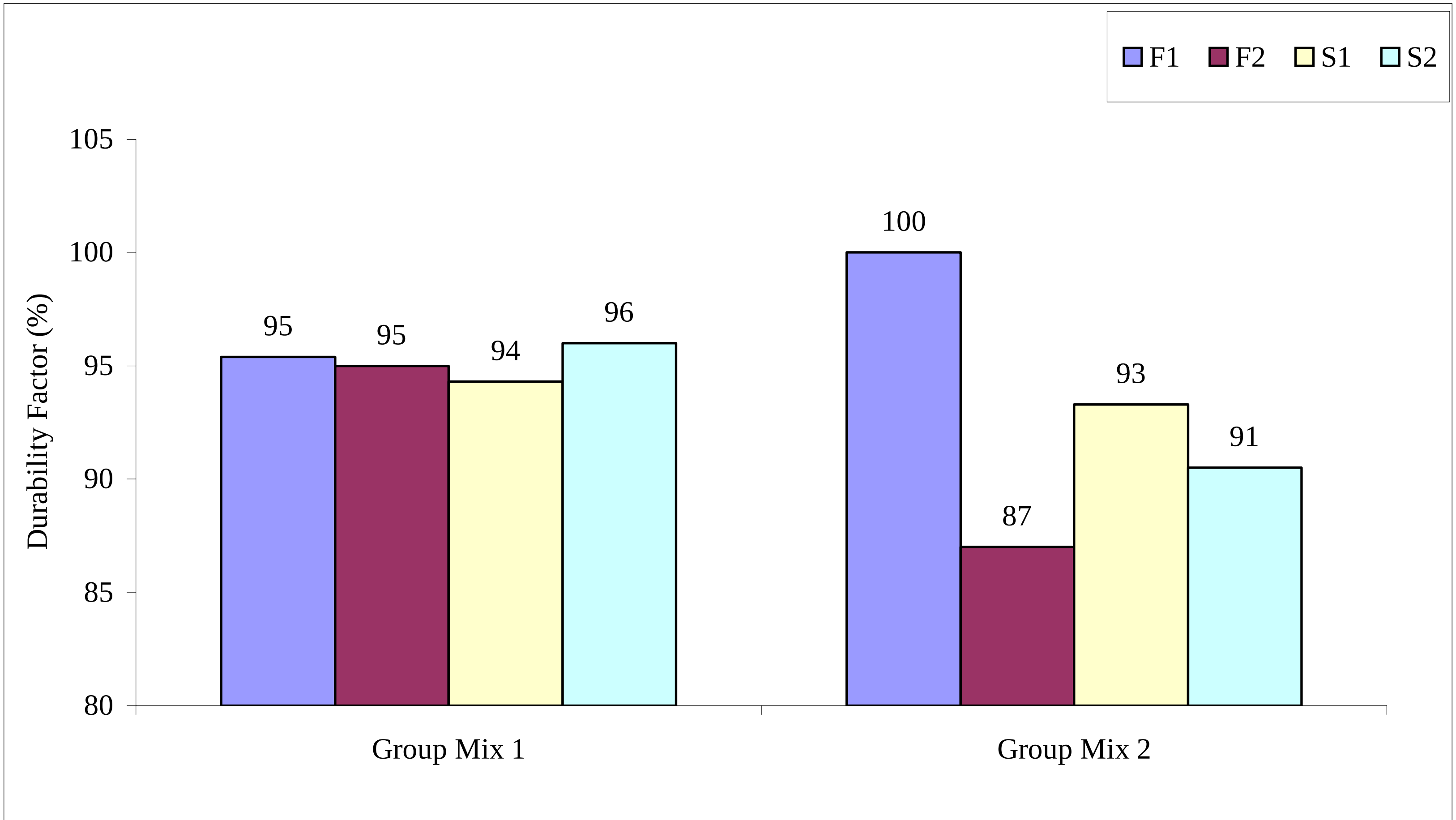

Figure 4.8: Summary of Freeze-Thaw Durability Factor.

The data for each mix type are shown in Figures 4.9 and 4.10. The figures show that drying shrinkage increased rapidly up to 14 days, and then the rate of increase reduced. After about 21 days, the rate of increase became quite low, with almost a flat slope. A logarithmic trend-line has been fitted for each set of data to show the general development of shrinkage strain as shown in Figures 4.11 and 4.12. Another trend-line is shown in the figures illustrating the predicted value for shrinkage using the standard shrinkage equation listed in ACI Publication 209R-92. The form of this equation is (moist cured concrete)

$\xi_{sh,t} = \{t/(t+35)\}* \xi_{sh,u}$

$\xi_{sh,t}$ = Shrinkage strain at age t, days.

t = Age of the concrete in days

$\xi_{sh,u}$ = Ultimate shrinkage strain as t goes to infinity.

The publication recommends an ultimate shrinkage strain value of 780 $\mu\xi$ be used if no better information is available. Figure 4.13 gives a better view of the final shrinkage. It shows the shrinkage values at 112 days.

4.6 Chloride Ion Permeability Results

The durability of the concrete in bridge decks has a major impact on maintenance costs. One of the most severe problems in bridge decks is the corrosion of their reinforcing steel, which is aggravated by chloride ions. Therefore, preventing the penetration of these ions into the concrete deck is critical to the overall durability of the deck. One of the major tasks of this project was to develop HPC mixture proportions that allow minimal penetration of chloride ions so that the corrosion potential of the reinforcing steel could be reduced. This task was accomplished by comparing the

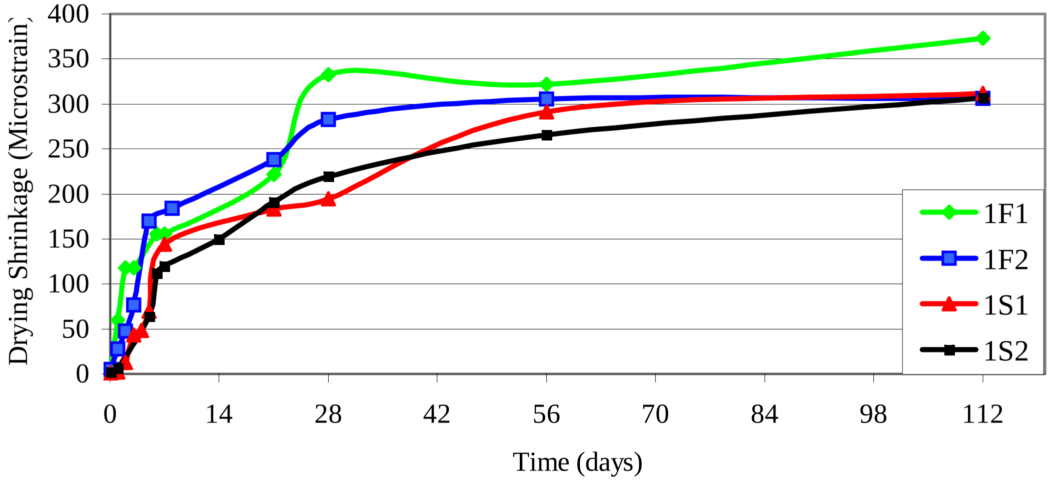

Figure 4.9: Drying Shrinkage @ 112 Days for Group1 Mixes.

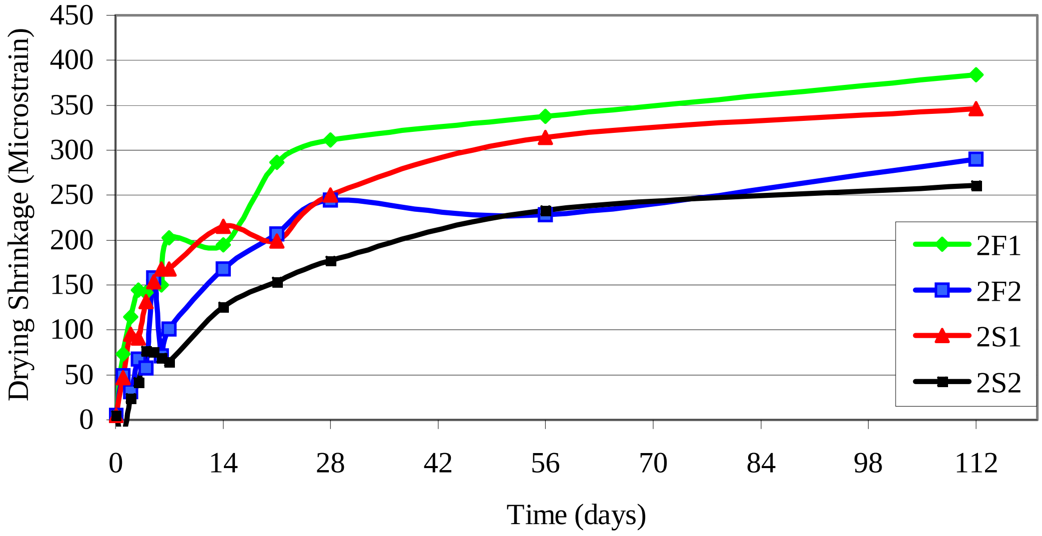

Figure 4.10: Drying Shrinkage @ 112 Days for Group2 Mixes.

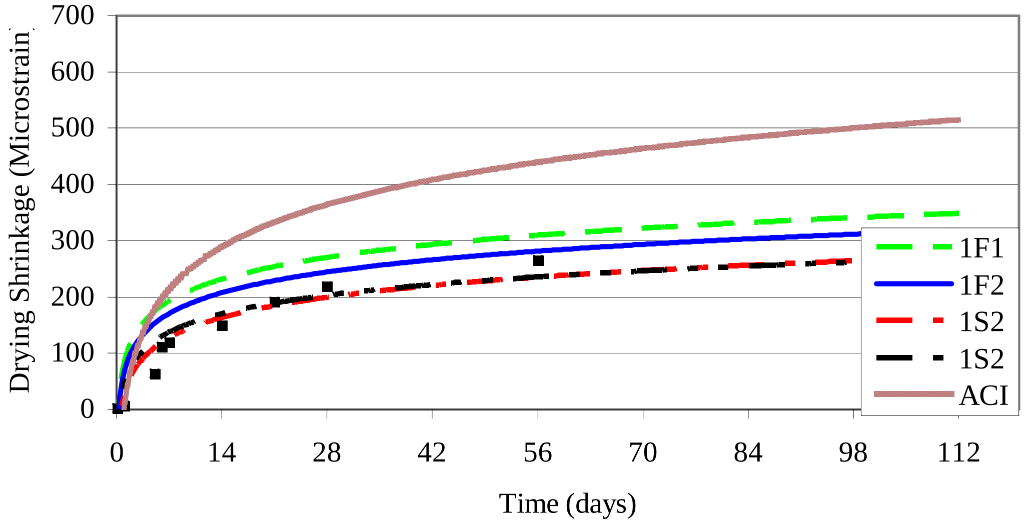

Figure 4.11: Best-Fit relationships of Drying Shrinkage for Group1 Mixes.

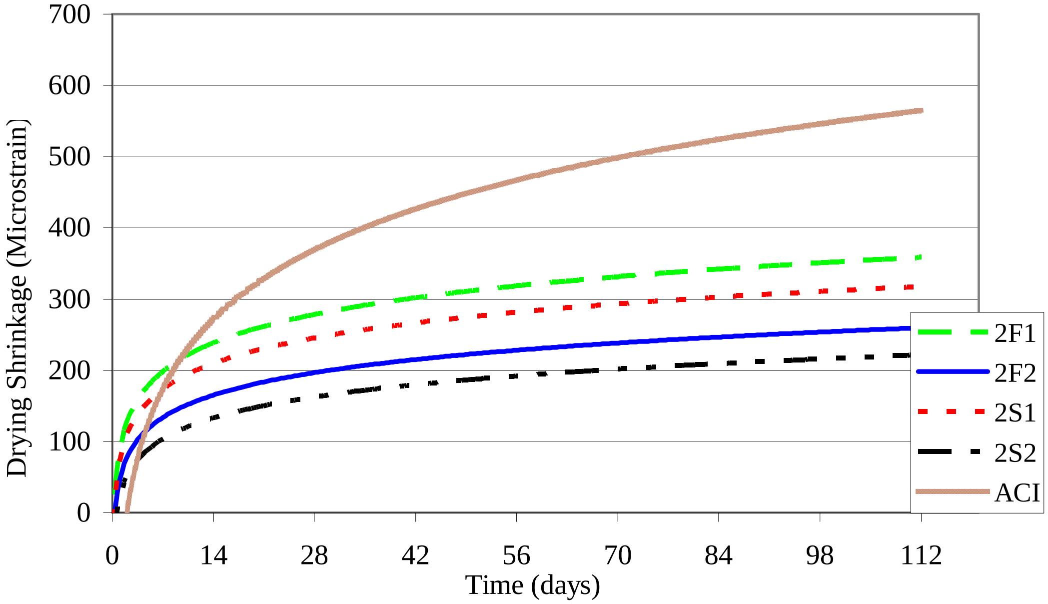

Figure 4.12: Best-Fit relationships of Drying Shrinkage for Group 2 Mixes.

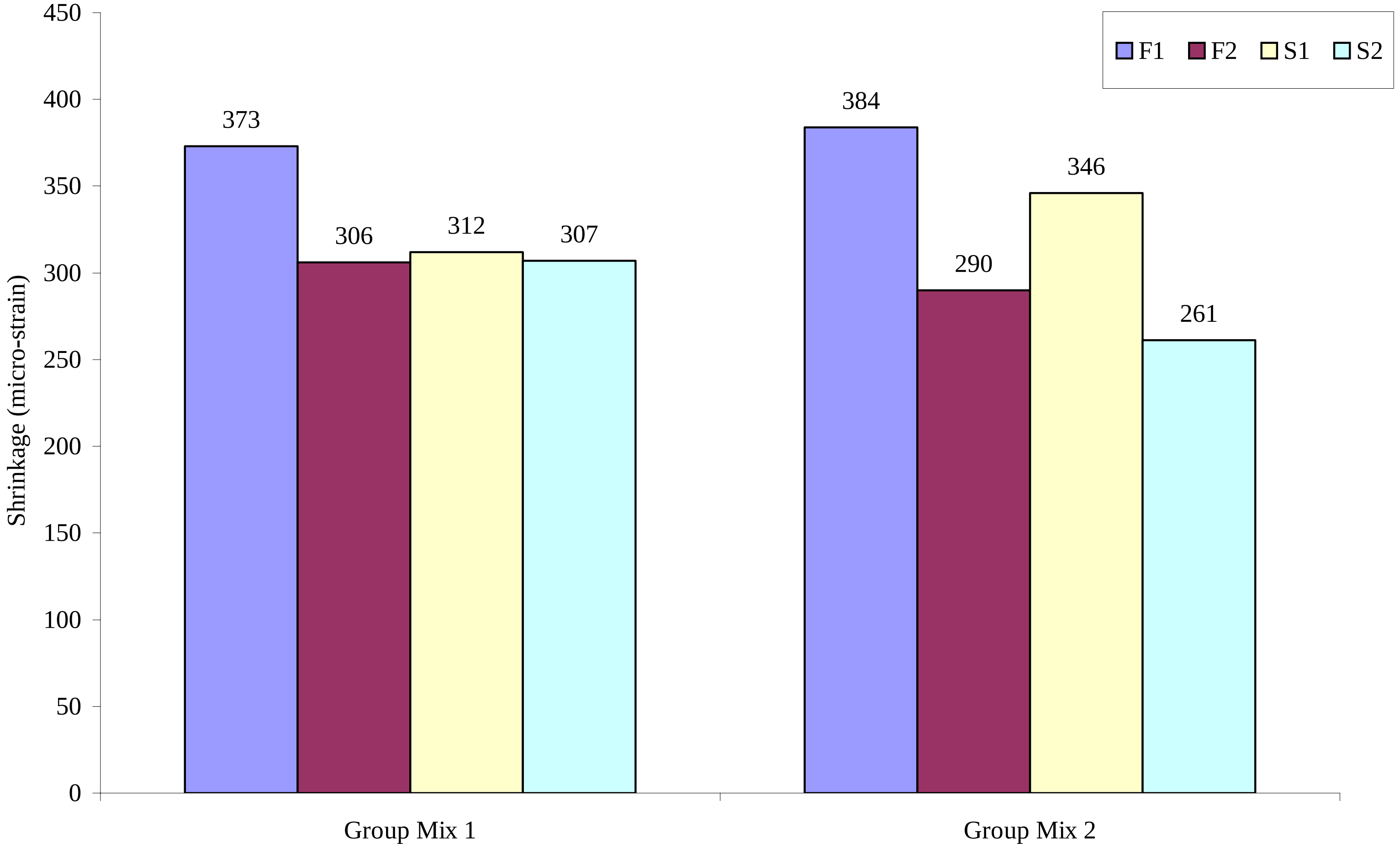

Figure 4.13: Summary of Drying Shrinkage at 112 Days of Age.

permeability of the various HPC mixes developed in this study to the permeability of TDOT’s standard bridge deck mix.

The permeability test results for the HPC mixes are shown in Figures 4.14 through 4.18. As shown in the figures, the Coulomb charge passing the specimen follows a general linear pattern with time, indicating that the final chloride permeability value specified at the end of the six-hour period can be reasonably estimated from the first 30-minute period, resulting in a significant reduction in test time.

According to ASTM C 1202, the permeability of concrete mixes is ranked as follows: 100-1,000 Coulombs, very low permeability; 1,000-2,000 Coulombs, low permeability; 2,000-4,000 Coulombs, medium permeability; and above 4,000 Coulombs, high permeability. As shown in Figure 4.14, among the mixes in Group 1, mix 1S2 had the lowest amount of charge (289 Coulombs) passed over a 6-hour period followed by mix 1F2 (401 Coulombs), indicating that the addition of silica fume to these mixes resulted in significant reductions in chloride ion permeability. However, mixes 1F1 and 1S1 achieved “low” chloride permeability levels (1470 and 1150 Coulomb, respectively), indicating that the addition of slag or fly ash alone to the currently gap-graded mixes used by TDOT would not be adequate to achieve high resistance to chloride penetration. Therefore, the addition of silica fume to HPC mixes can be crucial for achieving “very low” chloride ion penetration in concrete bridge decks where gap-graded aggregate is typically used. Thus, any TDOT-suggested specifications for HPC mixes should encourage or require the use of silica fume as a cement replacement for the gap-graded mixes. Based on the results of this research, a 5% silica fume replacement of cement by weight would be adequate.

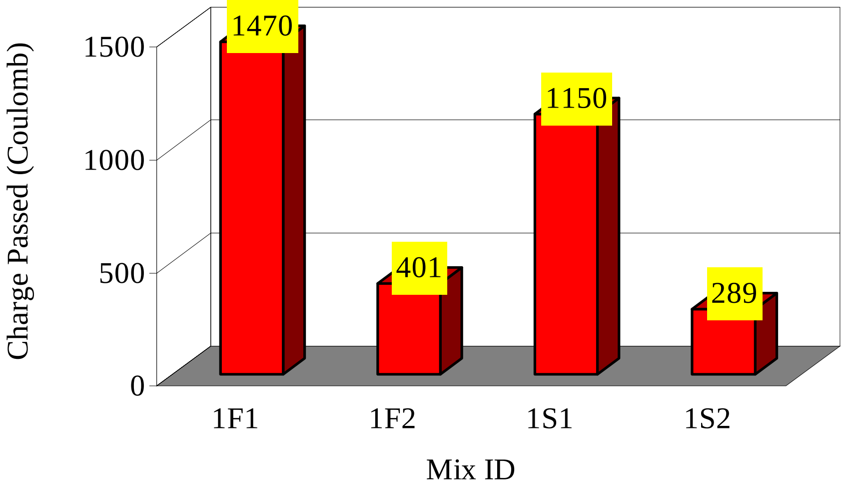

Figure 4.14: Chloride Ion Permeability for Group 1 Mixes at 56 Days.

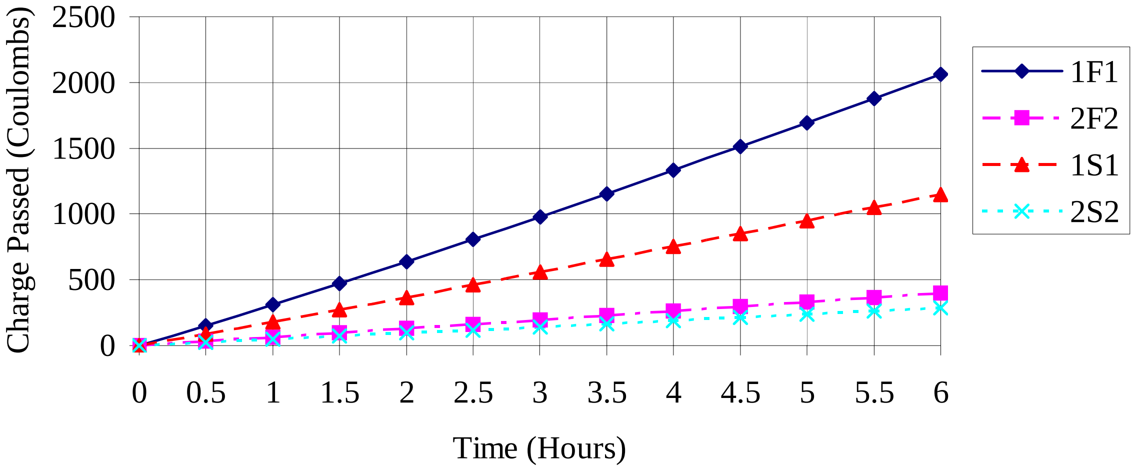

Figure 4.15: Chloride Ion Permeability for Group 1 Mixes.

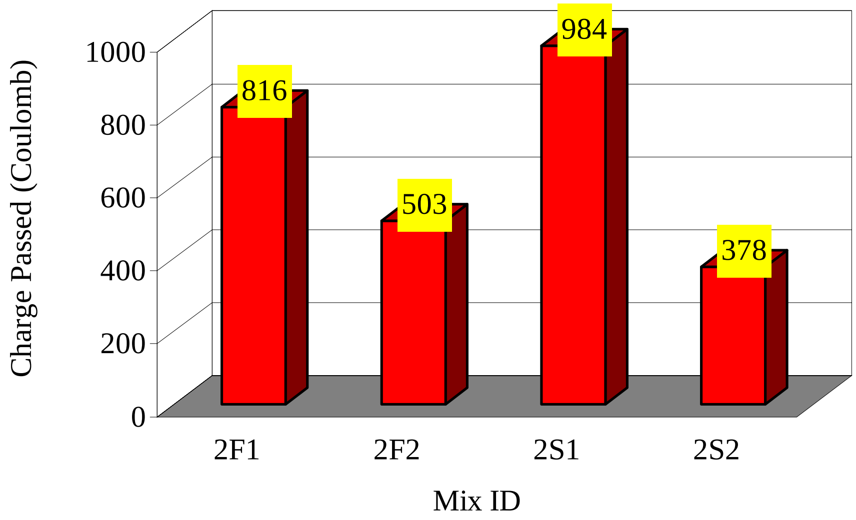

Figure 4.16: Chloride Ion Permeability for Group 2 Mixes at 56 Days.

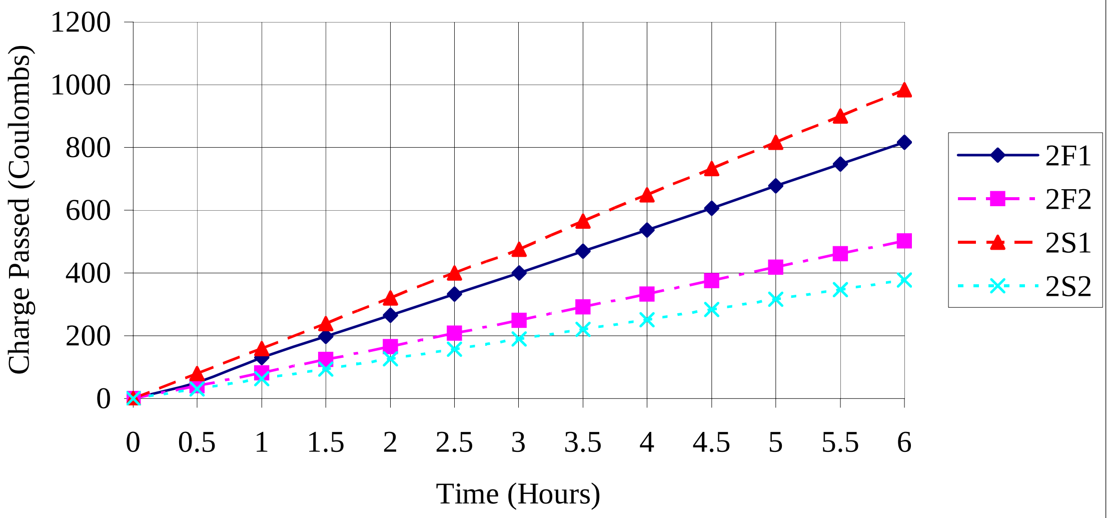

Figure 4.17: Chloride Ion Permeability for Group 2 Mixes.

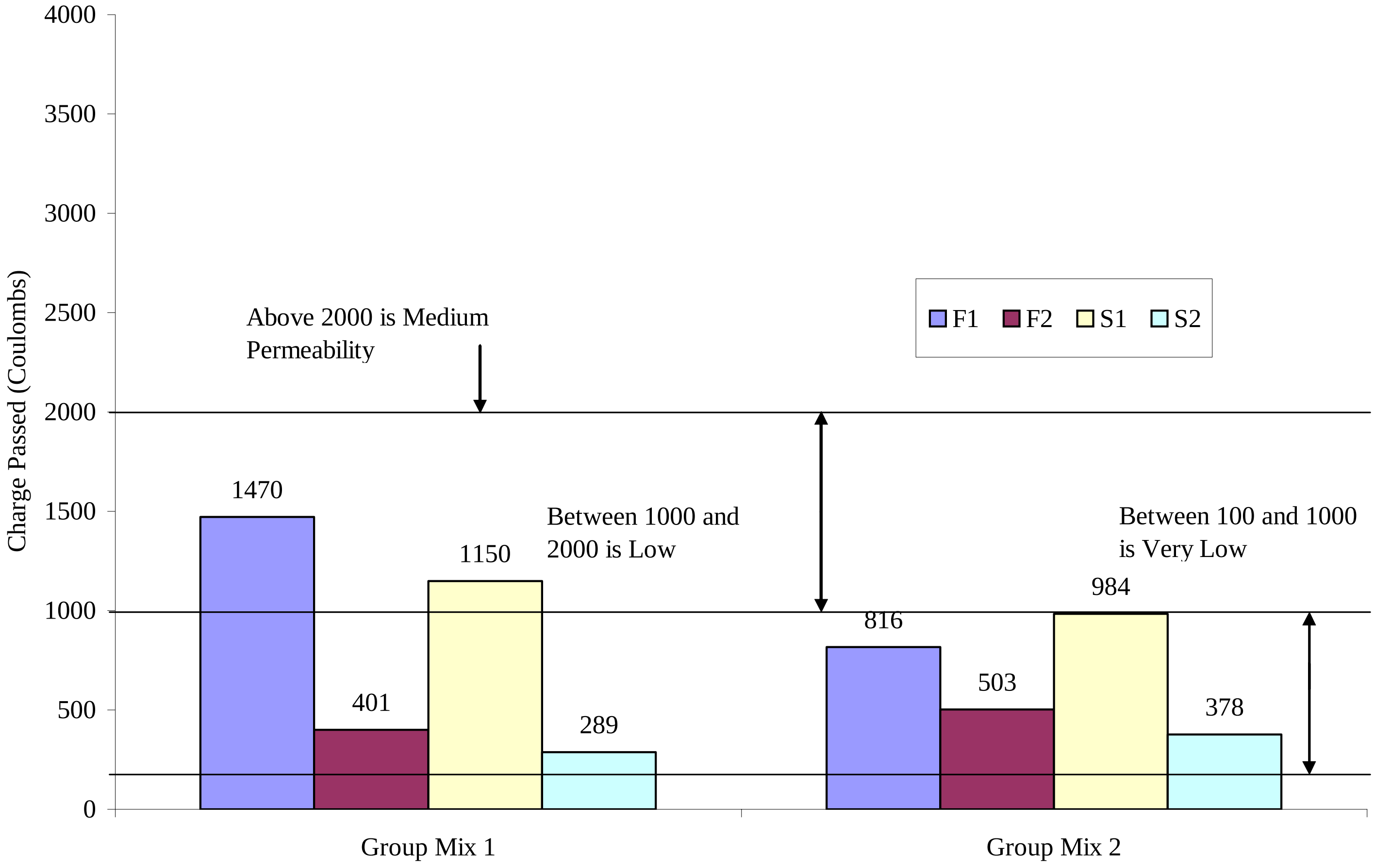

Figure 4.18: Summary of Chloride Ion Permeability.

The same trend was also observed for the dense-graded mixes in Group 2. However, the mixes that contained fly ash or slag alone without the inclusion of silica fume (2F1 and 2S1), had “very low” permeability values, while mixes 1F1 and 1S1 in Group 1 had “low” permeability levels. The achievement of very low permeability (below 1000 Coulomb) in these dense-graded mixes, without the need to include silica fume, was most likely caused by their dense aggregate gradation, which provided less permeability overall.

Even though the compressive strength of the Group 2 mixes was lower than that of the Group 1 mixes, the Group 2 mixes achieved comparable permeability to the Group 1 mixes. This goes against the general belief that increasing the strength of concrete always increases its long-term durability. Strength is related to the total pore volume whereas permeability is related to the distribution of the capillary pores in the cement paste. Therefore, it is possible to achieve very low permeability in normal-strength HPC by specifying mineral admixtures in the mix proportions.

Like other standard highway specifications, current TDOT specifications, as mentioned before, require at least a minimum cementitious content and specify a maximum w/cm ratio for Class D bridge decks. These specification requirements traditionally may have been prescribed to ensure adequate workability, finishibility, strength, and durability. However, with the advent of many mineral admixtures and cementitious replacement materials, the current prescriptive method of a minimum cementitious content and a maximum w/cm ratio may no longer be the most effective approach for addressing the durability performance of concrete. For example, the chloride ion permeability for standard Class D bridge deck concrete was found to be 4660

Coulombs, which is in the “high” permeability range. However, in this study, “very low” permeability was achieved by including mineral admixtures (slag, fly ash, and silica fume) while significantly reducing the cementitious content. Thus, it may be suggested that performance-related specifications, in which limits on chloride ion penetration, durability ratio, and maximum shrinkage are specified, would be more effective than the current prescriptive-related specifications in addressing the durability of HPC mixes.

4.7 Summary of Results

The selection of an optimum HPC mix should be based on the performance, practicality, and potential economical savings. Most of the evaluated mixes had comparable performance properties related to strength, freeze-thaw durability, drying shrinkage, and chloride ion permeability. Specifically, all the mixes had compressive strength well above 6000 psi at 28-days, freeze-thaw durability factor above 80 percent after 300 cycles of freezing and thawing, drying shrinkage between 300 and 400 micro-strain at 224 days, and, except for Mix 1F1 and 1S1, chloride ion permeability values below 1000 coulombs at 56 days. Therefore, most of the mixes can be classified as acceptable HPC mixes as shown in Table 4.6.

Based on the practicality criteria, the dense-graded mixes of Group 2 had three combined aggregate sizes. Modifying the gap-graded mixes to dense-graded mixes by adding only one aggregate is not considered to be too costly to rule out. The addition of one more aggregate bin to a plant is actually quite easy to accomplish. The selection of mixes based on practicality is shown in Table 4.7.

Based on the economical savings criteria as shown in Table 4.8, the mixes that included slag as cement replacement would cost more than the mixes that included fly

Table 4.6 Selection of Optimal HPC Mixes Based on Performance.

Performance Properties	1F1	1F2	1S1	1S2	2F1	2F2	2S1	2S2
Compressive Strength (psi)	√	√	√	√	√	√	√	√
Freeze-Thaw Durability (%)	√	√	√	√	√	√	√	√
Drying Shrinkage (Microstrain)	√	√	√	√	√	√	√	√
Chloride Ion Permeability (Coulomb)		√		√	√	√	√	√
Final Selection		**√**		**√**	**√**	**√**	**√**	**√**

Table 4.7 Selection of Optimal HPC Mixes Based on Practicality.

	1F1	1F2	1S1	1S2	2F1	2F2	2S1	2S2
2 and 3-Size Combinations	√	√	√	√	√	√	√	√

Table 4.8 Selection of Optimal HPC Mixes Based on Potential savings.

	1F1	1F2	1S1	1S2	2F1	2F2	2S1	2S2
Fly Ash Mixes	√				√			
Slag Mixes								
Silica Fume Mixes		√						
Final Selection	**√**	**√**			**√**			

ash since the cost of slag is equal to the cost of cement while the cost of fly ash is about half the price of the cement. Therefore, fly ash mixes will be selected among the slag and fly ash mixes that had comparable performance properties. Mixes with silica fume will have the highest unit cost per cubic yard, since the cost of silica fume is almost seven times the cost of cement. Therefore, only gap-graded mixes with silica fume will be selected since the dense-graded mixes attained the required performance properties without the need of silica fume.

The final selections are shown in Table 4.9. Mixes 1F2 and 2F1 are the only mixes to meet each of the criteria set forth by TDOT. However, the use of silica fume, while beneficial, complicates the mixing process and increases the cost of the mix.

Table 4.9 Final Selection of Optimal HPC Mixes.

	1F1	1F2	1S1	1S2	2F1	2F2	2S1	2S2
Final Selection		√			√			

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

Based on the discussion of results, the following conclusions are drawn:

- Optimized mix designs that result in improved durability performance compared with current TDOT Standard Class D concrete can be developed through the utilization of fly ash and a densely graded aggregate. Specifically, adjusting the typical gap-graded mixes used by TDOT to dense-graded mixes will result in high-performance concrete mixes with very low permeability, adequate compressive strength, high frost resistance, and low drying shrinkage - all without the need for the addition of silica fume. These mix designs also result in cementitious cost savings of about 20% over the TDOT Class D mix.
- A performance-based specification system to specify durable HPC mixes may serve as an alternative to the existing prescriptive TDOT system. Based on the new system, the concrete producer should aim to optimize specific durability criteria in mix designs rather than just follow indirect prescriptive w/cm ratio requirements. In the new system, the use of mineral admixtures, such as fly ash, as cement replacement will also be a valuable option to the concrete producers because of potential cost savings.
- The dense-graded mixes achieved the same workability as Class D while using only about half the amount of HRWRA.

- The chloride ion permeability of the gap-graded mix was above the "very low permeability" level without the use of silica fume. The addition of silica fume to these mixes substantially lowered the permeability below that level. A 5% silica fume replacement of cement by weight is adequate for achieving a significant reduction in concrete chloride ion penetrations.
- The drying shrinkage was more significant during the early drying period. About 65% to 75% of drying shrinkage occurred within the first 21 days.
- Mixes containing silica fume have significantly higher compressive strength and lower chloride ion permeability than mixes containing slag or fly ash alone at the same w/cm ratio.
- Dense-graded mixes containing fly ash are better for deck use than gap-graded mixes containing silica fume because silica fume mixes require much more care during the placing and curing of the concrete. For this reason, silica fume concrete is likely to cost more than fly ash concrete.
- HPC mixes containing fly ash achieve an average permeability of 750 Coulombs, which is an 88.9% reduction of the permeability achieved by TDOT standard Class D concrete mixes (4660 Coulombs) are shown in Figure 4.18.

5.2 Recommendations

As a result of this study, mix 2F1 is recommended over 1F2 because of the time and expense caused by the use of silica fume. Table 5.1 gives the mix proportions of the recommended mix. However, a more optimum mix can be achieved. Therefore, further research is recommended. Further mixes should include other dense-graded aggregate combinations. Also, since the recommended mix surpassed all requirements set forth by

TDOT, the use of lower cement content is also suggested, which would lower economic costs even more.

Table 5.1 Recommended HPC Mix Proportions.

Mix I.D.		2F1
Cement (1b/yd^3)		427.5
Fly ash (1b/yd^3) (% of Cement)		142.5 (25)
Slag (1b/yd^3) (% of Cement)		---
Silica Fume (1b/yd^3) (% of Cement)		---
Water (1b/yd^3)		228
Combined Aggregates (1b/yd^3)	#57	1131
	#7	822
	NS	1204
W/C Ratio		0.40
MBT AE 90 (oz/ yd^3)		8.0
HRWRA (oz/ yd^3)		61.0

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APPENDICES

Appendix A: Physical and Chemical Properties of Cementing Materials

Table A-1: Material Composition of Type I Portland Cement
[Signal Mountain Cement Company].

Composition	Percent (mass)
Silicon Dioxide	20.8
Aluminum Dioxide	5.4
Ferric Oxide	3.9
Calcium Oxide	64.2
Magnesium Oxide	1.5
Sulfur Trioxide	3.0
Loss in Ignition	0.95
Insoluble Residue	0.22
Total	100.0

Table A-2: Chemical Properties of Class C Fly Ash.

	Element	**Typical Range of Concentration**
1	Silica (SiO_2)	41 - 58%
	Amorphous	42 - 53.5
	Crysalline	3.0 - 7.0
2	Alumina (Al_2O_3)	18.1 - 28.6%
3	Iron oxide (Fe_2O_3)	9.9 - 26%
4	Calcium oxide (CaO)	0.8 - 4.5%
5	Magnesium oxide (MgO)	0.7 - 1.4%
6	Sodium oxide (Na_2O)	0.2 - 0.6%
7	Potassium oxide (K_2O)	1.5 - 3.3%
8	Titanium dioxide (TiO_2)	1.0 - 1.9%
9	Sulfur trioxide (SO_3)	0.1 - 2.2%
10	Phosphorus pentoxide (P_2O_5)	nil - 1.5%
11	Loss on ignition	1.9 - 8.0%
12	pH	4.1 - 9.5

Table A-3: Chemical and Physical Properties of Slag Cement.

Slag Chemical Data			Slag Physical Data		
	Aucem	ASTM (max)		Aucem	ASTM (max)
Sulfide Sulfur	0.76%	2.50%	Blaine	5270	n/a
Sulfate (SO3)	1.27%	4.00%	#325 (Ret.)	1.04%	20.0
			Spec. Gravity	2.93	n/a
			Air Content	5.70%	12.0

Table A-4: Physical Data of Silica Fume.

Boiling Point (^{o}C)	N/Ap
Percent VOC (w/w)	0
Freezing Point (^{o}C)	N/Ap
Vapor Pressure mmHg @20 (^{o}C)	N/Av
Vapor Density	> Air
Odor Threshold	N/Av
Appearance: Gray granular powder	
Water/Oil Distribution Coefficient	N/Av
Solubility in Water	Slight
Bulk Density	30-40 lb/ft^3
pH	N/Ap
Evaporation Rate	N/Av
Odor: Odorless	

N/Av: Not Available

N/Ap: Not Applicable

Appendix B: Test Data of Selected HPC Mix

Table B-1: Group Mix 2 - 7-day Stress Statistical Results.

Mix ID	Cylinder	Load (kips)	Strength (psi)	Average Stress (psi)	Range (psi)
2F1	1	150	5305	5187	212
	2	146	5164		
	3	144	5093		
2F2	1	151	5341	5411	106
	2	154	5447		
	3	154	5447		
2S1	1	165	5836	5812	354
	2	159	5623		
	3	169	5977		
2S2	1	170	6013	6048	106
	2	173	6119		
	3	170	6013		

Table B-2: Group Mix 2 - 28-day Stress Statistical Results.

Mix ID	Cylinder	Load (kips)	Strength (psi)	Average Stress (psi)	Range (psi)
2F1	1	171	6048	6166	212
	2	177	6260		
	3	175	6189		
2F2	1	187	6614	6519	424
	2	189	6685		
	3	177	6260		
2S1	1	181	6402	6402	0
	2	181	6402		
	3	181	6402		
2S2	1	194	6861	6873	106
	2	196	6932		
	3	193	6826		

Table B-3: Fundamental Frequency for Mix 2F1.

Fundamental Frequency (hz)	
Cycles	Specimen 1
0	2450
50	2468
100	2441
150	2466
200	2478
250	2437
300	2455

Table B-4: Relative Dynamic Modulus for Mix 2F1.

RDM, %		
Cycles	Specimen 1	Average
0	100.16	100
50	100.16	100
100	97.98	98
150	100	100
200	100.98	101
250	97.66	98
300	99.11	99

Table B-5: Specimen Weight for Mix 2F1.

Specimen Weight (lb)	
Cycles	Specimen 1
0	17.345
50	17.475
100	17.46
150	17.465
200	17.47
250	17.47
300	17.455

Table B-6: Weight Change Percentage for Mix 2F1.

Wt. Change Percentage		
Cycles	Specimen 1	Average
0	0	0
50	0.057	0.057
100	-0.029	-0.029
150	0	0
200	0.029	0.029
250	0.029	0.029
300	-0.057	-0.057

Table B-7: Summary of Relative Dynamic Modulus for Group 2 Mixes.

	Cycles	0	50	100	150	200	250	300
Mix								
2F1		100	100	99	100	100	99	100
2F2		96	96	94	92	92	90	87
2S1		97	97	95	97	95	95	93
2S2		96	96	97	94	94	93	91

Table B-8: Summary of Weight Change Percentage for Group 2 Mixes.

	Cycles	0	50	100	150	200	250	300
Mix								
2F1		0	0.749	0.663	0.692	0.721	0.721	0.634
2F2		0	0.505	0.389	0.331	0.360	0.259	0.101
2S1		0	-0.028	-0.056	-0.056	-0.098	-0.310	-0.563
2S2		0	-0.072	-0.143	-0.201	-0.459	-0.818	-1.148

Table B-9: Drying Shrinkage Data for Mix 2F1 – Specimen 1.

Shrinkage Measurements Specimen 1				
Time (days)	Reading 1	Reading 2	Reading 3	Avg.
0	8.74	8.68	8.69	8.703
1	8.58	8.58	8.57	8.577
2	8.56	8.54	8.5	8.533
3	8.52	8.51	8.36	8.463
4	8.52	8.53	8.39	8.480
5	8.47	8.48	8.44	8.463
6	8.5	8.5	8.44	8.480
7	8.43	8.41	8.36	8.400
14	8.42	8.45	8.39	8.420
21	8.28	8.31	8.22	8.270
28	8.23	8.28	8.22	8.243
56	8.19	8.25	8.16	8.200
112	8.11	8.18	8.08	8.123

Table B-10: Drying Shrinkage Data for Mix 2F1 – Specimen 2.

Shrinkage Measurements Specimen 2				
Time (days)	Reading 1	Reading 2	Reading 3	Avg.
0	8.68	8.65	8.7	8.677
1	8.56	8.57	8.62	8.583
2	8.48	8.5	8.53	8.503
3	8.47	8.48	8.5	8.483
4	8.45	8.48	8.5	8.477
5	8.44	8.45	8.46	8.450
6	8.43	8.44	8.48	8.450
7	8.36	8.37	8.39	8.373
14	8.36	8.4	8.37	8.377
21	8.24	8.26	8.25	8.250
28	8.19	8.21	8.21	8.203
56	8.15	8.19	8.16	8.167
112	8.06	8.18	8.08	8.107

Table B-11: Drying Shrinkage for Mix 2F1 - Specimen 1.

Shrinkage Calculations (Micro-Strain) - Specimen 1				
Time (days)	Reading 1	Reading 2	Reading 3	Average
0	0	0	0	0
1	107	67	80	84
2	120	93	127	113
3	147	113	220	160
4	147	100	200	149
5	180	133	167	160
6	160	120	167	149
7	207	180	220	202
14	213	153	200	189
21	307	247	313	289
28	340	267	313	307
56	367	287	354	336
112	420	334	407	387

Table B-12: Drying Shrinkage for Mix 2F1 - Specimen 2.

Shrinkage Calculations (Micro-Strain) - Specimen 2				
Time (days)	Reading 1	Reading 2	Reading 3	Average
0	0	0	0	0
1	80	53	53	62
2	133	100	113	116
3	140	113	133	129
4	153	113	133	133
5	160	133	160	151
6	167	140	147	151
7	213	187	207	202
14	213	167	220	200
21	293	260	300	285
28	327	293	327	316
56	354	307	360	340
112	414	313	414	380

Table B-13: Drying Shrinkage for Mix 2F1 – Average.

Shrinkage Calculations (Micro-Strain) - Average			
Time (days)	Specimen 1	Specimen 2	Average
0	0	0	0
1	84	62	73
2	113	116	115
3	160	129	145
4	149	133	141
5	160	151	156
6	149	151	150
7	202	202	202
14	189	200	195
21	289	285	287
28	307	316	311
56	336	340	338
112	387	380	384

Table B-14: Average Readings for Chloride Ion Penetration for Group 2 Mixes.

Time (min)	2F$_1$ (Coulombs)	2F$_2$ (Coulombs)	2S$_1$ (Coulombs)	2S$_2$ (Coulombs)
0	0	0	0	0
30	52	42	80	32
60	131	83	160	63
90	198	124	240	95
120	265	166	320	126
150	333	208	401	158
180	401	250	475	189
210	469	292	566	221
240	538	334	649	252
270	607	376	732	284
300	677	418	816	316
330	746	461	900	347
360	816	503	984	379

Table B-15: Readings for Chloride Ion Penetration for Group 2 Mixes.

2F$_1$			2F$_2$		2S$_1$		2S$_2$	
0	0	0	0	0	0	0	0	0
74	60	22	44	39	83	77	32	31
147	137	110	88	77	166	153	64	62
222	207	165	133	115	250	229	96	93
298	278	220	178	154	335	305	128	124
375	349	275	224	192	421	381	159	156
452	420	330	270	230	500	450	191	187
530	491	386	316	268	596	536	223	219
609	563	441	362	305	684	614	254	250
689	635	497	409	343	772	692	286	282
769	707	554	455	381	861	771	317	314
849	779	610	502	419	950	850	349	345
930	852	667	549	457	1039	929	380	377

VITA

Charles Herbert Hamblin, Jr. was born in Nashville, TN on May 20, 1981. He was raised in Joelton, TN. He graduated fifth in his class from Cheatham County Central High School in 1999. He started at the University of Tennessee, Knoxville in 1999 and received a B.S. in Civil Engineering in 2003 and an M.S. in Civil Engineering in 2004.