2001

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Exploratory Study on High Performance Concrete For Bridge Decks in West Virginia

Wenbo Zhang

Thesis

submitted to the

**College of Engineering and Mineral Resources
at West Virginia University
in partial fulfillment of the requirements
for the degree of**

**Master of Science
in
Civil Engineering**

**Julio F. Davalos, Ph.D., Chair
Indrajit Ray, Ph.D., Co-Chair
Karl E. Barth, Ph.D.**

**Department of Civil and Environmental Engineering
Morgantown, West Virginia**

2001

**Keyword: high performance concrete, experimental studies, compressive strength,
permeability, shrinkage, durability**

ABSTRACT

Exploratory Study on High Performance Concrete For Bridge Decks in West Virginia

Wenbo Zhang

This study is intended to provide guidelines for a future global effort on the development of HPC mixes in West Virginia using several types of available aggregates. The objective of the present study is to explore the performance of two types of mixtures using fly ash and slag, respectively, and the same limestone coarse aggregate, natural sand, and constant water/cementitious ratio. For each type, three mixtures were evaluated, with one of the three being defined according to the WVDOT-DOH specifications as a reference (20% fly ash and 30% slag by weight of total cementitious material), from which we varied the quantity of fly ash or slag by $\pm 5\%$ to defined the remaining two. Both fresh and hardened concrete properties were evaluated, and the parameters of interest were compressive strength, drying shrinkage, freeze-thaw durability and chloride permeability, which were not affected much by using higher fly ash or slag content. However, when using higher quantity of slag (35%), the chloride permeability was reduced by nearly 20%. Thus, in future studies, the mixture designs with higher slag and even fly ash content may provide better performance. The best mixture for each type will serve as a foundation for the future optimizations of HPC mix designs in WV.

ACKNOWLEDGEMENT

I would like to dedicate this work to my son Daniel and my husband Liyun whose love and whole-hearted support helped me to bring this work and my degree to completion. I would also like to give thanks to my father, mother and sister for encouraging me in pursuing my education. I love you all.

I would like to take this opportunity to express my gratitude to Dr. Julio F. Davalos, an excellent teacher and advisor, for giving me the opportunity to work on this interesting project, and for providing me with the guidance, support and resource necessary for accomplishing my research objectives.

My sincere appreciation also goes to Dr. Indrajit Ray, who served as a co-chair on my committee, for his tremendous support in doing this research, whose wise advice and valuable suggestions helped significantly improve the quality of my work. I would like to thank Dr. Karl E. Barth who served as member of my committee, and whose kindness helped me to overcome difficult circumstances during this study. Also I would like to thank Jennifer Morris, Vivian Luo, David Boyajian, and Haiyong Wu for their friendship and great help in doing lab-testing work.

I would like to acknowledge the financial support received from the West Virginia Department of Transportation, Division of Highways (WVDOT-DOH). This study is part of the WVDOT Research Project # 165. I would also like to acknowledge the support received from Arrow Concrete Company, and Master Builders Company, who provided all of the raw materials for this project.

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Chapter 1

Introduction

1.1 Research Background

Concrete is used in several types of structures including buildings, highways, bridges, and many others. It is estimated that more than 500 million tons of concrete is produced in the United States every year and several times this volume worldwide [Silver Spring].

Although concrete structures are thought of as being permanent, they are apparently not problem-free. Currently in the United States, about 33% of highway bridges are rated substandard [Better Roads Magazine, 1993] with major problems associated to concrete deterioration. Many concrete pavements and bridge decks failed before reaching their expected design life.

An estimate made in 1985 showed that over \$400 billion would be needed by the end of the 20th century for repair and rehabilitation of the nation's highways and bridges in the United States [Special Report202, 1987]. Another estimate made in 1987 indicated that between one and three trillion dollars would be needed to repair U.S. concrete structures over the next 20 years [NMAB-437, 1987].

However, many of the problems encountered with existing concrete materials can be overcome in future construction projects by using a new type of concrete termed High Performance Concrete (HPC). The advantages of HPC are increased strength and

durability and better overall performance characteristics [Moore, 1999], which in turn will result in increased safety and reduced life-cycle costs of structures.

Several states in United States have begun to use HPC in bridge construction. Four pioneering states are: Nebraska, New Hampshire, Texas, and Virginia. Their applications of HPC showed excellent results in strength, durability, and permeability.

In West Virginia, the Department of Highways (WVDOH) has developed specifications for class H high performance concrete as an initial reference to establish threshold mixture proportions. A large-scale and comprehensive 3-year study concerned with the development of HPC mixtures using locally available aggregate materials has also been initiated by the WVU College of Engineering and Mineral Resources research team listed at the end of this thesis. The purpose of this global research project using several aggregate materials available in the state is, to establish acceptable performance levels relevant to bridge deck concrete, and to develop specifications for various optimal HPC mixtures applicable to cast-in-place bridge decks in West Virginia. The most relevant performance requirements of HPC for bridge decks include strength, chloride permeability, and freeze-thaw durability.

The present study, which is part of the comprehensive program on HPC, was designed as an exploratory effort focusing on developing appropriate mixture proportions using limestone as one type of coarse aggregate abundant in West Virginia, natural sand, and both fly ash and slag.

1.2 Research Objective and Tasks

The present exploratory study serves to establish a foundation for the future optimizations of HPC mix designs by providing guidelines for the most likely mix proportions to be developed using several WV aggregates. Through this exploratory study, the future work will be defined more effectively, and the complexity of the global research will be minimized. At the same time, and as part of the present study, an Advisory Panel for this project will be established, based on the exploratory research results that are to be presented in this thesis.

The present study was organized into the following three tasks:

1. Development of mixture proportions;
2. Evaluation of performance of mixtures;
3. Analysis of test results and selection of optimum mixes.

1.3 Research Plan

A total of six different mixtures were prepared using Type I portland cement. Three of them are using Class F fly ash to replace cement in the amounts of 15, 20, and 25 percent by weight; the other three are defined using blast furnace slag to replace cement in the amounts of 25, 30, and 35 percent by weight. One type of coarse aggregate consisting of $\frac{3}{4}$ inch graded limestone was used. Air entrainment and high range water reducer agent (HRWRA) were also used. All the mixtures were prepared using a constant water-cementitious ratio of 0.4 by weight. Each mixture was tested in the

freshly mixed stage and hardened stage. In the fresh stage, the following tests were conducted: slump, air content, unit weight, and temperature. In the hardened stage, tests were conducted for compressive strength, rapid chloride permeability, shrinkage, and freeze-thaw cycles. The data from these tests was used to evaluate the performance of different mixtures and to establish the most optimum mixtures.

1.4 Thesis Organization

A review of the literature is presented in Chapter 2, in which, fundamentals of HPC as well as current research results and applications for HPC in bridge decks are presented. Chapter 3 presents the experimental program, including materials, sample preparation, and testing programs. Chapter 4 presents the test results and discussions. Chapter 5 summarizes the results of the entire research project covered in this thesis, followed by conclusions and recommendations for future work.

Chapter 2

Literature Review

In this chapter, the definition of HPC by various agencies is discussed along with the information provided by the four lead states: Nebraska, New Hampshire, Texas, and Virginia. These state DOT's contributed significant research results on HPC. The reports on studies of HPC properties relevant to bridge applications are also reviewed, and HPC mixtures produced in few other states is also discussed. In addition the specifications by the West Virginia Department of Highways (WVDOH) for class H concrete (equivalent to HPC) that is currently in use is described in this chapter.

2.1 Definition of HPC for Highway Structures

The American Concrete Institute (ACI) [Russell, 1999] gives a definition of high-performance concrete (HPC) as follows: *HPC is a type of concrete that meets special combinations of performance and uniform requirements that cannot always be achieved routinely using conventional constituents and normal mixing, placing, and practices.*

According to ACI, the following criteria must be satisfied to make a HPC:

- ❑ Ease of placement
- ❑ Compaction without segregation
- ❑ Early age strength

- ❑ Long-term mechanical properties
- ❑ Low permeability
- ❑ Density
- ❑ Heat of hydration
- ❑ Toughness
- ❑ Volume stability
- ❑ Long life in severe environment

The Strategic Highway Research Program (SHRP) [Zia, 1996] gives another definition for high-performance concrete using the following criteria:

- ❑ Maximum water-cementitious ratio (w/c) of 0.35
- ❑ Minimum durability factor of 80% as specified by ASTM C 666 Procedure A
- ❑ Minimum strength criteria of either
 - (1) 21 MPa (3,000 psi) within 4 hours after placement, which corresponds to very early strength (VES)
 - (2) 34 MPa (5,000 psi) within 24 hours (high early strength or HES), or
 - (3) 69 MPa (10,000 psi) within 28 days (very high strength or VHS)

It is noted that in the definition of HPC, the ACI cites fresh concrete properties, while the SHRP uses w/c as a mixing proportion criterion, but both cite long-term performance parameters. For bridge engineers to adopt a HPC performance definition, it must include adequate durability and strength parameters [Mather].

Goodspeed et al, 1996 proposed a definition of HPC for highway bridge applications. This definition is flexible and contains many variables, which allow designers the ability to choose different categories of performance. It consists of four

strength parameters and four durability parameters as listed in the Table 2.1. The specialized series of concrete are designed to provide several benefits in the construction of concrete structures as listed in the table 2.2. The " Grades" of performance, which are chosen based on the needs of the structure and design, are listed in the Table 2.3 and Table 2.4.

Table 2.1 Definition of HPC for Highway Bridge (Goodspeed et al, 1996)

STRENGTH CRITERIA	DURABILITY CRITERIA
Compressive Strength	Freeze-thaw
Modulus of Elasticity	Scaling
Shrinkage	Abrasion
Creep	Chloride Permeability

Table 2.2 Benefits of Using HPC in Highway Bridge Construction
(Goodspeed et al, 1996)

PERFORMANCE BENEFITS	COST & OTHER BENEFITS
Ease of placement and consolidation	Less material
Long-term mechanical properties	Fewer beams
Early high strength	Reduced maintenance
Toughness	Extended life cycle
Volume stability	Aesthetics
Longer life in severe environments	

Table 2.3 FHWA HPC Performance Grades in SI Unit (Goodspeed et al, 1996)

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 mm)	$2.0 > x \geq 1.0$	$1.0 > x \geq 0.5$	$0.5 > x$	
Chloride penetration ⁴ (x = coulombs)	$3000 \geq x > 2000$	$2000 \geq x > 800$	$800 \geq x$	
Strength ⁵ (MPa) (x = compressive strength)	$41 \leq x < 55$	$55 \leq x < 69$	$69 \leq x < 97$	$x > 97$
Elasticity ⁶ (GPa) (x = modulus of elasticity)	$28 \leq x < 40$	$40 \leq x < 50$	$x \geq 50$	
Shrinkage ⁷ (x = microstrain)	$800 > x \geq 600$	$600 > x \geq 400$	$400 > x$	
Creep ⁸ (per MPa) (x = microstrain/pressure unit)	$75 \geq x > 60$	$60 \geq x > 45$	$45 \geq x > 30$	$30 \geq x$

Table 2.4 FHWA HPC Performance Grades in US Customary Unit (Goodspeed et al, 1996)

PERFORMANCE CHARACTERISTIC	FHWA HPC PERFORMANCE GRADE			
	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)	$4 \leq x < 6 \times 10^6$	$6 \leq x < 7.5 \times 10^6$	$x \geq 7.5 \times 10^6$	
Shrinkage ⁷ (x = microstrain)	$800 > x \geq 600$	$600 > x \geq 400$	$400 > x$	
Creep ⁸ (per psi) (x = microstrain/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: (Blank cell indicating the grade not apply to the corresponding performance characteristic)

¹ Test in accordance to AASHTO T 161 (ASTM C 666 Procedure A)

² Test in accordance to ASTM C 672

³ Test in accordance to ASTM C944

⁴ Test in accordance to AASHTO T 277 (ASTM C 1202)

⁵ Test in accordance to AASHTO T2 (ASTM C 39)

⁶ Test in accordance to ASTM C 469

⁷ Test in accordance to ASTM C 157

⁸ Test in accordance to ASTM C 512

2.2 Research on High Performance Concrete Mixtures

There have been a number of studies focused on high performance concrete mixtures.

Ozyildirim 1988 studied the potential use of several supplemental cementitious materials to increase the resistance of hydraulic-cement concrete to penetration of chloride ions. The materials included Class F Fly ash, silica fume, ground granulated iron blast furnace slag, type I cement and type II cement, crushed granite gneiss, and siliceous sand.

The mixture proportions he studied were as follows:

- 1) 5% of the cement by mass was replaced with 1.2 times that mass by a Class F fly ash;
- 2) 25% of the cement by mass was replaced with 1.2 times that mass by a Class F fly ash;
- 3) 50% of the cement by mass was replaced with slag;
- 4) 7% of the cement by mass was replaced with silica fume.

Each combination of materials with type II cement was tested at w/c ratio of 0.35, 0.40, and 0.45. Each combination with type I cement was tested at a w/c ratio of 0.40. To investigate the effect of curing temperatures, specimens were prepared using control concrete as well as concrete with 15% fly ash, 50% slag, and 7% silica fume, and with type II and type III cements at a w/c ratio of 0.40. Three temperature levels of 40 °F (4 °C), 73 °F (23 °C) and 100 °F (38 °C), were considered.

All concrete contained an air entraining and a water-reducing admixture. The 2-inch specimens, for rapid chloride permeability test, were cut from the top of a 4-inch by 8-inch cylinders after two weeks of moist curing. The side surface of the specimens was coated with an epoxy resin to prevent lateral moist loss. They were set on a plastic sheet, and were kept in ambient laboratory conditions until the time of test. Compressive strength was determined at ages of 1, 7, 28, 90, and 365 days.

The following results were obtained:

- 1) *Chloride permeability*: The Coulomb value (denoted by Q) was reduced as the w/c ratio was decreased. Concrete with pozzolans and slag had lower Q values than the controls, and Q values were decreased as the age of the specimens was increased from 28 days to 90 days. The 365-day results were inconclusive as to whether changes in permeability occurred after 90 days.
- 2) *Strength*: Concrete with fly ash and slag had lower early strengths but generally higher ultimate strengths than the controls. For concrete containing silica fume, strengths were about the same or slightly higher at all ages.
- 3) *Relation of strength to chloride permeability*: For the same type and amount of supplemental cementitious materials, the Q values were decreased as strength was increased, indicating an inverse relationship between chloride permeability and strength. However, there was no specific relationship between the Q values and strength.
- 4) *Effects of curing temperature*: For those concretes containing slag and silica fume with type II and type III cements, there was little difference in chloride permeability performance for the same concrete at temperatures of 73 °F (23

°C), and 100 °F (38 °C), but for concrete cured at 40 °F (4 °C), the chloride permeability was generally higher. However, fly ash concrete made with either cement showed a very significant reduction in Q values as the curing temperature was increased. When the fly ash concrete was cured at 40 °F (4 °C) or 73 °F (23 °C), chloride permeability was in the high range, but when it was cured at 100 °F (38°C), the Q values were in or very close to the very low range. In general, the strengths of all concretes were highest when cured at 73 °F (23 °C).

Ozyildirim 1989 investigated whether the resistance to the penetration of chloride ions in the concrete with fly ash or slag could be improved by the addition of silica fume with either type II or type III cement. He also investigated whether early strengths of concrete with fly ash or slag could be improved by the addition of silica fume.

In his experimental program, 4 inch by 8 inch cylinders were made to test compressive strengths at 1, 3, 7, 28, and 90 days and chloride permeability at 28 and 90 days. For each concrete mixture, the total amount of cementitious material was 658 lb/yd³, and the w/c was 0.40. The nominal maximum size of coarse aggregate was ½ inch. Table 2.5 shows mixture proportions per cubic yard of concrete.

Table 2.5 Mixture Proportions per Cubic Yard of Concrete (unit: lb/yd³)

Identification	Cement	Water	Fly Ash	Slag	Silica Fume	Fine Aggregate	Coarse Aggregate
Control	658	263	-----	-----	-----	1432	1506
25F	494	263	164	-----	-----	1377	1506
50S	329	263	-----	329	-----	1413	1506
5SF	625	263	-----	-----	33	1422	1506
20F-5SF	494	263	132	-----	33	1377	1506
45S-5SF	329	263	-----	296	33	1404	1506

Note: 1.Type III cement used for all combinations. Additional specimens for control, 20F-5SF, and 45S-5SF was with Type II cement,

2. Numbers in the first column indicate the percentage of the supplemental material by mass in the total cementitious material, F= fly ash, S = slag, SF= silica fume.

In the chloride permeability study, specimens from the same batch of concrete were cured in two different ways. In the first curing procedure, specimens were moist cured for one day and then air-dried until tested. In the second curing procedure, specimens were moist cured for 14 days and then air-dried until tested. After the moist curing, the top 2 inch of the cylinders was cut off and used as the test specimens.

The following results were found in his study:

- 1) Lower permeability is attained by the addition of silica fume in the amount equal to 5% of the cementitious materials in both fly ash and slag concrete. The reduction in the chloride permeability of the specimens containing fly ash and silica fume is of particular significance. Specimens containing 25% fly ash had high chloride permeability, while those containing 20% fly ash and 5% silica fume had low chloride permeability (about 1000 to 1300 coulombs at 28 days). Specimens with a combination of 45% slag and 5% silica fume showed a further reduction in chloride permeability, and for either curing procedure, the values for permeability were below 800 coulombs at 28 days.
- 2) For concrete specimens made with type II cement, there were significant differences in permeability between those moist-cured for one day and those moist-cured for 14 days. However, for concrete made with type III cement, the moist-cured specimens for one day provided low chloride permeability.
- 3) Silica fume increased the strength of similar concrete to some degree, but not large.

- 4) Except for those specimens containing slag, one-day strengths higher than 3000 psi are obtainable, with a w/c of 0.40, in all the tested concrete with type III cement as well as the control concrete with type II cement. Concrete with slag and silica fume reached compressive strength of 3000 psi in a little more than 2 days.

Ozyildirim, 1994 evaluated the strength and the permeability of various combinations of silica fume and slag in concrete. He found that when silica fume is added in small amounts, 3% to 5 %, to concrete with up to 47 % slag at a water-cement ratio from 0.40 to 0.45, economical concretes with very low permeability (353 coulombs at 28 days) and adequate strength (55.6 MPa or 8050 psi) can be produced.

He also found that curing concrete containing silica fume and slag at a temperature that is higher than room temperature for some initial curing period is beneficial, in that lower permeability will be developed within 28 days. In his study, it was found that concrete cured for the first 3 days at 38 °C (100 °F) had a permeability ranging from 353 to 589 coulombs for type III cement and 536 to 1166 coulombs for type II cement. He indicated that concrete with slag and silica fume had a lower permeability when steam-cured compared to moist-cured at room temperature.

Naik et al., 1994 studied the influence of addition of a class C fly ash on concrete strength and permeability. Fine aggregate with a 6.35-mm maximum size of natural sand and coarse aggregates with 25-mm nominal maximum size of rounded and crushed limestone were used in the mixtures. A superplasticizer and an air-entraining agent were

also added in each mix. The concrete mixtures were proportioned to replace cement with Class C fly ash in the range from 0 to 70% by weight. For each concrete mixture, compressive strength, chloride permeability, air permeability, and water permeability were determined.

They found the following results:

1. At ages up to 28 days, no-fly ash concrete attained lower air permeability and water permeability compared to high volume fly ash concrete. At age of 91 days, the mixture having 50% cement replacement exhibited the lowest air permeability and water permeability, the high volume fly ash concrete having 50% cement replacement showed lowest permeability (coulomb value was 2000) to chloride ions among all the mixtures tested;
2. In general, the high volume fly ash concrete attained lower strengths compared to the concrete without fly ash. At age of 90 days, the strength of the concrete with 70% of fly ash was about 5200 psi, and the strength of the concrete without fly ash was almost 6800 psi.
3. Test results revealed that concrete permeability was not strongly correlated to compressive strength, especially at later age.

Cabrera et al., 1997 made a laboratory study on the strength and chloride permeability resistance of fly ash and Ordinary Portland Cement (OPC) concrete and mortars. Concrete was exposed to salt water to record diffusion measurement data under two simulated exposure conditions: a) intermittent splashing, b) capillary-osmosis absorption, both of which were attempted to simulate site exposure conditions. During

their study, the unique advantage of fly ash as a workability enhancer was confirmed, i.e. the reduction of the water content without impairing the concrete workability. Also they found that concrete with 33% replacement of OPC by fly ash gave the same compressive strengths as OPC mixtures at 28 days. The chloride diffusion coefficient of OPC concrete was reduced from 25% to 42% by the substitution of OPC with fly ash.

2.3 High Performance Concrete Bridge Decks in Four Lead States

Researches over the past twenty years have shown that HPC has superior properties to normal performance concrete. The Federal Highway Administration (FHWA) initiated a national program in 1994 to implement high-performance concrete (HPC) in highway bridge [Moore, 1999]. In order to promote the implementation of HPC technology for use in highways and highway structures, the American Association of State Highways and Transportation Officials (AASHTO) Task Force on Strategic Highway Research Program (SHRP) Implementation, in conjunction with FHWA and its transportation research board, established a HPC lead state team in 1996 to coordinate the effort.

Due to this collaborative effort, HPC was successfully used for bridge deck construction in the four lead states including Virginia [Ozyildirim, 1999], Nebraska [Beacham, 1999], New Hampshire [Waszczuk, et al., 1999] and Texas [Ralls, 1999].

Since durability and strength are the most important aspects of HPC, all of the four lead states focused their concrete research and development on these two issues. The mixture proportions of HPC developed by these four states are summarized in Table 2.6. The compressive strength and the permeability results are also furnished in Table 2.7.

Table 2.6 Four Lead States Mixture Proportions

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 ENTR. oz	WRA oz	RETARD oz/yd ³	HRWRA oz/yd ³	W/CM
Virginia	329		329	1774 (N/A)	1173					0.4
	329		329	1787 (N/A)	1158					0.38
New Hampshire	506			1388 (L)	910	5	20		158	0.384
Texas	383	148		1856 (L)	1243	2.6		45	None	0.43
	474	221		1810 (L)	1303	None		22	160	0.35
	492 (Type II)	211		1900 (G)	1216	3.9		28	204	0.31
	427 (Type II)	184		1856 (G)	1239	3.9		26	None	0.42
	610 (Type II)	0		1856 (G)	1243	3.9		26	None	0.42
Nebraska	754 (Type IP)	76		1400 (L)	1409			30	135	0.38

Note: L—Limestone; G—Gravel

Table 2.7 Compressive Strength and Rapid Chloride Permeability Results of Lead States

STATE	28-D COMPRESSIVE STRENGTH (PSI)	28-D COULOMB VALUE
Virginia	8710	898
	6680	983
New Hampshire	8163 - 9614	609 - 896
Texas	> 4060	1730
	> 4060	900
	> 4060	690
	4060	1380
	4060	2490
Nebraska	9133 (56 - d)	598 (56 - d)

2.4 HPC Bridge Decks in Other States

The showcases of the four lead states promoted HPC technology by demonstrating the capabilities and cost effectiveness of this alternative to conventional concrete construction. To date, approximately 15 states (including the four lead states) have become involved with HPC in bridge construction. This section reviews the application

of HPC in other three states, Florida, New York and Montana, as examples of the growing application of HPC in bridge deck construction.

2.4.1 Florida HPC Experience

Due to severe deterioration along the coastline and intracoastal waterways, the Florida State Department of Transportation (FDOT) initiated the research on HPC by using fly ash in the 1970's [Larsen, 1995]. They found that the addition of fly ash pozzolanic material benefited a structure in three ways: 1) improved its corrosion protective properties, 2) improved its sulfate resistance, 3) reduced the heat of hydration for equal 28 days strength [Larsen, 1987].

FDOT has focused their concrete research on the durability aspects of HPC, since this has greater potential for economic impact on their program than increased concrete design strength [Edwards, 2000]. In one project of the New Skyway Bridge, they kept the maximum water cementitious ratio as 0.44 with fly ash replacement from approximately 20% to 50% of total cementitious quantities. The mixture designs surpassed the highest concrete strength requirement for the structure was 38 MPa (5500 psi), the laboratory permeability testing showed that chloride penetration resistance of the HPC was within the Grade 1 range (see Table 2.3).

2.4.2 New York State HPC Experience

The New York State Department of Transportation began using HPC in 1994 to produce long-lasting and more durable bridge decks. Two materials were recommended

for the application. One was concrete with low cement content using pozzolans as substitution for cement and low ratio of water to total cementitious materials while maintaining the workability by using water-reducing admixture. The concrete contains 20% Class F fly ash and 6% microsilica, with a total cementitious content of 405 Kg/m³ [Streeter, 1996]. The other material was the use of penetrating protective sealers. Use of protective sealers was deemed an appropriate means to protect “green” concrete from the early exposure to chloride. Curing period was increased from 7 days to 14 days. The performance of HPC in New York has been very good to date with some important characteristics as: average 28-d compressive strength of 37.25 MPa (5400 psi). Permeability in the field of 50% to 70% less than that of conventional concrete, average permeability of 1600 coulombs, and reduced shrinkage cracking [Streeter, 2000].

2.4.3 Montana State HPC Experience

The Montana Department of Transportation (MDT) started a HPC project in 1998 [Freeman, 1999]. The primary intention of their research was to decrease permeability, rather than increase compressive strength. Since very high strength concrete may cause drying and thermal shrinkage cracking problems. Four mixtures using two kinds of aggregates were designed: 1) “Special Deck Concrete” (SDC) mixture currently used for bridge decks in Montana; 2) HPC mixture with fly ash as the only mineral admixture; 3) HPC mixture with silica fume as the only mineral admixture; and 4) HPC mixture with both fly ash and silica fume. The mixture proportions are shown in Table 2.8.

Table 2.8 Mixture Proportions in Montana State [Freeman, 1999]

TYPE	FLY ASH (%)	SILICA FUME (%)	MAXIMUM W/(C+P)	SLUMP (MM)
Special Deck Concrete	0	0	0.40	38 to 76
HPC with Fly Ash	25	0	0.35	76 to 178
HPC with Silica fume	0	10	0.35	76 to 178
HPC with Fly Ash and Silica fume	15	10	0.35	76 to 178

Note: Percentage replacement of cement by mass

All the HPC mixture proportions maintained the cement factor of 7 sacks/cy (390 kg/m³). It was found that all three HPC mixtures had good workability and they met the current Montana Department of Transportation (MDT) specifications for bridge decks. The HPC mixtures generally had higher compressive strengths (average 28-day strength was 39.1 MPa or 5670 psi) than the normal Portland cement concrete mixture (Special Deck Concrete), which had 28-day strength of 33.1 MPa (4800 psi). The HPC with silica fume attained the highest strength at 7 days, which was 31.1 MPa (4800 psi). The mixtures with highest strengths at 28 days and 56 days were those having silica fume and those with both fly ash and silica fume.

2.5 West Virginia Division of Highways HPC Specifications

The West Virginia Division of Highways (WVDOH) developed their own specifications for Class H concrete used in bridge decks. These specifications are discussed in detail in this section.

2.5.1 *Materials*

The free moisture content of each aggregate type, at the time of batching, shall not exceed 7% of the saturated-surface dry weight of the fine or coarse aggregate, or 8% total for both aggregates. The sources of coarse aggregate shall be approved by WVDOH.

2.5.2 *Proportioning*

Class H concrete shall consist of a homogeneous mixture of cement, fine aggregate, coarse aggregate, micro silica admixture, and fly ash or ground granulated blast furnace slag, chemical admixtures and water.

Design mixture testing shall include air content, slump, and compressive strength results at 28 days and results of rapid chloride permeability tests (RCPT) at age of 35 to 42 days. The results of RCPT shall not exceed 750 coulombs. The 28-day compressive strength of the test mix that satisfies the 750 coulombs shall be accepted as Class H concrete. Table 2.9 summarizes these specifications.

Table 2.9 WVDOH Specifications of HPC

SPECIFICATION ITEMS	VALUES
Class of Concrete	H
Max W/(C+P) ratio	0.40
Entrained Air (%)	6.5 ± 0.5
Max Slump (inch)	7
Coulomb value of RCPT at 35-42 days	< 750
Design 28-d compressive strength (psi)	> 4000

Note: W = water, C = cement, P = pozzolans

2.5.3 Testing

Compressive strengths shall be greater than eighty percent of the 28-day compressive strength of the approved test mix. Drying shrinkage test shall be tested at 1, 4, 7, 14, 28, 56, and 90 days. These tests shall be performed at the same frequency as the compressive tests.

Note: The chloride permeability requirement of less than 750 coulombs would correspond to the highest performance Grade 4 as suggested by FHWA (see Table 2.3) and this is an unrealistic expectation that will need to be revised.

2.6 Significance of Current Research

The present study is part of a comprehensive project on the development of HPC mixes for cast-in-place bridge decks in West Virginia, using locally available aggregate materials from different locations within the state, and fly ash, and/or slag available in the state. This is an exploratory study to establish appropriate mixture designs using limestone, a type of coarse aggregate abundant in West Virginia, natural sand, and both slag and fly ash. The West Virginia Department of Highways (WVDOH) specifications for class H concrete is used to establish threshold mixture proportions. The proposed HPC mixtures are studied for various parameters relevant to bridge deck concrete, including strength, chloride permeability, and freeze-thaw durability. From the results, the best possible combinations for fly ash and slag will be selected and used as prototypes to focus a subsequent global research program on HPC for the state of West Virginia.

Chapter 3

Experimental Program

The materials, sample preparations, and testing of HPC mixtures are discussed in this chapter.

The testing procedures and materials used throughout this study are according to relevant ASTM and AASHTO specifications and standards. All materials and testing procedures also comply with the requirements of the West Virginia Division of Highways (WVDOH).

3.1 Materials

The primary Materials used in this experimental program, such as Portland cement, fly ash, slag, fine aggregate and coarse aggregate were supplied by Arrow Concrete Company, a local ready-mix concrete plant approved by the WVDOH. Chemical admixtures such as air entraining agent (AEA), high range water reducing agent (HRWRA), and silica fume were supplied by Master Builders Company (USA). The WVDOT-DOH uses concrete mixtures supplied by ready-mix companies like Arrow Concrete Company, for most of their Class H concrete. Most of the Class H concrete is mixed with chemical admixtures supplied by Master Builders Company.

3.1.1 Portland Cement

Commercially available Type I Portland cement conforming to ASTM C 150 was used in this study. For the purpose of mixture proportioning a specific gravity of 3.15 was assumed. Table 3.1 lists the chemical compound composition of the Type I cement used in this study.

Table 3.1 Material Composition of Type I Portland Cement [*Arrow Concrete Company*]

ELEMENT	PERCENTAGE (MASS)
Tricalcium Silicate	49
Dicalcium Silicate	25
Tricalcium Aluminate	12
Tetracalcium Aluminoferrite	8
Calcium Sulfate	2.9
Calcium Oxide	0.8
Magnesium Oxide	2.4
Mass Total: 100 %	

3.1.2 Coarse Aggregate

The coarse aggregate used in this study was graded limestone of $\frac{3}{4}$ inch nominal size known as Greer Limestone. Table 3.2 and Table 3.3 show some of the properties. Figure 3.1 shows the typical limestone coarse aggregate used in the study.

Table 3.2 Properties of Coarse Aggregate (Limestone)[*Arrow Concrete Company*]

PROPERTIES	VALUE
Nominal Size	$\frac{3}{4}$ inch
Absorption (%)	0.53 %
SSD Specific Gravity	2.690
Bulk Specific Gravity	2.680
Apparent Specific Gravity	2.719

Table 3.3 Sieve Analysis of Coarse Aggregate [*Arrow Concrete Company*]

Sieve Size	SPECIFICATIONS		PERCENTAGE PASSING
	Low	High	Greer #57 (L)
1"	95	100	100
$\frac{3}{4}$ "	-	-	-
$\frac{5}{8}$ "	-	-	73
$\frac{1}{2}$ "	25	60	45
$\frac{3}{8}$ "	-	-	-
#4	0	10	1
#8	0	5	3

Figure 3.1 Graded Limestone of $\frac{3}{4}$ inch Size

3.1.3 Fine Aggregate

The fine aggregate used in this experimental program was graded river sand of 3/8 inch nominal size with some of the properties shown in Table 3.4 and Table 3.5.

Figure 3.2 shows the typical sand used in the study.

Table 3.4 Properties of Fine Aggregate [*Arrow Concrete Company*]

PROPERTIES	VALUE
Absorption	100 %
SSD Specific Gravity	2.611
Bulk Specific Gravity	2.586
Apparent Specific Gravity	2.653

Table 3.5 Sieve Analysis of Fine Aggregate [*Arrow Concrete Company*]

Sieve	SPECIFICATIONS		PERCENTAGE PASSING
	Low	High	River Sand
3/8"	100	100	100
#4	95	100	97.2
#8	80	100	82.3
#16	50	85	69
#30	25	60	54.6
#50	5	30	16.1
#100	0	10	2
#200	-	-	0.7
FM	2.3	3.1	2.79

Figure 3.2 Typical River Sand

3.1.4 Fly Ash

The Class F fly ash used in this study conformed to ASTM C 618 (Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete). The specific gravity of the fly ash was 2.4.

3.1.5 Slag

The ground granulated blast-furnace slag used in this study conformed to ASTM C989 (Standard Specification for Ground Granulated Blast-Furnace Slag for Use in Concrete). The specific gravity of the slag was 2.8.

3.1.6 Silica Fume

The silica fume used in this study conformed to ASTM C 1240 (Standard Specification for Use of Silica Fume as a Mineral Admixture in Hydraulic Cement Concrete, Mortar, and Grout). The specific gravity of the silica fume was 2.2.

Figure 3.3, 3.4, and 3.5, respectively, show the typical fly ash, slag, and silica fume used in this study.

Figure 3.3 Class F Fly Ash

Figure 3.4 Slag

Figure 3.5 Silica Fume

3.1.7 Chemical Admixtures

Two chemical admixtures were used in the experimental program: high range water reducing agent (HRWRA) and air entraining agent (AEA). The HRWRA used in the mixtures was a naphthalene-based superplasticizer conforming to ASTM C 494 type F. The air-entraining agent (AEA) used was based on neutralized vinsol resin meeting the requirements of ASTM C 260.

3.1.8 Mixing Water

The mixing water used in this study was tap water from the Morgantown city water supply and was assumed to have a specific gravity of 62.4 lbs per cubic foot (1000 kg per cubic meter).

3.2 Mixture Proportions

A total of six mixtures were prepared for the study, three with fly ash and other three with slag. One of the three mixtures for each fly ash and slag was made according to the WVDOT-DOH specifications for Class H concrete and was used as a reference recipe. The other mixtures were developed by varying the quantity of fly ash or slag while keeping the quantity of cementitious material and water-cementitious ratio constants. In all the mixtures, the slump and air content values were maintained within a close range. This was done to facilitate the direct comparisons among the mixtures. In order to achieve a uniform range of slump, the quantity of HRWRA was varied. The

quantity of fine aggregate was adjusted to maintain the yield of the mixtures. The mixtures considered are as follows:

Group F: Fly Ash Group

- **F1:** Cement/Fly ash/Silica fume = 75/20/05 (by weight)

This mix design is based on WVDOT-DOH specifications for class H concrete, and is considered a reference mixture for group F.

- **F2:** Cement/Fly ash/Silica fume = 70/25/05 (by weight)
- **F3:** Cement/Fly ash/Silica fume = 80/15/05 (by weight)

Group S: Slag Group

- **S1:** Cement/Slag/Silica fume = 65/30/05 (by weight)

This mix design is based on WVDOT-DOH specifications for class H concrete, and is considered a reference mixture for group S.

- **S2:** Cement/Slag/Silica fume = 60/35/05 (by weight)
- **S3:** Cement/Slag/Silica fume = 70/25/05 (by weight)

The quantities of fine aggregate, coarse aggregate, high range water reducing agent (HRWRA), and air-entraining agent (AEA) were finalized through several trial mixes in the laboratory and guidelines obtained from the studies of the four lead states. A slump of 5 to 7 inches and air content of 5% to 8% as specified for Class H concrete by the WVDOT-DOH were maintained for all the mixtures. The mixing proportions are given in table form as follows:

Table 3.6a Mixture Proportion of Group F in US Customary Units

MIX	CEMENT	FLY ASH	SILICA FUME	WATER	COARSE AGGREGATE	FINE AGGREGATE	AIR ENTR.	HRWRA	W/C
	(Lbs)	(Lbs)	(Lbs)	(Lbs)	(Lbs)	(Lbs)	Oz.	Oz.	
F1	470	124	30	250	1750	1206	18	100	0.4
F2	438	156	30	250	1750	1221	20	75	0.4
F3	500	94	30	250	1750	1214	20	108	0.4

Note: All values are based on one cubic yard of concrete. Cm= cementitious material= Cement+ fly ash or slag+ silica fume

Table 3.6b Mixture Proportion of Group F in SI Units

MIX	CEMENT	FLY ASH	SILICA FUME	WATER	COARSE AGGREGATE	FINE AGGREGATE	AEA	HRWRA	W/C
	(Kg)	(Kg)	(Kg)	(Kg)	(Kg)	(Kg)	ml	ml	
F1	279	74	18	148	1038	716	696	3868	0.4
F2	260	93	18	148	1038	724	773	2907	0.4
F3	297	56	18	148	1038	720	773	4183	0.4

Note: All values are based on one cubic meter of concrete. Cm= cementitious material= Cement+ fly ash or slag+ silica fume

Table 3.7a Mixture Proportion of Group S in US Customary Units

MIX	CEMENT	SLAG ASH	SILICA FUME	WATER	COARSE AGGREGATE	FINE AGGREGATE	AIR ENTR.	HRWRA	W/C
	(Lbs)	(Lbs)	(Lbs)	(Lbs)	(Lbs)	(Lbs)	Oz.	Oz.	
S1	423	194	30	259	1750	1178	38	82	0.4
S2	391	226	30	259	1750	1174	35	105	0.4
S3	455	162	30	259	1750	1198	38	97	0.4

Note: All values are based on one cubic yard of concrete. Cm= cementitious material= Cement+ fly ash or slag+ silica fume

Table 3.7b Mixture Proportion of Group S in SI Units

MIX	CEMENT	FLY ASH	SILICA FUME	WATER	COARSE AGGREGATE	FINE AGGREGATE	AIR ENTR.	HRWRA	W/C
	(Kg)	(Kg)	(Kg)	(Kg)	(Kg)	(Kg)	ml	ml	
S1	251	115	18	154	1038	699	1471	3187	0.4
S2	232	134	18	154	1038	696	1354	4072	0.4
S3	270	96	18	154	1038	711	1471	3748	0.4

Note: All values are based on one cubic meter of concrete. Cm= cementitious material= Cement+ fly ash or slag+ silica fume

3.3 Mixing Procedure

All mixing was done in a standard laboratory rotary drum mixer. Figure 3.6 shows the type of mixer used in this study. The moisture content of the fine aggregate was determined just prior to the mixing to obtain the correct value of surface moisture and to adjust the mixing water accordingly to maintain the effective water-cementitious ratio as 0.40. The coarse aggregate was assumed to be in a state of saturated surface dry condition.

The mixing sequence was as follows:

1. Approximately one third of water was added.
2. All the coarse aggregate and fine aggregate were added. The mixer was rotated until the aggregates were well mixed.
3. All cementitious materials such as cement, fly ash or slag, silica fume and remaining mixing water were added and mixed well for about five minutes or until the mixture was uniform.
4. HRWRA and AEA were added while the mixer was kept rotating for another 8 to 10 minutes or until the mixture was uniform. The HRWRA dosage was controlled to obtain the desired slump.

As soon as the mixing was complete, slump, temperature, air content and unit weight of concrete were measured according with relevant ASTM and AASHTO standards.

Figure 3.6 Drum Mixer of 3-cubic Feet Capacity

3.4 Preparations of Test Specimens

3.4.1 Cylinder Specimens for Compressive Strength

Twenty-three cylinder specimens of 4 in. dia. x 8 in. long (101.6 mm x 203.2 mm) cylinder specimens were casted in two batches for each mixture using plastic molds. Twenty one of them were used for compressive strength tests at different ages and the other two were used for rapid chloride permeability tests. Figures 3.7 and 3.8 show the test specimens. The cylinders were casted in accordance with ASTM C 192 (Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory). Within few minutes after casting, the specimens were covered with a plastic sheet and left to cure at room temperature in the laboratory for 24 hours. The specimens were then demolded and subjected to the specified curing condition as described later in section 3.4.5.

Figure 3.7 Compressive Strength Test Specimen
(4 in. dia. x 8 in. high)

3.4.2 *Beam Specimens for Length Change Measurement (Shrinkage)*

Three 3 in. x 3 in. x 10 in. long (76.2 mm x 7.2 mm x 254 mm long) beam specimens (Figure 3.8) were casted in two batches (two from batch 1, the other one from batch 2) using steel molds. The dimensions of the beam specimen were according to ASTM C 157 (Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete). After casting, the specimens were covered with a plastic sheet. After 24 hours, the demolded specimens were subjected to the specified curing condition as described in section 3.4.5.

Figure 3.8 Shrinkage Beam Specimen
(3 in. x 3 in. x 10 in.)

3.4.3 Beam Specimens for Freeze/Thaw Durability Test

Two 3 in. x 4 in. x 16 in. long (76.2 mm x 101.6 mm x 406.4 mm long) beam specimens (Figure 3.9) were casted in two batches for each mixture by using steel molds. The dimensions of the beam specimen were in accordance with ASTM C 666 (Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing). Soon after casting, the specimens were covered with a plastic sheet. After 24 hours, the demolded specimens were subjected to the specified curing condition as given in section 3.4.5.

Figure 3.9 Freeze-Thaw Beam Specimen
(3in. x 4 in. x 16 in.)

3.4.4 Disk Specimen for Rapid Chloride Permeability Test

For each mixture, two disks of 4 inch diameter x 2 inch thickness were cut from the top portions of the cylinder specimens for chloride permeability tests. The top portion of the cylinder was selected to simulate the exposed section of a concrete bridge deck, which is normally subjected to de-icing salts. Two disks for each mixture were cut by taking one sample from each batch. The diameter of the disk specimen was in accordance with ASTM C 1202 (Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration).

Figure 3.10 Rapid Chloride Permeability Test Specimen
(2 in. thickness x 4 in. diameter)

3.4.5 Curing of Different Types of Test Specimens

Twenty-four hours after casting, all of the specimens were removed from the molds, and the cylinder specimens were placed under wet burlaps in a curing room at 73°F until the day of testing. The wet burlap ensured the relative humidity to be about 100 percent.

The freeze-thaw test specimens were cured under limewater for 14 days at 73°F and then subjected to the rapid freezing-thawing cycles. Specimens for measurement of length change (shrinkage) were cured under limewater for 7 days at 73°F and then stored in a curing room at a temperature of 73°F $\pm$ 3°F and a humidity of about 50% $\pm$ 4%. The length-change (shrinkage) specimens were stored in such a way as to allow as much air circulation around the surfaces of the specimen as possible.

3.5 Testing Procedures

3.5.1 Testing on Fresh Concrete

These tests include slump, air content, unit weight and temperature.

3.5.1.1 Slump Test

The slump of fresh concrete was measured in accordance with ASTM C 143 (Standard Test Method for Slump of Hydraulic Cement Concrete) and AASHTO T 119. Soon after the mixing was completed, the quality of mixture was also noticed visually. Figure 3.11 shows the test apparatus.

Figure 3.11 Metal cone to Measure Slump

3.5.1.2 Air Content

The air content of fresh concrete was determined by the pressure method in accordance with ASTM C 231 (Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method) and AASHTO T 152. Figure 3.12 shows the test apparatus.

Figure 3.12 Apparatus Used to Measure the Air Content in the Concrete

3.5.1.3 Unit Weight

The unit weight of fresh concrete was measured per ASTM C 138 (Standard Test Method for Unit Weight, Yield, and Air Content of Concrete) and AASHTO T 121.

3.5.1.4 Temperature

The temperature of concrete was measured with a standard thermometer, with an accuracy of ± 0.5 °F, soon after mixing was completed, in accordance with ASTM C

1064 (Standard Test Method for Temperature of Freshly Mixed Portland Cement Concrete).

3.5.2 Testing of Hardened Concrete

These tests include compressive strength, shrinkage, freeze-thaw, and rapid chloride permeability.

3.5.2.1 Compressive Strength

Compressive strength of cylinder specimens was measured in accordance with ASTM C 39 (Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens) and AASHTO T 22 using 350,000 lbs capacity hydraulic type compression machine. Tests were conducted on specimens at 1, 3, 7, 14, 28, 56, and 90 days. For each mixture three specimens were tested taking two from batch 1 and one from batch 2. Figure 3.13 shows the testing machine with specimen. Figure 3.14 shows the specimen after typical compression failure.

Figure 3.13 Compressive Strength Test Machine

Figure 3.14 Specimen After Shear Failure

3.5.2.2 Length Change Measurement (Shrinkage) Test

The length change of concrete specimens was measured in accordance with ASTM C 157 (Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete) and AASHTO T 160. For each mixture, three specimens were tested, taking two from batch one and one from batch two. The readings were taken by a standard length change comparator at 4, 7, 14, 21, 28, 35, 42, 49 and 56 days, after curing of specimens for 7 days as mentioned before in 3.4.5. Figure 3.15 shows a length-change test in progress.

Figure 3.15 Length Change Measurement in Progress

3.5.2.3 Freeze/thaw Durability Test

The freeze-thaw test was conducted in accordance with ASTM C 666 (Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing) and AASHTO T 161. After removing the specimen from the lime saturated water, they were wiped with damp towels, and then, measurements of length, mass and fundamental frequency were taken before they were subjected to freeze-thaw cycles. The length, mass and fundamental frequency values were noted after each 50 cycles. The fundamental transverse frequency was determined to measure the change in dynamic modulus of elasticity of concrete after each 50 cycles, and the procedure used followed the ASTM C 215 standard. Each cycle of freezing-thawing was maintained as follows: the temperature was lowered to 0°F from 40°F in a time period of 2 hours and then raised to 40 °F also within 2 hours. The entire procedure is planned to continue for a total of 300 cycles or

until there is visible and noticeable deterioration. Due to the slow ratio of freeze-thaw cycles (about 6 cycles a day), the maximum cycles of 300 was not reached for specimen S1. The present thesis, therefore, reports the durability factor of all the mixtures except fro S1. However we report the resonant frequency values for S1 up to 250 cycles. Figure 3.16 shows the freeze-thaw testing chamber with the specimens. Figures 3.17a and 3.17b show the fundamental transverse frequency test in progress.

Figure 3.16 Freeze/Thaw Testing Chamber with Specimens

Figure 3.17a Sonometer (Resonance Frequency Test Apparatus)

Figure 3.17b Fundamental Transverse Frequency Test in Progress

3.5.2.4 Rapid Chloride Permeability Test

The chloride permeability test of disc specimens cut from cylinders was conducted in accordance with ASTM C 1202 (Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration) and AASHTO T 277. Figure 3.18a shows the apparatus. The circular surfaces of the disc specimens were coated with an epoxy sealant. The specimen was then brought to a standard moisture condition by the following vacuum saturation procedure: Vacuum was applied to the dry specimen for 3 hours, and then continued for 1 more hour with the specimen immersed in de-aerated water. After then the specimen was soaked in the same water for an additional 18 ± 2 hours at atmospheric pressure. The ends of the specimen were then sealed into hollow, polymethyl methacrylate (Plexiglas) chambers. The side of the cell containing the top of the sample was filled with a 3% sodium chloride solution, and the other side containing the bottom with a 0.3N sodium hydroxide solution. An electric current of sixty volts DC was applied across the specimen between copper screen electrodes contained in each cell. The total charge passed, or the integral of the current with respect to time, during a 6 hours period is a measure of the chloride permeability of the concrete. The test was conducted on properly cured 35 to 42 days old specimen. For each mixture, two specimens were tested, taking one from each batch. Figures 3.18b and 3.18c show the test in progress.

Figure 3.18a Vacuum System and Charge-Passed Measurement Apparatus

Figure 3.18b Chloride Permeability Test in Progress

Figure 3.18c Close View of Chloride Permeability Test Specimens

Chapter 4

Test Results and Discussions

This chapter presents the test results and discussions for fresh and hardened concrete. The results are presented in tables and figures. The fresh concrete properties include slump, air content, unit weight, temperature, and mix characteristics. The hardened concrete properties include compressive strength at different ages, drying shrinkage, freeze-thaw durability, and chloride permeability.

4.1 Fresh Concrete Properties

Each mixture of fresh concrete was tested according to the following standard methods: *ASTM C143, Slump of Hydraulic Cement Concrete*; *ASTM C138, Unit Weight and Air Content*; *ASTM C231, Air Content of Freshly Mixed Concrete by Pressure Method*; *ASTM C1064, Temperature of Freshly Mixed Portland Cement Concrete*, and their corresponding AASHTO standards.

Table 4.1 and Table 4.2 show the mixture proportioning of HPC for the fly ash group (Group F) and slag group (Group S), respectively. Table 4.3 and Table 4.4 list the fresh properties of HPC for the fly ash group (Group F) and slag group (Group S), respectively.

Table 4.1. Mixture Proportioning of HPC of Fly Ash Group (Group F)

Mixture Type	Cement (lb/yd ³)	Fly ash (lb/yd ³)	Fine Aggr. (lb/yd ³)	AEA (oz./yd ³)	HRWRA (oz./yd ³)	W/Cm ratio	C/FA/SF ratio by weight
F1(current DOH)	470	124	1206	18	100	0.40	75/20/5
F2	438	156	1221	20	75	0.40	70/25/5
F3	500	94	1214	20	108	0.40	80/15/5

Note: For all mixtures, Coarse aggregate (Limestone) = 1750 lb/yd³, silica fume (SF) = 30 lb/yd³, Water = 250 lb/yd³, Cementitious material (Cm) = 624 lb/yd³, Cm = C + FA + SF.

Table 4.2. Mixture Proportioning of HPC of Slag Group (Group S)

Mixture Type	Cement (lb/yd ³)	Slag (lb/yd ³)	Fine Aggr. (lb/yd ³)	AEA (oz./yd ³)	HRWRA (oz./yd ³)	W/Cm ratio	C/SG/SF ratio by weight
S1(current DOH)	423	194	1178	38	82	0.40	65/30/5
S2	391	226	1174	35	105	0.40	60/35/5
S3	455	162	1198	38	97	0.40	70/25/5

Note: For all mixtures, Coarse aggregate (Limestone) = 1750 lb/yd³, silica fume (SF) = 30 lb/yd³, Water = 259 lb/yd³, Cementitious materials = 647 lb/yd³, Cm = C + SG + SF.

Table 4.3. Characteristics of Freshly Mixed HPC of Fly Ash Group (Group F)

MIXTURE	AIR CONTENT	SLUMP		UNIT WEIGHT		TEMPERATURE	
	(%)	(mm)	(inch)	(kg/m ³)	(pcf)	(°C)	(°F)
F1 (Current DOH)	6.0	175	7.0	2387	149	17.2	63
F2 (More FA)	5.0	140	5.5	2371	148	17.8	64
F3 (Less FA)	7.0	150	6.0	2323	145	16.7	62

Table 4.4. Characteristics of Freshly Mixed HPC of Slag Group (Group S)

MIXTURE	AIR CONTENT	SLUMP		UNIT WEIGHT		TEMPERATURE	
	(%)	(mm)	(inch)	(kg/m ³)	(pcf)	(°C)	(°F)
S1 (Current DOH)	6.0	165	6.5	2371	148	18.3	65
S2 (More SG)	5.2	180	7.0	2371	148	15.6	60
S3 (Less SG)	7.0	180	7.0	2291	143	18.3	65

Table 4.3 shows that values of both air content (5.0% to 7.0%) and slump (5.5 to 7 inches) of the Group F HPC were within the range of the WVDOH requirements for Class H concrete. Table 4.1 indicates that the demand for HRWRA decreases as the fly ash content increases, while slump is within a range of 5.5 to 7 inches, and the W/Cm ratio and cement factor are constant. Though the F2 mixture shows a relatively low slump value of 5.5 inches, it could have been improved by a little addition of HRWRA. Table 4.3 further shows that the unit weight of the current WVDOH Class H concrete was the maximum for the Group F, though all of the mixtures are within a narrow range of normal unit weight of 145 pcf to 149 pcf. The temperature of the concrete just before placement was also within a reasonable range of 62 °F to 64 °F. Increasing or decreasing the fly ash content by $\pm 5\%$ (by weight) over the current WVDOH mixture did not influence the fresh properties much.

Table 4.4 shows that values of both air content (5.2% to 7.0%), and slump (6.5 to 7.0 inches) of the Group S HPC were within the range of the WVDOH requirements for Class H concrete. Table 4.2 indicates that the demand for HRWRA was maximum for the HPC with slag content of 35%, while the current WVDOH Class H concrete needed the minimum quantity of HRWRA to achieve a slump range of 6.5 to 7.0 inches, and

constant W/Cm ratio and cement factor. Table 4.4 indicates that the variation of slag by $\pm 5\%$ from the current WVDOH mixture did not influence the fresh properties much. The temperature of the concrete just before placement was within 60 °F to 65 °F. It is important to observe that the HPC for the Slag Group needed about 73% to 110% more AEA than the HPC for the Fly ash Group to achieve almost the same range of air content. The S2 and S3 mixtures were more workable than the counterpart F2 and F3 mixtures.

In general, it was observed that increasing the fly ash or slag content over over current WVDOH mixture made the concrete more cohesive and free from any segregation and bleeding, even under vibration with a needle vibrator. While decreasing the fly ash or slag content over the corresponding WVDOH mixtures made the mixtures susceptible to bleeding and segregation to some extent during vibration.

4.2 Compressive Strength

The compressive strength test was performed according to the standard method *ASTM C39, Compressive Strength of Cylindrical Concrete and AASHTO T 22*. The compressive strength was measured at ages of 1, 3, 7, 14, 28, 56, and 90 days, and the values for the individual specimen, along with the mean and the variations from the mean values are given in Appendix A (Tables A.1 through A.6).

Table 4.5 and Table 4.6 present the average values of compressive strengths for the Group F and Group S concrete, respectively. The variations of compressive strength among the specimens for each age were within the prescribed limit of ASTM C 39. Figure 4.1 shows the best-fit compressive strength-age relationship for all HPC of the fly ash group, while figure 4.2 shows the same relationship for all HPC of the slag group.

Table 4.5 Compressive Strength of Group F HPC at Different Ages

Time (Days)	F1 (current DOH)	F2 (5% more FA)	F3 (5% less FA)
	(psi)	(psi)	(psi)
1	3640	3340	3420
3	5190	5070	5410
7	6530	5910	6420
14	7520	6680	7270
28	8500	8070	7740
56	9140	8760	8450
90	9630	9610	9200

Table 4.6 Compressive Strength of Group S HPC at Different Ages

Time (Days)	S1 (current DOH)	S2 (5% more SG)	S3 (5% less SG)
	(psi)	(psi)	(psi)
1	2800	2290	2370
3	4620	4230	4990
7	6620	6140	6420
14	7620	7170	7910
28	9210	8730	8610
56	9940	9120	9380
90	11170	9650	10030

The general form of the best-fit equation in Figs. 4.1 and 4.2 is

$$F(t) = A - Be^{-mt^n} \quad (4.1)$$

Where,

t – time in days, $1 \leq t \leq 90$,

F(t) – compressive strength at time t

The corresponding parameters for all six mixtures are furnished in Table 4.7. Figures 4.3 and Figure 4.4 show the comparison of 28-day compressive strengths for the Group F and Group S HPC mixtures, respectively. Figure 4.5 and Figure 4.6 show the comparison of 56-day compressive strengths for the Group F and group S mixtures, respectively.

Table 4.7 Parameters for Weibull Data Fit

	F1	F2	F3	S1	S2	S3
A	10321	21936	11320	16031	9847	11605
B	11798	31649	74182	27416	11784	56432
m	1.76	1.86	0.44	1.38	2.26	0.55
n	0.35	0.12	0.10	0.19	0.48	0.15
R	0.998	0.997	0.997	0.998	0.998	0.999

Figure 4.1 Compressive Strength – Age Relationship of Group F HPC

Figure 4.2 Compressive Strength – Age Relationship of Group S HPC

Figure 4.3 28-day Compressive Strength of Group F HPC Mixtures

Figure 4.4 28-day Compressive Strength of Group S HPC Mixtures

Figure 4.5 56-day Strengths of Group F Mixtures

Figure 4.6 56-day Strengths of Group S Mixtures

Table 4.5 and 4.6, as well as figures 4.1 through 4.6 indicate that the compressive strengths of all HPC mixtures increased steadily with time.

Tables 4.5 and 4.6 show that up to 3 days, the HPC with fly ash achieved compressive strengths at higher rates than the HPC with slag. The W/Cm ratio and cement factor being the same, the design of the mixture proportioning allowed fly ash concrete to develop strength at a faster rate than that of slag concrete at very early age (up to 3 days). However beyond 3 days, the slag concrete gained strength at a much faster rate than the fly ash concrete. Ozyildirim and Hasteed (1988) reported a similar behavior of slag-concrete. The maximum compressive strength achieved by the HPC of Group F at 90 days was 9630 psi (F1 mixture), while the corresponding value for the HPC of Group S was 11170 psi (S1). At 28 days (the age of traditional design strength) the HPC of Group F achieved 8500 psi (F1), while the HPC of Group S reached 9210 psi (S1).

Figure 4.1 indicates that among the fly ash group, initially the rate of gain of compressive strength for F1 (current WVDOH) was slightly higher than the other two mixtures, but at later age (90-day), the HPC with higher fly ash content (F2) reached almost the same value as that of F1 (current WVDOH). The pozzolanic reaction of relatively higher quantity of fly ash helped the F2 mixture gain higher compressive strength at later ages.

Comparisons of both 28-day and 56-day compressive strengths among the F Group mixtures (Figure 4.3 and Figure 4.4) show that the current WVDOH HPC (F1) has the highest compressive strength followed by F2 and F3. Both 28-day and 56-day compressive strengths are important, since 28-day strength is used traditionally as design

strength and 56-day strength was recently recommended by the Federal Highway Administration (FHWA) (Good Speed et al., 1996) as the strength parameter in assessing HPC.

Test results show that higher fly ash content (25% by weight of total cementitious material) reduces slightly the 28-day and 56-day compressive strength over the current WVDOH mixture, but the strength values are reasonably good (8070 and 8760 psi respectively) to be used as HPC for bridge decks. The mixture with lower fly ash content (15% by weight of total cementitious material) exhibited the lowest 28-day and 56-day compressive strengths.

Similar results were obtained in Texas, where two bridges were constructed with HPC (Ralls, 1999) using Fly ash as partial replacement of cement at 28% to 32% by weight. The 28-day strength of the bridge deck concrete was in the range of 6000 to 8000 psi. Also in Nebraska (Beacham, 1999), Class C Fly ash was used in one HPC bridge deck, with 10% fly ash by weight, they reported a 56-day compressive strength of 9100 psi.

Among the HPC corresponding to the slag group, rate of gain of compressive strength (Figure 4.2) of S1 (current WVDOH) was the highest, and the rate of compressive strength gain of S2 and S3 were lower than S1 but close to each other. Figure 4.4 and 4.6 show that the S1 mixture has the highest 28-day and 56-day compressive strengths, while the corresponding 28-day values for S2 and S3 mixtures were close to the S1 mixture. Although concrete with higher slag content (35% by weight of cementitious material) attained lower compressive strength compared to the current WVDOH, the values are reasonably good (28-day strength = 8730 psi, and 56-

day strength = 9120 psi) to be used as HPC for bridge decks. The mixture with lower slag content, which is the one with higher cement content, exhibited the lowest 28-day compressive strength.

Ozyildirim (1999) showed in a recent study that the 28-day strengths of two bridge deck concretes made with 50% of slag (by weight) were 8710 psi and 6681 psi, respectively.

4.3 Length Change Measurement (Shrinkage)

The length change test, or free 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 noted at 4, 7, 14, 21, 28, 35, 42, 49, and 56 days to measure the drying shrinkage in air at $50\% \pm 4\%$ Rh and 73 ± 3 °F. 7 days of moist curing was chosen to simulate the field condition of class H concrete in bridge decks, where the minimum curing time is recommended as 7 days.

The data for each specimen and mixture type are listed in Appendix B (Tables B.1 through B.6). We summarize the test results of all the mixtures in Tables 4.8 and Table 4.9.

Table 4.8 Drying Shrinkage of Group F HPC at Different Ages

Days of drying (After Curing)	F1 (current DOH)	F2 (5% more FA)	F3 (5% less FA)
	(micro strain)	(micro strain)	(micro strain)
4	152	165	195
7	232	251	288
14	285	339	344
21	336	308	365
28	331	349	347
35	304	360	373
42	307	384	411
49	301	341	395
56	315	341	368

Table 4.9 Drying Shrinkage of Group S HPC at Different Ages

Days of Drying (After curing)	S1 (current DOH)	S2 (5% more SG)	S3 (5% less SG)
	(micro strain)	(micro strain)	(micro strain)
4	212	172	144
7	272	248	197
14	296	296	325
21	356	301	339
28	368	315	323
35	392	376	360
42	356	339	339
49	360	315	315
56	336	296	320

Figure 4.7 and Figure 4.8 show the best-fit shrinkage-time plots for Group F HPC and Group S HPC, respectively. The general form of best-fit equation is

$$S(t) = S_s (1 - e^{(-t/t_0)}) \quad (4.2)$$

Where,

t – drying time in days, $1 \leq t \leq 56$

$S(t)$ – shrinkage at time t

and the parameters defining EQ.4.2 are furnished in Table 4.10.

Figures 4.9 and 4.10 present the average shrinkage values of Group F and Group S HPC, respectively, for data collated at 14 days of drying. Figures 4.11 and 4.12 present the shrinkage values of Group F and Group S HPC, respectively, at 28 days of drying.

Table 4.10 Parameters for Weibull Data Fit

Parameters	F1	F2	F3	S1	S2	S3
S_s	302.1	346.2	372.8	357.0	326.0	334.1
t_0	5.03	5.63	5.14	4.96	5.27	6.58
R	0.972	0.985	0.987	0.985	0.980	0.986

Figure 4.7 Drying Shrinkage of Group F HPC at Different Ages
(After 7 days of moist curing at 73 ± 3 °F)

Figure 4.8 Drying Shrinkage of Group S HPC at Different Ages
(After 7 days of moist curing at 73 ± 3 °F)

Figure 4.9 Drying Shrinkage of Group F HPC at 14 Days of Drying

Figure 4.10 Drying Shrinkage of Group S HPC at 14 Days of Drying

Figure 4.11 Drying Shrinkage of Group F HPC at 28 Days of Drying

Figure 4.12 Drying Shrinkage of Group S HPC at 28 Days of Drying

Table 4.8 shows the average drying shrinkage values of F1, F2 and F3 mixtures for up to 56 days, which are plotted in Figure 4.7 by a best-fit curve. The Figure shows that drying shrinkage increased rapidly up to 7 days, and then the rate of increase reduced. After about 14 days, the rate of increase became quite low, with almost a flat slope. Among the three mixtures, F1 (the Current WVDOH mixture) reached a steady state earlier than F2 and F3. Finally at 56 days the mean value for F1 was the lowest, followed closely by F2 and F3. In fact, the 56 days drying shrinkage values of all the specimens for all three mixtures were within a close range of 300 micro strains to 375 micro strains. The present test was conducted on specimens that were moist cured for the first 7 days, and the initial drying shrinkage soon after casting was not recorded. The present test was conducted following ASTM C 157 as stated in the WVDOH

specification for Class H concrete. Therefore, the data here may be assumed to be useful for all practical purposes as long as a continuous seven days moist curing of HPC deck is performed in the field.

Table 4.9 shows the average drying shrinkage values of S1, S2 and S3 mixtures for up to 56 days, which are plotted in Figure 4.8 by a best-fit curve. The Figure shows that shrinkage increased rapidly up to 7 days, and then the rate of increase reduced. After about 14 days, the rate of increase became very low, and the trend is very much similar to that of the fly ash concrete. Among the three mixtures of the Group S HPC, the S2 and S3 showed a slightly lower range of drying shrinkage (about 320 micro strains at 28 days) compared to S1 (current WVD OH) that was about 370 micro strains at 28 days. In general the 56-day shrinkage values for all the Group S mixtures were within a reasonable range of about 300 micro strains to 350 micro strains and also were very close to the Group F values. As with the Group F, very early shrinkage data was not collected for the Group S specimens.

In summary, the shrinkage values in general changed very little beyond 14 days of drying. Figures 4.9 and 4.11 show that for the Group F mixtures, the current WVD OH mixture performed slightly better than the F2 and F3 variations. However, the mixture with maximum cement paste/aggregate volume ratio (F3) had the maximum drying shrinkage, which is quite common. For the Group S mixtures, both the current WVD OH mixture (S1) and S2 showed similar shrinkage values, while the mixture with the highest value of cement paste/aggregate volume ratio (S3) exhibited the maximum drying shrinkage.

4.4 Rapid Freeze/Thaw Test

The freeze/thaw test was performed according to the standard method *ASTM C666, Resistance of Concrete to rapid freezing and thawing*, and *AASHTO T 161*.

The detail test data for each specimen of each mixture type can be found in Appendix C (Tables C.1 through C.7). Table 4.11 and Table 4.12 summarize the dynamic modulus of elasticity values for both Group F and Group S HPC. Table 4.13 provides the percent durability factor, based on the dynamic modulus at 300 cycles with respect to the initial value. Since mixture S1 did not reach the required 300 cycles, the measurement of durability factor is not reported.

The dynamic Young's modulus, E_d , was calculated using the following equation [ASTM C 215]

$$E_d = C W n^2 \quad (4.5)$$

where:

W – Specimen weight in lb

n – Fundamental traverse frequency in Hz

C – 0.00245 ($L^3 T/bt^3$) for the prism specimen

L – Specimen length in inch

b, t – specimen cross section dimensions in inch

T – A correction factor followed by interpolation as 1.23

The relative dynamic modulus of elasticity after 300 cycles was calculated using the following equation [ASTM, C 666]

$$P_c = (n_1^2/n^2) \times 100 \quad (4.6)$$

Where:

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

n = initial fundamental transverse frequency before freezing and thawing, and

n_1 = fundamental transverse frequency after c cycles of freezing and thawing.

The durability factor was calculated using the following equation [ASTM, C 666]

$$DF = P_c N/M \quad (4.7)$$

Where:

DF = durability factor of the test specimen,

P_c = relative dynamic modulus of elasticity at N cycles in percentage,

N = number of cycles at which P_c reaches the specified minimum value for discontinuing the test or the specified number of cycles at which the exposure is to be terminated, whichever is less, and

M = Specified number of cycles at which the exposure is to be terminated.

Since in this study, $N = M$, $DF = P_c = (n_1^2/n^2) \times 100$.

Table 4.11 Dynamic Modulus of Mixtures in Group F

Freezing/Thawing	F1	F2	F3
(Cycles)	($\times 10^4$ psi)	($\times 10^4$ psi)	($\times 10^4$ psi)
Initial	608	638	620
50	600	640	630
100	608	647	608
150	590	632	611
200	600	632	611
250	608	644	621
300	605	629	607

Table 4.12 Dynamic Modulus of Mixtures in Group S

Freezing/Thawing	S1	S2	S3
(Cycles)	($\times 10^4$ psi)	($\times 10^4$ psi)	($\times 10^4$ psi)
Initial	647	604	613
50	629	592	601
100	600	577	604
150	614	598	578
200	623	592	612
250	621	606	598
300	N/A	602	601

Note: N/A = Not available at this time

Table 4.13 Durability Factor of Group F and Group S HPC

Mixture	F1	F2	F3	S1	S2	S3
DF (%)	99	97	96	N/A	99	96

Note: N/A = Not available at this time

Table 4.11 shows that among the Group F mixtures, F2 had the highest initial dynamic modulus of elasticity (E_d) followed by F3 and F1. The results indicate that E_d values as measured by resonant frequency changed little even after 300 cycles. Since all the concrete mixtures have air contents of 5% to 7% and are much less permeable, thus less porous, than conventional concrete, the deterioration due to freeze-thaw cycles was not significant. The durability factor (Table 4.13) indicates that the F1 (current WVDOH) HPC was practically unchanged after 300 cycles, although the F2 and F3 mixtures also deteriorated very little.

Table 4.12 shows that among the Group S mixtures, S1 (current WVDOH) had the highest initial dynamic modulus of elasticity followed by S3 and S2. The data for

300 cycles of Freeze-thaw was not available for S1, but a comparison of mixtures for up to 250 cycles showed that S1 had undergone maximum loss in E_d (by 4.1%), while the change in E_d for S2 was negligible (1.8%), and S3 did not show any significant changes for up to 250 cycles. In general, the deterioration under Freeze-thaw was not significant because the mixtures had an air content of 5.2% to 7.0%.

It is presumed that samples under freeze-thaw cycles beyond 400 may show some sign of deterioration. It was observed that the bottom of most of the specimens had some of degree of scaling. The loss of weight of the specimens, however, was not significant.

As a point of comparison, a HPC mixture developed in New Hampshire (Waszczuk and Juliano, 1999) for bridge decks using Type II cement, silica fume, AEA, HRAWA, corrosion inhibitor, and local aggregates showed a freeze-thaw durability factor of 96% to 99%.

4.5 Rapid Chloride Permeability Test

The Rapid chloride permeability test (RCPT) was performed on 35-42 days cured disc specimens according to the standard test method *ASTM C1202, Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration*, and *AASHTO C 277*.

Table 4.14 and Table 4.15 summarize the permeability test results of Group F and Group S HPC, respectively. More detail data can be found in Appendix D (Tables D.1 through D.4). Figure 4.13 shows the charge-passed over 6 hours time for Group F mixtures, including the data variation for each type of specimen. Figure 4.14 shows the charge-passed over 6 hours time for Group S mixtures, including the data variation for

each type of specimens. Figure 4.15 shows a comparison of chloride permeability values among all the mixtures (Group F and Group S).

Table 4.14 Charge-passed (Coulombs) in RCPT for Group F HPC

Time (Minutes)	F1	F2	F3
	(Coulombs)	(Coulombs)	(Coulombs)
0	0	0	0
30	60	66	90
60	118	136	185
90	179	207	285
120	241	282	388
150	304	358	496
180	369	437	606
210	435	517	718
240	502	599	832
270	570	683	950
300	639	769	1068
330	708	857	1188
360	779	947	1310

Table 4.15 Charge passed (Coulombs) in RCPT for Group S HPC

Time (Minutes)	S1	S2	S3
	(Coulombs)	(Coulombs)	(Coulombs)
0	0	0	0
30	62	52	70
60	126	105	144
90	192	159	220
120	258	212	299
150	325	267	379
180	393	322	460
210	460	376	543
240	527	431	627
270	594	486	712
300	660	540	798
330	727	595	885
360	794	649	972

Figure 4.13 Charge-passed in RCPT Test of Mixtures in Group F

Figure 4.14 Charge-passed in RCPT Test of Mixtures in Group S

Figure 4.15 Comparison of Chloride Permeability Among Mixtures

Table 4.14 and Figure 4.13 show that among the Group F mixtures, F1 (current WVDOH) had the lowest quantity of charge passed over 6 hours, while F3 (HPC with less fly ash) had the highest value. The charge passed through F2 (HPC with more fly ash) was close (21% higher) to that recorded for F1.

Table 4.15 and Figure 4.14 indicate that among the Group S mixtures, S2 (HPC with more slag than the current WVDOH) had the lowest amount of charge passed over 6 hours, while S3 (HPC with less slag content) had the highest value. The corresponding values of S1 (current WVDOH) were found to lie in between S2 and S3.

A comparison among all the mixtures indicates that in general the range of chloride permeability (measured as charge passed) was lower for the slag concrete (649

to 972 Coulombs) than for the fly ash concrete (779 to 1310 Coulombs). The largest difference was between F3 and S3, and there was practically no difference between F1 and S1 (Figure 4.15).

In general all of the six HPC mixtures, except for F3, showed “very low” (ASTM C1202) chloride ion permeability based on charge passed. Ozyidirim (1999) reported about 900 Coulombs of charge using the same test, for 28-day concrete with 50% of slag by weight, and W/Cm ratio of 0.4; the specimens were moist-cured for one week at room temperature (73 °F) and subsequently three weeks at 38 °C (100 °F).

For a bridge project in Nebraska, Beacham (1999) reported that the HPC mixture they used with 10% of fly ash by weight at W/Cm ratio 0.31 had a chloride permeability of 589 Coulombs at 56 days. However, the curing condition of the specimens was different from the one used in the present study.

Chapter 5

Summary and Conclusions

This chapter summarizes the test results of the six HPC mixture designs developed in this study and draws some conclusions on the basis of the results. A recommendation for future work is also included.

5.1 Summary of Results

Based on the experiments and observations of the developed HPC mix proportions, the results are summarized as follows:

1. All of the mixtures except those with lower fly ash and slag contents (compared to the current WVDOH) were cohesive and free from bleeding during pouring and vibration of concrete.
2. The concretes with lower fly ash and slag were susceptible to bleeding and segregation under vibration.
3. The range of slump of all HPC mixtures, for both the fly ash and slag groups, was between 5.5 to 7 inches, which satisfies the WVDOH Class H concrete requirement. The slump was measured approximately 40 to 45 minutes after the addition of water to the cementitious materials.

4. The air content of all HPC mixtures, for both the fly ash and slag groups, was between 5% to 7%, which satisfies the WVDOH Class H concrete requirement. The slag concrete needed more AEA to maintain the air content.
5. The six HPC mixtures attained reasonably good compressive strengths, even at only 3 days of curing. The fly ash HPC attained 8% to 20% more 3-day strength than the slag HPC. However, at later ages (after 14 days), the slag HPC reached higher strength than the fly ash HPC. At 28 days, the slag HPC had 8% to 12% higher compressive strength than the corresponding fly ash HPC.
6. Among the fly ash HPC, the current WVDOH mixture had the highest compressive strength, and the HPC with lower fly ash content showed lowest compressive strength.
7. Among the slag HPC, the current WVDOH mixture had the highest compressive strength, and the HPC with lower slag content showed the lowest compressive strength.
8. All six HPC mixtures exhibited gains in strength for up to 90 days (the time-span for which it was measured) under continuous moist curing.
9. The drying shrinkage was more significant during the early periods of drying. About 72% to 75% of drying shrinkage occurred within the first 7 days of drying.
10. The best fit curve used for the fly ash HPC specimens indicates that the concrete with the lowest quantity of fly ash had the maximum shrinkage, and the current WVDOH mixture used as a base design had the lowest shrinkage.

11. The best fit curve used for the slag HPC specimens indicates that the current WVDOH mixture had the highest shrinkage, while the shrinkage values for the HPC with lower and higher slag contents were close to each other.
12. The freeze-thaw durability tests of 250 cycles for the fly ash and slag HPC mixtures did not alter significantly the dynamic modulus of elasticity values. The durability factors computed for the F1, F2, F3 and S3 mixtures show that all those HPC are highly durable, since these durability factors were between 96% and 99%. It is presumed that because of proper mixture proportioning with appropriate air content, the freeze-thaw cycles of up to 300 did not affect the concrete integrity.
13. The rapid chloride permeability test indicated that among the fly ash HPC, the current WVDOH mixture had the lowest chloride permeability value of about 780 Coulombs is the “very low” category according to ASTM C 1202. The permeability of HPC with high fly ash was about 21% more than of the current WVDOH mixture.
14. Among the slag HPC mixtures, the HPC with more slag showed the lowest chloride permeability value (about 18% lower than that of the WVDOH mixture). All the slag HPC mixtures had values within the “very low” category of chloride permeability.

5.2 Conclusions

Table 5.1 shows the grade of each of the six HPC mixtures for each of its properties, such as compressive strength, drying shrinkage, freeze-thaw, and chloride

permeability. The grading is made in accordance with the published document “High-Performance Concrete Defined for Highway Structures” (Goodspeed et al, 1996). The highest grade is 4, and conversely, the lowest grade is 1.

Table 5.1 Grades for Different Characteristics of All Mixtures

	HPC	56-day Compressive Strength	Drying Shrinkage	Durability	Chloride Permeability
Fly ash	Current WVDOH	Grade 2	Grade3	Grade2	Grade3
	More FA	Grade 2	Grade3	Grade2	Grade2
	Less FA	Grade 2	Grade3	Grade2	Grade2
Slag	Current WVDOH	Grade 3	Grade3	N/A	Grade3
	More SG	Grade 2	Grade3	Grade2	Grade3
	Less SG	Grade 3	Grade3	Grade2	Grade2

Note: N/A = not presently available

From the fly ash group, the current WVDOH and the higher fly ash mixtures are recommended for future studies. From the slag group, also the current WVDOH and the higher slag mixtures are recommended for future studies. Between the fly ash and slag groups, the overall performance of slag group was better. Among all the six HPC mixtures, the concrete with higher slag (35% slag) was the most cost effective.

In general, for the WV source of limestone and natural river sand aggregates used in this study, the current WVDOH Class H concretes, as proportioned in this study, are good if the cement factors are specified 624 for the fly ash group, and 647 for the slag group, and if the water/cementitious material ratio is maintained as 0.40 for all mixtures. The addition of more fly ash and slag to the corresponding WVDOH mixtures did not

affect the performance significantly, but by using a higher quantity of slag, the chloride permeability of the concrete improved. Thus, for future development of HPC mixtures for WV using other sources of aggregates, the mixture designs with higher contents of slag and fly ash (pozzolanic materials) may be recommended due to cost benefits and environmental considerations aimed at using pozzolanic materials as well as reducing CO₂ emissions during cement production.

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Appendix A: Compressive Strength Test Results

Table A.1 Mix F1 Compressive Strength (Units: psi)

Curing Time (Days)	Compressive Strength			Average	Variation
	Specimen 1	Specimen 2	Specimen 3		
1	3742	3662	3503	3636	122
3	5135	5255	5175	5188	61
7	6688	6449	6449	6529	138
14	7962	7484	7126	7524	419
28	8678	8376	8439	8498	160
56	9156	8758	9514	9143	378
90	9621	9477	9789	9629	156

Table A.2 Mix F2 Compressive Strength (Units: psi)

Curing Time (Days)	Compressive Strength			Average	Variation
	Specimen 1	Specimen 2	Specimen 3		
1	3304	3384	N/A	3344	56
3	4936	5135	5135	5069	115
7	5653	5772	6290	5905	339
14	6648	6887	6489	6675	200
28	8201	8201	7803	8068	230
56	8997	9236	8041	8758	632
90	9554	9315	9952	9607	322

Table A.3 Mix F3 Compressive Strength (Units: psi)

Curing Time (Days)	Compressive Strength			Average	Variation
	Specimen 1	Specimen 2	Specimen 3		
1	3662	3941	2667	3424	670
3	5454	5573	5215	5414	182
7	6648	6409	6210	6423	219
14	7205	7086	7524	7272	226
28	7564	7842	7803	7736	151
56	8360	8479	8519	8453	83
90	9475	9116	8997	9196	249

Table A.4 Mix S1 Compressive Strength (Units: psi)

Curing Time (Days)	Compressive Strength			Average	Variation
	Specimen 1	Specimen 2	Specimen 3		
1	2747	2747	2906	2800	92
3	4538	4618	4697	4618	80
7	6369	6568	6927	6622	282
14	7365	7301	8201	7622	502
28	8798	9475	9355	9209	361
56	9891	9981	N/A	9936	64
90	11306	11027	N/A	11166	197

Table A.5 Mix S2 Compressive Strength (Units: psi)

Curing Time (Days)	Compressive Strength			Average	Variation
	Specimen 1	Specimen 2	Specimen 3		
1	2389	2189	N/A	2289	141
3	4180	4260	4260	4233	46
7	6091	5971	6369	6144	204
14	7285	7126	7086	7166	105
28	8002	9076	9116	8731	632
56	8917	8917	9514	9116	345
90	9634	9634	9674	9647	23

Table A.6 Mix S3 Compressive Strength (Units: psi)

Curing Time (Days)	Compressive Strength			Average	Variation
	Specimen 1	Specimen 2	Specimen 3		
1	2468	2269		2369	141
3	5175	5175	4618	4989	322
7	6250	6449	6568	6423	161
14	8280	7564	7882	7909	359
28	8599	8121	9116	8612	498
56	9037	9793	9315	9382	383
90	9793	10350	9952	10032	287

Appendix B: Shrinkage Test Results

Table B.1 Mix F1 Shrinkage at Different Ages (Units: $\mu\epsilon$)

Time (Days)	Shrinkage			Average	Variation
	Spec 1	Spec 2	Spec 3		
0	0.0	0.0	0.0	0.0	0.0
4	184.0	152.0	120.0	152.0	26.1
7	256.0	224.0	216.0	232.0	17.3
14	264.0	296.0	296.0	285.3	15.1
21	360.0	336.0	312.0	336.0	19.6
28	336.0	328.0	328.0	330.7	3.8
35	304.0	264.0	248.0	272.0	23.6
42	336.0	296.0	288.0	306.7	21.0
49	328.0	296.0	280.0	301.3	20.0
56	336.0	312.0	296.0	314.7	16.4

Table B.2 Mix F2 Shrinkage at Different Ages (Units: $\mu\epsilon$)

Time (Days)	Shrinkage			Average	Variation
	Spec 1	Spec 2	Spec 3		
0	0.0	0.0	0.0	0.0	0.0
4	136.0	160.0	200.0	165.3	26.4
7	224.0	256.0	272.0	250.7	20.0
14	320.0	352.0	344.0	338.7	13.6
21	312.0	304.0	308.0	308.0	3.3
28	344.0	336.0	368.0	349.3	13.6
35	360.0	352.0	368.0	360.0	6.5
42	376.0	392.0	384.0	384.0	6.5
49	328.0	344.0	352.0	341.3	10.0
56	336.0	328.0	360.0	341.3	13.6

Table B.3 Mix F3 Shrinkage at Different Ages (Units: $\mu\epsilon$)

Time (Days)	Shrinkage			Average	Variation
	Spec 1	Spec 2	Spec 3		
0	0.0	0.0	0.0	0.0	0.0
4	216.0	176.0	192.0	194.7	16.4
7	272.0	320.0	272.0	288.0	22.6
14	312.0	368.0	352.0	344.0	23.6
21	336.0	392.0	368.0	365.3	22.9
28	304.0	384.0	352.0	346.7	32.9
35	344.0	392.0	384.0	373.3	21.0
42	344.0	448.0	440.0	410.7	47.3
49	368.0	416.0	400.0	394.7	20.0
56	328.0	400.0	376.0	368.0	29.9

Table B.4 Mix S1 Shrinkage at Different Ages (Units: $\mu\epsilon$)

Time (Days)	Shrinkage			Average	Variation
	Spec 1	Spec 2	Spec 3		
0	0.0	0.0	0.0	0.0	0.0
4	192.0	80.0	232.0	212.0	28.3
7	248.0	88.0	296.0	272.0	33.9
14	288.0	-64.0	304.0	296.0	11.3
21	344.0	192.0	368.0	356.0	17.0
28	352.0	224.0	384.0	368.0	22.6
35	384.0	96.0	400.0	392.0	11.3
42	344.0	120.0	368.0	356.0	17.0
49	352.0	184.0	368.0	360.0	11.3
56	328.0	168.0	344.0	336.0	11.3

Table B.5 Mix S2 Shrinkage at Different Ages (Units: $\mu\epsilon$)

Time (Days)	Shrinkage			Average	Variation
	Spec 1	Spec 2	Spec 3		
0	0.0	0.0	0.0	0.0	0.0
4	150.0	236.0	296.0	172.0	66.0
7	160.0	272.0	312.0	248.0	64.3
14	240.0	312.0	336.0	296.0	40.8
21	240.0	312.0	352.0	301.3	46.3
28	264.0	320.0	360.0	314.7	39.4
35	320.0	392.0	416.0	376.0	40.8
42	296.0	344.0	376.0	338.7	32.9
49	280.0	320.0	344.0	314.7	26.4
56	248.0	304.0	336.0	296.0	36.4

Table B.6 Mix S3 Shrinkage at Different Ages (Units: $\mu\epsilon$)

Time (Days)	Shrinkage			Average	Variation
	Spec 1	Spec 2	Spec 3		
0	0.0	0.0	0.0	0.0	0.0
4	168.0	128.0	136.0	144.0	17.3
7	224.0	192.0	176.0	197.3	20.0
14	352.0	328.0	296.0	325.3	22.9
21	352.0	336.0	328.0	338.7	10.0
28	352.0	328.0	288.0	322.7	26.4
35	392.0	368.0	320.0	360.0	29.9
42	368.0	344.0	304.0	338.7	26.4
49	360.0	296.0	288.0	314.7	32.2
56	352.0	328.0	280.0	320.0	29.9

Appendix C: Rapid Freeze/Thaw Test Results

Table C.1 Mix F1 Rapid Freeze/Thaw Test Results

Freezing /Thawing Cycles	Weight (lbs)		Length (inch)		Cross Section (inch ²)		Frequency (Hz)	
	1	2	1	2	1	2	1	2
Initial	16.31	16.63	16.00	16.00	3 X 4	3 X 4	1786	1803- 1814
50	16.31	16.63	16.00	16.00	3 X 4	3 X 4	1772	1797
100	16.33	16.63	16.00	15.97	2.97 x4	3 X 4	1780	1792
150	16.33	16.64	15.97	16.00	3.03 x 4	3.03 x 4	1786	1813
200	16.31	16.62	16.00	16.00	3 x 4	3.03 x 4	1794	1803
250	16.32	16.62	15.97	16.00	2.97 x 4	3.06 x 4	1799	1827
300	16.29	16.60	15.94	15.93	3 x 4	3 X 4	1792	1819
350	16.24	16.59	16.00	15.97	3 x 4	3 X 4	1787	1818

Note: First 50-cycle test starts at 14 days after initial reading

Table C.2 Mix F2 Rapid Freeze/Thaw Test Results

Freezing /Thawing Cycles	Weight (lbs)		Length (inch)		Cross Section (in ²)		Frequency (Hz)	
	1	2	1	2	1	2	1	2
Initial	17.04	16.91	16.00	16.00	3 x4	3 x4	1810	1816
50	16.89	17.02	16.00	16.03	3 x4	3 x4	1812	1818
100	16.88	17.00	16.00	16.03	3 x 4	3 x 4	1823	1827
150	16.91	17.00	16.00	16.00	3.03 x 4	3 x 4	1820	1819
200	16.89	16.96	16.00	15.93	3.03 x 4	3 x 4	1820	1835
250	16.86	16.89	16	16	3 x 4	3 x 4	1825	1830
300	16.85	16.88	16	16	3.03 x 3.97	2.97 x 4	1828	1838

Note: First 50-cycle test starts at 14 days after initial reading

Table C.3 Mix F3 Rapid Freeze/Thaw Test Results

Freezing /Thawing Cycles	Weight (lbs)		Length (inch)		Cross Section (inch ²)		Frequency (Hz)	
	1	2	1	2	1	2	1	2
Initial	16.47	16.99	16.00	16.00	3 x 4	3 x 4	1780	1820
50	N/A	16.99	N/A	15.97	N/A	3 x 4	N/A	1807
100	16.49	16.98	16.06	16.00	3.03 x 4	3 x 4	1760	1819
150	16.48	17.00	16.06	16.00	3.03 x 4	3 x 4.03	1771	1825
200	16.48	16.98	16.03	16.00	3.03 x 4	3 x 4	1769	1825
250	16.48	16.96	16.00	16.00	3 x 4	3 x 4	1773	1832
300	16.45	16.93	16.03	15.97	3 x 4	3.06x4	1779	1843

Note: First 50-cycle test starts at 14 days after initial reading

Table C.4 Mix S1 Rapid Freeze/Thaw Test Results

Freezing /Thawing Cycles	Weight (lbs)		Length (inch)		Cross Section (inch ²)		Frequency (Hz)	
	1	2	1	2	1	2	1	2
Initial	16.92	17.07	16.00	16.00	3 x 4	3 x 4	1814	1837
50	16.90	17.06	16.00	16.00	3 x 4	3.06 x 4	1815	1839
100	16.90	17.06	15.97	16.00	3.03 x 4	3.06 x 4	1797	1830
150	16.92	17.07	16.00	16.00	3.06 x 4	3.06 x 4	1818	1845
200	16.86	17.04	16.00	16.00	3.03 x 4	3.06 x 4	1822	1846
250	16.84	17.03	16.00	16.00	3.03 x 4	3.06 x 4.03	1816	1853
300	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Note: First 50-cycle test starts at 14 days after initial reading

Table C.5 Mix S2 Rapid Freeze/Thaw Test Results

Freezing /Thawing Cycles	Weight (lbs)		Length (inch)		Cross Section (inch ²)		Frequency (Hz)	
	1	2	1	2	1	2	1	2
Initial	16.68	16.43	16.00	16.00	3 x 4	3 x 4	1780	1792
50	16.68	16.44	15.97	15.97	3 x 4.03	3.03 x 4	1784	1797
100	16.68	16.44	16.00	16.00	3.03 x 4.03	3.03 x 4	1773	1779
150	16.69	16.45	16.00	16.00	3.03 x 4	3 x 4	1788	1794
200	16.69	16.42	16.00	16.00	3 x 4	3.06 x 4	1789	1801
250	16.65	16.37	16.00	16.00	3 x 4	3 x 4	1784	1801
300	16.65	16.35	16.00	16.00	3 x 4.03	3 x 4	1795	1806

Note: First 50-cycle test starts at 14 days after initial reading

Table C.6 Mix S3 Rapid Freeze/Thaw Test Results

Freezing /Thawing Cycles	Weight (lbs)		Length (inch)		Cross Section (inch ²)		Frequency (Hz)	
	1	2	1	2	1	2	1	2
Initial	16.54	16.48	16.00	16.00	3 x 4	3 x 4	1810	1795
50	16.49	16.53	16.00	16.00	3 x 4	3 x 4	1779	1790
100	16.51	16.53	16.00	16.00	3 x 4	3.03 x 4	1799	1805
150	16.52	16.56	16.00	16.00	3.06 x 4	3.03 x 4	1789	1786
200	16.49	16.53	16.00	16.00	3 x 4	3 x 4	1797	1806
250	16.49	16.49	16.00	16.00	3.06 x 4	3 x 4	1798	1816
300	16.44	16.50	16.00	16.00	3.03 x 4	3 x 4	1786	1814

Note: First 50-cycle test starts at 14 days after initial reading

Appendix D: 42-Day RCPT Test Results

Table D.1 Mix F1 42-Day Permeability Test Results

	Specimen 1		Specimen 2	
Time	Current	Charge	Current	Charge
(Min.)	(AMP)	(Coulombs)	(AMP)	(Coulombs)
0	0	0	0	0
30	0.032	63	0.030	56
60	0.033	123	0.031	113
90	0.034	185	0.032	172
120	0.035	249	0.033	233
150	0.036	314	0.033	294
180	0.036	381	0.034	357
210	0.038	449	0.034	421
240	0.038	518	0.035	485
270	0.039	588	0.036	551
300	0.039	659	0.037	618
330	0.040	730	0.037	686
360	0.040	803	0.038	755

Table D.2 Mix F2 42-Day Permeability Test Results

	Specimen 1		Specimen 2	
Time	Current	Charge	Current	Charge
(Min.)	(AMP)	(Coulombs)	(AMP)	(Coulombs)
0	0	0	0	0
30	0.038	68	0.036	64
60	0.039	140	0.037	131
90	0.041	214	0.038	200
120	0.043	292	0.038	271
150	0.045	372	0.039	343
180	0.046	456	0.040	417
210	0.048	542	0.041	491
240	0.050	630	0.041	567
270	0.051	721	0.043	645
300	0.052	814	0.043	723
330	0.053	910	0.044	803
360	0.056	1008	0.046	885

Table D.3 Mix F3 42-Day Permeability Test Results

	Specimen 1		Specimen 2	
Time	Current	Charge	Current	Charge
(Min.)	(AMP)	(Coulombs)	(AMP)	(Coulombs)
0	0	0	0	0
30	0.049	86	0.053	93
60	0.051	177	0.056	193
90	0.054	273	0.059	297
120	0.056	372	0.06	404
150	0.058	475	0.063	516
180	0.059	581	0.064	630
210	0.061	689	0.065	747
240	0.061	799	0.066	865
270	0.063	912	0.069	987
300	0.063	1025	0.069	1110
330	0.064	1140	0.07	1236
360	0.065	1256	0.071	1363

Table D.4 Mix S1 42-Day Permeability Test Results

	Specimen 1		Specimen 2	
Time	Current	Charge	Current	Charge
(Min.)	(AMP)	(Coulombs)	(AMP)	(Coulombs)
0	0	0	0	0
30	0.029	53	0.039	70
60	0.030	108	0.041	144
90	0.030	164	0.042	219
120	0.031	221	0.042	295
150	0.031	279	0.042	371
180	0.032	337	0.041	448
210	0.031	395	0.041	524
240	0.031	453	0.042	601
270	0.032	510	0.042	677
300	0.031	567	0.042	753
330	0.031	624	0.042	830
360	0.031	681	0.042	906

Table D.5 Mix S2 42-Day Permeability Test Results

	Specimen 1		Specimen 2	
Time	Current	Charge	Current	Charge
(Min.)	(AMP)	(Coulombs)	(AMP)	(Coulombs)
0	0	0	0	0
30	0.031	55	0.026	48
60	0.031	112	0.027	98
90	0.031	169	0.027	148
120	0.031	226	0.027	198
150	0.031	284	0.027	249
180	0.031	343	0.028	300
210	0.032	402	0.027	350
240	0.032	461	0.027	401
270	0.033	520	0.027	451
300	0.031	579	0.027	501
330	0.032	638	0.027	551
360	0.032	697	0.027	600

Table D.6 Mix S3 42-Day Permeability Test Results

	Specimen 1		Specimen 2	
Time	Current	Charge	Current	Charge
(Min.)	(AMP)	(Coulombs)	(AMP)	(Coulombs)
0	0	0	0	0
30	0.044	79	0.034	61
60	0.046	162	0.035	126
90	0.048	248	0.036	192
120	0.05	338	0.037	259
150	0.051	430	0.037	327
180	0.052	524	0.037	396
210	0.053	621	0.038	465
240	0.055	718	0.039	535
270	0.055	818	0.038	606
300	0.056	918	0.039	677
330	0.056	1020	0.039	749
360	0.057	1122	0.04	821