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Final Report

**HIGH-PERFORMANCE CONCRETE BRIDGE
DECKS: A FAST-TRACK IMPLEMENTATION
STUDY**

Volume 2: Materials

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Introduction

Concrete bridge decks are inherently exposed to harsh environment. They are known to undergo degradation due to numerous distresses, of which the most common are corrosion of reinforcement (caused by ingress of chloride through bulk concrete), freeze-thaw damage, shrinkage cracking (aggravating both corrosion and freeze-thaw problems) and surface scaling due to deicing salts.

Recently, high performance concrete (HPC) has become widely utilized in the applications where severe environment leads to premature deterioration. The premise behind the development of HPC was to address the durability issues typical of normal concrete. HPC almost always implies incorporation of mineral admixtures, such as fly ash (FA) or silica fume (SF). These materials are added to concrete for variety of reasons, of which the most important is improvement (compared to plain ordinary portland cement (OPC) concrete) of certain durability-related properties. However, there are also some downsides of utilizing them in concrete. For example, fly ash is known to reduce the early age strength and early age resistance to chloride-ion penetration, as well as resistance to salt scaling and carbonation. Furthermore, fly ash concrete has been reported to be fairly sensitive to curing conditions compared to plain cement concrete. The shortcomings of incorporating silica fume, on the other hand, include increased susceptibility to shrinkage cracking and potentially reduced resistance to freezing and thawing.

In the view of the aforementioned side effects associated with the use of fly ash or silica fume in binary mixtures, the ternary cementitious systems have been introduced as a potentially viable solution to address those durability issues. Superior properties of this kind of cementitious systems are

commonly attributed to so-called synergistic effect taking place when both pozzolanic materials are utilized. The intuitively obvious benefit of the use of OPC/FA/SF mixtures is that the presence of fly ash in concrete compensates for the deficiencies of silica fume, and vice versa. For example, FA offsets the increase in water demand, heat of hydration and alkalinity of pore solution resulting from addition of SF. On the other hand, SF compensates for the low early age strength and sensitivity to curing (with respect to development of both mechanical and durability-related properties of FA concrete). From the economical perspective, a relatively low cost of FA counterbalances higher cost of SF. Although a fairly large amount of research has been conducted to evaluate the properties of ternary cementitious mixtures containing fly ash and silica fume, relatively little was done with the emphasis on the durability-related issues encountered when these binder systems are used in the bridge deck concrete. Up to date, no data exists on the actual performance of ternary concrete containing class C fly ash and silica fume in bridge decks.

The purpose of this research was to examine the applicability of ternary binder systems containing ordinary portland cement, class C fly ash and silica fume for bridge deck concrete. This was accomplished in two parts, the laboratory part and a field application part.

During the laboratory studies, four ternary mixtures, each containing either 20% or 30% FA and either 5% or 7% SF were subjected to four different curing regimes (air drying, 7 days curing compound application and 3 or 7 days wet burlap curing). The properties studied included water and chloride solution controlling properties, compressive strength, free shrinkage and

resistance to shrinkage cracking, salt scaling and freezing-thawing resistance.

The field application part of the research was conducted on the bridge carrying SR-23 over US-20 in South Bend, Indiana. This bridge deck was constructed in two phases, one constructed in

late fall 2004 and the other constructed during spring 2005. Test specimens were collected for laboratory evaluation from both construction phases and visual observations of the deck performance were conducted at 6, 10 and 24 months after construction.

Findings

During the laboratory part of the study it was observed that all four ternary mixtures exhibited very good water and chloride solution transport-controlling properties (resistance to chloride-ion penetration, chloride diffusivity and rate of water absorption). However, it was concluded that in order to ensure adequate strength, good freezing and thawing resistance, satisfactory resistance to salt scaling, and adequate shrinkage cracking resistance the FA content should not exceed 20%, SF content should not exceed 5% (by total mass of binder) and paste content should be kept below 24% by volume of concrete. Further, wet burlap curing for a minimum of 3 days was required to achieve satisfactory performance and to obtain a reliable assessment of in-situ compressive strength (up to 28 days) using maturity method.

The properties developed by field concrete from construction phase 2 (which took place in spring of 2005) were comparable to those of concrete which was made and cured under laboratory conditions. However, the chloride-ion penetrability (evaluated using rapid chloride permeability (RCP test) of concrete from phase 1 (which took place in fall of 2004) remained

relatively high for prolonged period of time (4000 coulombs at 90 days). Using maturity method developed for the purpose of this study, it was determined that the unexpectedly high RCP values were mostly attributed to low ambient temperature. Additional applications of the developed maturity method were also demonstrated, which include assessment of risk of scaling and reduction in time to corrosion initiation as a function of construction date as well as estimation of long-term RCP values for concrete subjected to accelerated curing.

The visual inspections of the bridge indicated that the Phase 1 deck underwent moderate surface scaling. This was likely the result of low ambient temperatures the concrete was exposed to shortly after placement and early application of deicing salts which took place within a month after construction. The Phase 2 deck experienced some transverse cracking, which was likely due to additional restraint at the construction joint and higher thermal stresses resulting from elevated ambient temperature.

Implementation

The main purpose of this research was to evaluate applicability of ternary binder systems incorporating ordinary portland cement, fly ash and silica fume for bridge deck concrete. In general, it was concluded that OPC/FA/SF concrete is capable of developing a unique combination of properties, such as very low chloride-ion diffusivity and water absorptivity, good resistance to salt scaling and shrinkage cracking, satisfactory strength and satisfactory resistance to freezing and thawing. However, the following issues are crucial to successful application of OPC/FA/SF concrete in bridge decks:

1. Mixture composition: fly ash content should be 20%, silica fume content should not exceed 5% (by total mass of binder) and paste content should not exceed 24%.

2. Curing conditions: wet burlap curing for a minimum of 3 days (at temperature of 23°C) is required. If the temperature is lower, wet burlap curing period needs to be extended.
3. Calendar restrictions: In order to minimize scaling problem, calendar restrictions should be specified for the use of ternary concrete in bridge decks constructed in the late fall season. Alternatively, contractor should be required to extend the moist curing period to 14 days or to use heating blankets in order to maintain higher temperature essential for hydration of slower reacting supplementary cementitious materials.
4. Construction practices: concrete should not be placed in bridge decks using a pump unless contractor can prove during the preconstruction trial pumping that placement of concrete using pump can be achieved

without altering not only the air content but also the air-void system parameters. To minimize the propensity of ternary concrete to cracking, it is recommended that the time-span of two-phased projects be limited to minimum. Also, the restraint at the

construction joint between the individual phases should be minimized. Finally, placement of concrete should take place in moderate temperatures to minimize the thermal shrinkage stresses.

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FINAL REPORT

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High-Performance Concrete Bridge Decks: A Fast-Track Implementation Study

Volume 2 Materials

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CHAPTER 1 – INTRODUCTION

1.1 Problem Statement

Concrete bridge decks are inherently exposed to harsh environment. They are known to undergo degradation due to numerous distresses, of which the most common are corrosion of reinforcement (caused by ingress of chloride through bulk concrete), freeze-thaw damage (Russel 2004, Xi et al. 2003), shrinkage cracking (aggravating both corrosion and freeze-thaw problems) and surface scaling due to deicing salts.

Recently, high performance concrete (HPC) has become widely utilized in the applications where severe environment leads to premature deterioration. The premise behind the development of HPC was to address the durability issues typical of normal concrete. HPC almost always implies incorporation of mineral admixtures, such as fly ash (FA) or silica fume (SF). These materials are added to concrete for variety of reasons, of which the most important is improvement (compared to plain ordinary portland cement (OPC) concrete) of certain durability-related properties (Blomberg 2003, Bouzoubaâ and Fournier 2005, Hassan et al. 2000, Nikam and Tambvekar 2003). However, there are also some downsides of utilizing them in concrete. For example, fly ash is known to reduce the early age strength (Carette and Malhotra 1983, Sarkar et al. 1991, Thomas and Bamforth 1999) and early age resistance to chloride-ion penetration, as well as resistance to salt scaling (Marchand et al. 1992) and carbonation (Sulapha et al. 2003). Furthermore, fly ash concrete has been reported to be fairly sensitive to curing conditions compared to plain cement concrete (Bentz 2002, Bouzoubaâ et al. 2004, Khan and Ayers 1993, Ramezani pour and Malhotra 1995). The shortcomings of incorporating silica fume, on the other hand, include increased susceptibility to shrinkage cracking (Cohen et al., Shah and Weiss 2000, Whiting et al. 2000), and potentially reduced resistance to freezing and thawing (Palecki and Setzer 2006, Pigeon et al. 1986).

In the view of the aforementioned side effects associated with the use of fly ash or silica fume in binary mixtures, the ternary cementitious systems have been introduced as a potentially viable solution to address those durability issues. Superior properties of this kind of cementitious systems is commonly attributed to so-called synergistic effect taking place when both pozzolanic materials are utilized (Bouzoubaâ et al. 2004, Mehta and Gjrv 1982, Popovics 1993, Khan 2003a, Sarkar et al. 1991, Shehata and Thomas 2002, Thomas and Bamforth 1999). The intuitively obvious benefit of the use of OPC/FA/SF mixtures is that the presence of fly ash in concrete compensates for the deficiencies of silica fume, and vice versa. For example, FA offsets the increase in water demand, heat of hydration (Thomas and Bamforth 1999) and alkalinity of pore solution (Shehata and Thomas 2002) resulting from addition of SF. On the other hand, SF compensates for the low early age strength (Carette and Malhotra 1983, Khan and Lynsdale 2002, Thomas and Bamforth 1999) and sensitivity to curing (with respect to development of both mechanical and durability-related properties) of FA concrete (Bouzoubaâ et al. 2004). From the economical perspective, a relatively low cost of FA counterbalances higher cost of SF.

It should be recognized at this point that bridge deck concrete needs to exhibit a wide range of certain characteristics to be considered suitable for such applications. The desirable properties of bridge deck concrete include satisfactory strength development, as well as high resistance to chloride-ion penetration, shrinkage cracking, salt scaling and freeze-thaw action. However, the large number of specific properties that bridge deck concrete ought to display is not the only challenge. The situation is further complicated by the fact that some of most of these properties are related to one another and achieving some of is often in be direct conflict with others. For instance, high resistance to chloride-ion penetration is usually achieved by using relatively low water-binder ratio and by incorporation of supplementary cementitious materials (SCMs) in the mixture. This, however, typically results in high (often excessive) levels of elastic modulus, increased shrinkage (particularly autogenous) and low creep. All of these three side-effects unfortunately imply increased potential to shrinkage cracking. It is, therefore, imperative to select the optimum composition of

mixture and proportions of constituents to ensure that the aforementioned desired characteristics of concrete are attainable.

Although a fairly large amount of research has been conducted to evaluate the properties of ternary cementitious mixtures containing fly ash and silica fume, relatively little was done with the emphasis on the durability-related issues encountered when these binder systems are used in the bridge deck concrete. Up to date, no data exists on the actual performance of ternary concrete containing class C fly ash and silica fume in bridge decks.

1.2 Objectives

While drafting the reconstruction plans for the west part of I-465 around Indianapolis, the Indiana Department of Transportation (INDOT) put in place a requirement that all bridges included in this project should have an extended service life and only require minimal maintenance. The primary objective of this research was to develop a mixture design to be used on I-465 bridges (and ultimately on other bridges elsewhere in the state) to achieve the enhanced performance. As a part of this project, the performance of candidate ternary mixtures containing fly ash and silica fume has been evaluated through a series of laboratory tests and a field study on the pilot HPC bridge on SR 23 near South Bend, Indiana. Volume 1 of SPR-2911 report (Frosch and Aldridge 2006) deals with structural behavior of HPC bridge decks, whereas this volume contains the findings of both field and laboratory research conducted to investigate the materials-related aspects of high performance concrete for bridge decks.

The initial composition of the mixtures selected for this project was based on the results of the previous study (SPR-2325) (Olek et al. 2002). Although that study identified the feasible ranges of several supplementary cementitious materials, namely, class C fly ash, silica fume and ground granulated blast furnace slag, to be used either on its own or in a ternary system, the focus of the current research was solely on the use of ternary concrete containing fly ash and silica fume (OPC/FA/SF). Also, the impact of curing quality on several mechanical and durability-related properties has been assessed.

1.3 Scope and Organization of the Report

This report consists of six chapters. Chapter 1 contains problem statement and describes the objectives and scope of the report. Chapter 2 summarizes major findings from review of literature concerning the use of ternary systems containing fly ash and silica fume.

Chapters 3, 4 and 5 constitute the main body of the report. Chapter 3 contains a preliminary optimization analysis undertaken primarily to establish whether the ranges of fly ash and silica fume used in ternary cementitious system are feasible. In addition, the influence of fresh concrete properties (slump and air content) and source of raw materials on several properties was examined.

Chapter 4 describes multiple, in-depth laboratory studies which were mainly focused on evaluation of effect of mixture composition and curing conditions on several properties of the ternary mixtures relevant to bridge decks applications. These included: transport properties, scaling resistance, strength development, shrinkage, resistance to shrinkage cracking and freeze-thaw resistance.

Chapter 4 also contains the examination of applicability of maturity method for strength estimation of high performance concrete subjected to different curing regimes. It should be noted that the approach taken in the current study was different from that used in majority of optimization studies published to date (for instance, Bajorski and Streeter 2000, Ghezal and Khayat 2002, Nehdi and Sumner 2002), which included a wide range of mixture compositions, but were focused on rather limited number of mixture properties that were evaluated. As previously mentioned, in this work, a wide range of concrete properties was characterized, but for relatively limited number of mixtures. Specifically, four ternary mixtures with water-binder ratio of 0.41 containing 20% or 30% fly ash and 5% or 7% silica fume (by total mass of binder) were primarily studied. This relative tight range of mixture-related experimental variables has been determined as a result of recent research conducted by Olek et al. (2002), whose objective was to identify feasible ranges of supplementary cementitious materials in binary and ternary HPC mixtures.

Chapter 5 contains the results of research related to construction of the pilot bridge deck on SR 23 near South Bend, IN using high performance concrete

mixtures. Provided in that chapter are the general information regarding the mixture design, construction practices and curing and testing methods. This is followed by presentation of the results of tests carried out on concrete of the same nominal composition sampled during both phases of construction of the bridge deck as well as that made during the trial batch demonstration and independently in the laboratory. Subsequent section of chapter 5 deals with failed material investigation which involved testing of concrete cores obtained from the bridge deck constructed during Phase 1 and laboratory experiments conducted to determine the possible reasons for the air loss due to pumping encountered during construction Phase 1. Next, the observations from visual inspection of the bridge deck conducted approximately two years after construction are described. The final part of this chapter contains the results of study on the utilization of maturity method for prediction of resistance to chloride-ion penetration and possible applications of the developed methodology for quality monitoring purposes.

Finally, chapter 6 contains the summary of findings from the research performed within the scope of this project and the recommendations regarding materials-related issues of high performance concrete to be used for bridge decks. The provided recommendations, once implemented in the respective specifications, should lead to broader utilization of more durable concrete. This, in turn, should yield a substantial increase in the service life of bridge decks constructed in the state.

The last section of this report contains the list of all references used in this research. Also, attached is an appendix which is supplementary to the literature review included in chapter 2.

CHAPTER 2 – LITERATURE REVIEW

Volume 1 of this report (Frosch and Aldridge 2006) provides an up-to-date review of the literature related to the use of high performance (HPC) concrete in general, as well as specifically for bridge decks. The attempt of the literature study conducted in connection with the materials-related part of SPR-2911 (presented in this volume of the report) was to provide a state of the art-of-the-art review of the use of ternary mixtures with fly ash and silica fume. However, this chapter only summarizes the major findings from the extensive literature review. The appendix contains the main experimental details and research variables used by authors of the publications reviewed.

Total of fifty publications reporting the results of studies on the OPC/FA/SF cementitious systems were found (see Appendix), however two of them (Bouzoubaâ and Fournier 2005, Magee and Olek 2000) only report the results of survey, rather than laboratory tests data. It should be noted that only publications in English have been considered in this review.

The authors of majority of papers did not have any specific application of evaluated concrete mixtures in mind. Of all papers reviewed, the authors of ten reported the results of research driven by potential consideration of utilization of ternary concrete with fly ash and silica fume specifically for bridge decks. Other applications considered include transportation structures (in general), high strength concrete, fiber-reinforced shotcrete, structural lightweight concrete, massive concrete for hydropower station and roller-compacted concrete pavements.

Fig. 2.1 presents the number of papers on OPC/FA/SF mixtures published during the period 1982-2008. The earliest publication on this topic appeared in 1982 (Mehta and Gjrv 1982). It can be seen that there has been an increasing interest in simultaneous incorporation of fly ash and silica fume in the mixtures for general purposes

in the last decade. The use of ternary (OPC/FA/SF) mixtures for bridge decks started around 1992.

Fig. 2.1 – Number of papers on OPC/FA/SF mixtures published during the time period 1982-2008

Although most (thirty five) publications report the data on concrete mixtures, the information on mortar and paste mixtures was also quite common. Since most of the literature studies included obviously more than one OPC/FA/SF mixture, the total number of mixtures for which the data were gathered is over 300. This number constitutes an excellent database, which may prove very useful for the purposes of identification of what mixtures composition (in terms of materials type and relative proportions) have been previously evaluated, and what properties have already been studied. However, the detailed mixture composition and materials used have not been available in all referenced publications (see Appendix).

Regarding the materials used by the researchers, it was observed that in vast majority of the studies, ordinary Type I cement was used. Also in most (thirty six) cases published, class F fly ash was incorporated. The use of class C fly ash was reported in twelve cases only. Silica fume was used in variety of forms most commonly being dry,

densified material. In five papers the use of silica fume in the form of slurry was reported, whereas two cases mentioned the use of undensified and preblended silica fume.

Regarding the mixture proportions used, the frequency of using various values of water-cementitious materials ratio (w/cm), total binder content and FA and SF content are given in the form of histograms shown in Fig. 2.2-Fig. 2.5, respectively. These figures provide a very useful insight into what ranges of ternary mixture composition have been previously considered by numerous researchers. The information on application specifically in the bridge decks have been also provided.

It can be seen that although water-cementitious materials ratio ranged from 0.25 to 0.85, for vast majority of cases it was within a relatively tight range of 0.30-0.45 (Fig. 2.2). The range of w/cm used in the mixtures considered specifically for bridge decks was 0.30-0.45 but in most cases w/cm of 0.40-0.45 was chosen.

Fig. 2.2– Frequency of given w/cm used for ternary OPC/FA/SF mixtures

The total cementitious materials content of OPC/FA/SF mixtures reported in the literature ranged from 200 to 750 kg/m³, although the amount selected most frequently was from 350 to 450 kg/m³ (Fig. 2.3). Interestingly, the distribution is less symmetric and is skewed toward the higher values for bridge deck applications. The most commonly used cementitious materials content was about 400 kg/m³.

Fig. 2.3 – Frequency of given binder content used for ternary OPC/FA/SF mixtures

Fly ash (FA) content considered in the literature varied from 5% to 60% (by mass of binder), although in most cases it ranged from 10% to 30% (Fig. 2.4). For bridge decks, the range of fly ash content was from 5% to 40%, with 20% being the most prevailing value.

Fig. 2.4 – Frequency of given fly ash content used for ternary OPC/FA/SF mixtures

Silica fume (SF) content of the mixtures reported in the literature varied from 2% to 26% (by mass of binder), while most of the mixtures contained 4-10% (Fig. 2.5). The silica fume content of the mixtures considered specifically for bridge decks, varied from 4% to 16%, with the most frequently reported amount being 4-6%.

Fig. 2.5 – Frequency of given silica fume content used for ternary OPC/FA/SF mixtures

The mean values of the ternary mixture compositions reported in the literature reviewed are summarized in Table 2.1. It is interesting to note that the mean total cementitious materials used for the concretes considered as candidate mixtures for bridge decks is slightly higher than that for all concrete mixtures. Nonetheless, both of these numbers (401 kg/m^3 and 385 kg/m^3) are higher than the entire range of total cementitious materials content used in the present study ($308\text{-}367 \text{ kg/m}^3$), as will be discussed in Chapters 3 and 4. This implies that the mixtures studied in the present research may prove to be more economical than most of the mixtures encountered in the literature studies. However, all other parameters (FA and SF content and w/cm) selected in the current study appear to fit well within commonly used values, as in this study FA content varies from 20% to 30%, SF content varies from 5% to 7%, whereas w/cm used is 0.41. It will be also shown that, based on the results of the current study the most

desirable mixture for bridge deck applications should include 20% FA and 5% SF. Coincidentally, the mean FA and SF content of bridge deck concretes considered in the literature is 19% and 6%, respectively (Table 2.2).

Table 2.1 – Mean values of ternary mixture compositions reported in the literature

Case	Total Cementitious Materials (kg/m^3)	FA (%)	SF (%)	w/cm
All	-	23	8	0.40
Concretes	385	22	7	0.40
Bridge Deck Concretes	401	19	6	0.39

As a result of research conducted by some authors, certain recommendations regarding the optimum content of fly ash and silica fume have been established with respect to the properties that were evaluated. Khan (2003a) showed that 8-12% SF by mass of binder yielded the lowest permeability and lowest total permeable porosity, irrespectively of FA content. In other study the same author concluded that with respect to permeability, porosity and sorptivity the optimum level of FA content was in the range of 15-20% for low water-binder ratio mixtures with 10% SF (Khan 2003b). Thomas et al. (1999) suggested that combination of relatively small levels of silica fume (3 to 6%) and moderate levels of high CaO fly ash (20 to 30%) were the most efficient in reducing expansion due to alkali-silica reaction (ASR). In contrast, when fly ash was used in the binary system, 50% FA was required to control expansion due to ASR. However, it should be noted that high CaO fly ash is typically not recommended for ASR expansion control.

In the researched aimed at development of mixture for bridge decks, Xi et al. (2001) proposed the following optimal ranges for concrete design parameters: 276-288 kg/m^3 of cement, 4% silica fume, 20-25% of class F fly ash and w/cm between 0.37 and 0.41. Similarly, Nehdi and Sumner (2002) found that when rheological, mechanical, durability and cost requirements are combined, the use of SF in ternary OPC/FA/SF is not economic beyond levels of about 3-5%. However, the optimization analysis also indicated that, from the standpoint of properties evaluated, the presence of fly ash in the ternary system with silica fume was not desired. As a result of research undertaken to

develop optimized HPC mixture for bridge superstructures, Olek et al. (2002) recommended incorporation of 5-7% SF and 25-30% FA. Based on the results of the second phase of the same study, the optimum binder composition comprised 25% FA and 6% SF. In the optimization analysis performed by Lawler et al. (2005), the mixture containing 15% class F fly ash and 5% SF displayed very promising performance and ranked slightly higher than the mixture with 25% class C FA and 5% SF. The mixture with 40% class C fly ash and 8% SF exhibited similar performance, except for lower resistance to salt scaling.

Table 2.2 summarizes all tests performed in the literature studies reviewed for the purpose of this report. These tests have been grouped into four categories. Vast majority (thirty seven cases) of the studies were focused on mechanical properties of concrete. In twenty two out of forty eight studies (46%), the authors attempted to characterize the transport-related properties of concrete using one of the common test methods. The durability-related parameters of ternary systems (other than the ones associated with transport phenomena) were investigated in 40% of all references reviewed. Finally, fifteen (31%) studies included microstructural examination.

These percentages give a valuable insight into the general focus of existing research on ternary systems with fly ash and silica fume. However, within the durability group (Table 2.2) there are several parameters of concrete which require, in author's opinion, additional studies. For example, there were only six studies in which freeze-thaw resistance and salt scaling resistance were assessed. Likewise, only three studies included evaluation of resistance to shrinkage cracking. Similarly, it is felt that insufficient attention has been given to evaluation of microstructure and some transport-related parameters, such as chloride diffusion and rate of water absorption.

Table 2.2 – Summary of tests performed by the authors of literature reviewed for this study

Group of Tests	Test	No. of Cases	Total Number
Microstructural	SEM	4 (8%)	15 (31%)
	TEM	1 (2%)	
	MIP	5 (10%)	
	XRD	5 (10%)	
	TGA/DTA	7 (14%)	
	PSD	3 (6%)	
	IR Spectroscopy	1 (2%)	
Mechanical	Compressive strength	36 (75%)	37 (77%)
	Splitting-tensile strength	5 (10%)	
	Flexural strength	5 (10%)	
	Modulus of elasticity	8 (17%)	
	Creep	2 (4%)	
	Free drying shrinkage	9 (19%)	
	UPV	1 (2%)	
	Fracture properties	1 (2%)	
Durability (transport-related)	Chloride diffusion	1 (2%)	22 (46%)
	Chloride ponding	2 (4%)	
	RCP	12 (25%)	
	RMP	1 (2%)	
	Chloride conductivity	1 (2%)	
	Rate of absorption	2 (4%)	
	ISAT	2 (4%)	
	Water permeability	2 (4%)	
	Oxygen permeability	3 (6%)	
	CO ₂ gas permeability	1 (2%)	
	Permeable porosity	3 (6%)	
Durability (non-transport related)	F-T resistance	6 (13%)	19 (40%)
	Salt scaling resistance	6 (13%)	
	Restrained shrinkage resistance	3 (6%)	
	Plastic shrinkage resistance	4 (8%)	
	Autoclave expansion	1 (2%)	
	ASR resistance	2 (4%)	
	Sulfate resistance	2 (4%)	
	Carbonation resistance	2 (4%)	
	Calorimetry	3 (6%)	
	Abrasion resistance	1 (2%)	
	Air-void system analysis	2 (4%)	

To sum up, the reviewed literature suggests that using ternary OPC/FA/SF systems is very beneficial from the standpoint of various properties. In almost all cases reported such ternary systems outperformed both binary mixtures with either fly ash or silica fume alone, or the plain OPC concrete mixtures (Bouzoubaâ et al. 2004, Khan

2003a, Mehta and Gjrv 1982, Nikam and Tambvekar 2008, Ozyildirim and Halstead 1994, Popovics 1993, Shehata and Thomas 2002, Stoitchkov et al. 1996, Streeter 1996, Yařar 2004). From the perspective of the properties particularly critical for bridge decks applications, ternary concrete with fly ash and silica fume appears to exhibit excellent resistance to chloride-ion penetration (Blomberg 2003, Bouzouba et al. 2004, Jones et al. 1997, Olek et al. 2002, Olek et al. 2003, Thomas and Bamforth 1999, Suksawang et al. 2005a), good resistance to salt scaling (Blomberg 2003, Lawler et al. 2005, Olek et al. 2002, Streeter 1996, Suksawang et al. 2005a) and good resistance to freezing and thawing (Blomberg 2003, Lawler et al. 2005, Olek et al. 2002). Furthermore, the elastic modulus and its rate of increase with time for concrete with 5% fly ash and 20% silica fume was found to be lower than that of control OPC concrete (Nassif et al. 2005), which is beneficial from the standpoint of shrinkage cracking resistance.

Despite the above mentioned advances in generating information on the design and performance of ternary OPC/FA/SF concrete, several issues which can be considered critical from the standpoint of bridge decks applications have been studied only briefly or not at all. These include the following:

- Optimum content of class C fly ash and silica fume in the ternary system;
- Transport-related properties;
- Impact of curing quality on the properties of ternary concrete (compressive strength, transport-related properties, scaling resistance, drying shrinkage and resistance to shrinkage cracking);
- Critical air-void system parameters required to ensure freeze-thaw resistance;
- Applicability of maturity method for strength prediction and potentially for prediction of resistance to chloride-ion penetration.

It was also evident from the literature review that, although numerous researchers studied variety of properties of ternary OPC/FA/SF concrete using laboratory procedures, there is a lack of published data on the performance such concrete in the actual bridge decks. Since the laboratory testing can only provide some indication of

general (and often relative) behavior of concrete, the actual performance of concrete can be verified only through the field use in the actual structure.

Only one publication (Streeter 1996) reporting field trials of utilization of OPC/FA/SF concrete in several bridge decks in New York State has been encountered during the literature study. According to the author of that paper, all the trial bridge decks exhibited satisfactory performance (i.e., no cracking was observed), which subsequently led him to recommendation that the ternary OPC/FA/SF concrete should be used in the future bridge decks projects. However, this recommendation was based on rather short-term (less than one year) data and further verification is needed for several distresses typical of bridge decks (corrosion of reinforcement, surface scaling and freeze-thaw damage), which start to emerge only after substantial amount of time in service. Also, for bridge decks described by Streeter, class F fly ash was used. Since the focus of current research is on incorporation of class C fly ash, which is prevailing in the state of Indiana (Tanikella 2008), certain disparities in performance of concrete should be anticipated, and thus verification is needed whether utilization of OPC/FA/SF with class C fly ash in bridge deck concrete will be equally successful.

CHAPTER 3 – PRELIMINARY OPTIMIZATION ANALYSIS OF TERNARY OPC/FA/SF CONCRETE

3.1 Introduction

Based on an extensive body of research data available in the literature on concretes containing pozzolanic materials, there is a general agreement that the best performing mixtures typically incorporate ternary or quaternary cementitious blends rather than just a single mineral admixture (Bleszynski et al. 2002, Hooton and Titherington 2004, Mehta and Gjrv 1982, Toutanji et al. 2005). In particular, a combination of cement, fly ash and silica fume has been suggested to be very promising for bridge decks applications (Bouzouba et al. 2004). Although the optimum (and relatively wide) ranges have been established for incorporation of individual pozzolanic materials in binary cementitious mixtures, i. e. 20-40% for fly ash, and 5-10% for silica fume by mass of binder (Atcin 1998, Neville 1995, Nikam and Tambvekar 2003, Whiting and Detwiler 1998), the issue of optimum proportions of these materials when used in multicomponent system remains unresolved.

In order to adequately select the best, i.e., optimum mixture, one has to carry out a complete optimization process which involves selection of experimental variables, constraints, objective functions, and properly assigned weights. Furthermore, the optimization method and the method of selection of the optimum mixture need to be adequately chosen. Several approaches have been proposed, as far as concrete mixture optimization methodology is concerned, including factorial designs (Ghezal and Khayat 2002, Olek and Lu 2004, Patel et al. 2004, Simon and Lagergren 1997, Sonebi et al. 2004), mixture method, response surface method, Tagushi's method (Lin et al. 2004), genetic algorithm (Lim et al. 2004) and artificial neural networks (Tao et al. 2006). Among these methods, response surface methodology (RSM) appears to be the most

popular (Abbasi et al. 1987, Bayramov et al. 2004, Muthukumar et al. 2003, Nehdi and Sumner 2004). This method was developed for relatively large and complex experimental matrices, which results in selective examination (production and testing) of considered mixture designs. Subsequently, regression models are developed, which approximate the properties of all mixture designs included in the experimental program (Ruiz et al. 2005). Although this is undeniably a great advantage from the standpoint of workload and time savings, it also implies that in order to draw any valuable conclusions about hardened concrete properties, the fresh properties of all mixtures need to be relatively similar.

This study constitutes a preliminary optimization analysis of ternary concrete mixtures for bridge decks. It was aimed at identification of the optimum mixture from a relatively narrow range of four ternary (cement + fly ash + silica fume) mixture designs, each of which exhibited different fresh concrete properties, i.e. slump and air content. Moreover, in order to determine the sensitivity of evaluated concrete properties to change in the source of raw materials, i.e. cement, fine and coarse aggregate, the study was conducted using two series of mixtures (Series 1 and Series 2), each consisting of 20 and 10 mixtures, respectively. The detailed description of mixture proportions and summary of properties of materials used in Series 1 and 2 are provided in the following section.

3.2 Experimental Details

3.2.1 Materials

Ordinary ASTM C 150 Type I portland cement (ASTM 2008a) from two different sources was used. One source of Class C fly ash, meeting requirement of ASTM C 618 (ASTM 2008c), and one source of dry, densified silica fume was utilized in both series of mixtures. Table 3.1 contains the chemical composition and physical properties of all cementitious materials used in the study, as provided on the plant certificates.

Different sources of Indiana #8 (INDOT 2006) crushed limestone, with the maximum aggregate size of 19 mm, and natural siliceous Indiana #23 sand (INDOT 2006)

were used for each series of mixtures. The physical properties of fine and coarse aggregate are given in Table 3.2, while gradations are shown in Fig. 3.1.

Table 3.1 – Physical properties and chemical composition of cementitious materials used in preliminary optimization analysis (as reported by the suppliers)

Property	Cement (Series 1)	Cement (Series 2)	Class C Fly Ash (Series 1&2)	Silica Fume (Series 1&2)
<i>Physical</i>				
Specific Gravity	3.15	3.15	2.67	2.20
Fineness – Retained on #325 Mesh	-	99	9.6	-
– Blaine’s Surface Area (cm ² /g)	3470	3620	-	-
Compressive Strength Tested on Mortar				
Cubes (MPa) – 1 day	16.5	15.9	-	-
– 3 day	26.0	26.8	-	-
– 7 day	32.2	33.6	-	-
– 28 day	-	41.8	-	-
<i>Chemical</i>				
Silicon Dioxide SiO ₂ (%)	20.04	20.60	36.16	93.07
Aluminum Oxide Al ₂ O ₃ (%)	5.84	4.70	20.32	0.62
Ferric Oxide Fe ₂ O ₃ (%)	2.28	2.60	7.58	0.41
Calcium Oxide CaO (%)	64.87	64.90	23.94	0.66
Magnesium Oxide MgO (%)	1.63	2.60	5.47	1.16
Sulfur Trioxide SO ₃ (%)	3.28	2.50	1.91	<0.01
Loss on Ignition (%)	1.13	1.31	0.43	2.71
Sodium Oxide Na ₂ O (%)	0.14	-	-	-
Potassium oxide K ₂ O (%)	0.88	-	-	-
Total Alkali as Sodium Oxide Na ₂ O (%)	0.72	0.53	1.35	0.67
Insoluble Residue (%)	0.47	0.34	-	-
<i>Potential Compound Composition (Bogue)</i>				
Tricalcium Silicate C ₃ S (%)	60	65	-	-
Dicalcium Silicate C ₂ S (%)	12	10	-	-
Tricalcium Aluminate C ₃ A (%)	12	8	-	-
Tetracalcium Aluminoferrite C ₄ AF (%)	7	8	-	-

Three chemical admixtures were used in all mixtures: synthetic air entraining agent, polycarboxylate-based high-range water reducing admixture (HRWRA) and mid-range water reducing/set retarding admixture. The dosages of air entraining agent and high-range water reducer varied, and were adjusted individually for each mixture in order to reach the target values of the air content in the range 5.0÷8.5% and

slump in the range 100÷190 mm. The mid-range water reducer was used to ensure retention of satisfactory workability of concrete.

Fig. 3.1 – Gradations of fine (a) and coarse (b) aggregate for Series 1 and 2 mixtures

Table 3.2 – Physical properties of aggregate

	Coarse Aggregate (Series 1)	Fine Aggregate (Series 2)	Coarse Aggregate (Series 1)	Fine Aggregate (Series 2)
Specific Gravity (SSD)	2.69	2.62	2.65	2.66
Absorption (%)	1.4	1.2	1.1	1.6
Aggregate Correction Factor for Air Content (%)	0.05		0.50	

3.2.2 Mixture Proportions

This laboratory study was focused on evaluation of four ternary mixtures containing different combinations of fly ash (20% or 30% by mass of cementitious materials) and silica fume (5% or 7% by mass of cementitious materials). These are the extreme contents allowed by the special provision for recently constructed bridge deck on SR 23 over SR 20 near South Bend, Indiana (INDOT 2004), as discussed broadly in chapter 8. The cement content of all four mixtures was the same (231 kg/m^3) and they were all prepared at a constant water-cementitious materials ratio of 0.41. As a result, the paste

content of these mixtures varied from 23% to 28% of their total volume. The detailed proportions of mixtures studied are given in Table 3.3.

Since different sources of fine and coarse aggregate were used in Series 1 and 2 mixtures, the proportions needed to be adjusted to account for their different specific gravity and absorption values. The adjusted mixture proportions are provided in Table 3.3.

Table 3.3 – Mixture proportions for part 1 laboratory studies

Mix Designation		20FA/5SF	20FA/7SF	30FA/5SF	30FA/7SF
Total Cementitious materials (kg/m ³)		308.3	317.0	355.9	367.4
Paste (%)		23.1	23.9	26.9	27.9
Cement (kg/m ³)		231.3	231.3	231.3	231.3
Fly Ash (kg/m ³)		61.7	63.4	106.8	110.3
Silica Fume (kg/m ³)		15.3	22.3	17.8	25.8
Fine Agg. (SSD) (kg/m ³)	Series 1	747	738	706	696
	Series 2	(739)	(720)	(695)	(688)
Coarse Agg. (SSD) (kg/m ³)	Series 1	1116	1104	1055	1040
	Series 2	(1142)	(1129)	(1072)	(1063)
Water (kg/m ³)		126	130	146	151
Air Entraining Agent (ml/m ³)	Series 1	50÷101	52÷83	58÷93	60÷108
	Series 2	40÷51	41÷52	58÷70	60÷72
HRWRA (ml/m ³)	Series 1	805÷3017	1034÷1551	928÷1625	958÷1676
	Series 2	704÷1609	724÷1344	812	719
Mid-Range Water Reducing/Set Retarding Admixture (ml/m ³)		402	414	464	479

3.2.3 Mixing Procedure

All mixtures were prepared in a laboratory pan mixer with a nominal capacity of 0.05 m³. The following procedure was used for preparation of all laboratory mixtures. Coarse and fine aggregate were first mixed for 30 seconds. Slightly diluted air entraining admixture was then introduced. Within the next 30 seconds cement, fly ash and silica fume were added followed by addition of mixing water. Subsequently, a mid-range water reducer and high-range water reducer were added. From the moment of starting the mixer, mixing took place for 3 minutes. This was followed by 2 minutes of rest and mixing was then resumed for another 3 minutes.

Slump, air content and unit weight were measured immediately after completion of mixing following the relevant ASTM procedures.

3.2.4 Test Procedures

Compressive strength test was carried out after 28 days of standard moist curing on three 102×203 mm cylinders. The average results will be reported.

In order to evaluate the resistance of concrete to chloride ion penetration, the rapid chloride permeability (RCP) test (AASHTO T 277 (AASHTO 2006f)) was carried out on specimens moist cured for a period of 56 days. The reported RCP results represent an average obtained from four 52 mm thick disks, two sampled from the upper part of the 102×203 mm cylinder (after discarding about 6.5 mm thick top layer) and two sampled from the bottom part of a cylinder (located about 114-165 mm from the top).

Free shrinkage test was performed on three 76×76×286 mm prismatic specimens following the AASHTO T 160 (AASHTO 2006c) procedure, except for the changes in moist curing method (moist room instead of lime-saturated water curing) and the change in the length of curing period (7 days instead of 28 days). At the end of the curing period the specimens were removed from the moist room and subjected to drying at 23°C and 50% RH. The shrinkage was monitored using a length comparator over a period of 365 days, however the average values of shrinkage after 56 days of drying have been used for analysis. With the exception of free shrinkage test which was performed on only Series 2 mixtures, all tests described above were conducted on mixtures from both series.

3.3 Test Results and Statistical Analysis

Fig. 3.2 presents the relationship between slump and air content along with the corresponding target ranges for these properties. It can be seen that 8 out of 20 mixtures for Series 1 and 8 out of 10 mixtures for Series 2 fell into the target range of both slump and air content. It can also be noticed that, particularly for Series 1 mixtures, there exists a strong relationship between slump and air content. This is confirmed by relatively high

coefficient of determination ($R^2 = 0.78$) for Series 1 mixtures. That coefficient was reduced to 0.58 if both series were considered simultaneously. The reason for this reduction is most likely related to the fact that for some of the Series 2 mixtures the dosage of the air entraining admixture or HRWRA was readjusted if the concrete exhibited too low slump or air content.

Fig. 3.2 – Air content vs. slump (Series 1 and 2 mixtures)

In the statistical models developed for all three tested hardened concrete properties (i.e., 28-day compressive strength, 56-day rapid chloride permeability and 56-day free shrinkage) discussed in the following section, only air content was taken into account as a variable representing the fresh concrete properties. This was done to avoid serious multicollinearity effects, which would have resulted in obtaining erroneous models due to intercorrelation between the predictor variables, in spite of the fact that an improved R^2 value can be obtained (Kutner et al. 2005).

Fig. 3.3 shows the development of free shrinkage results over time. The variation in strains within a certain nominal mixture design is practically as high as between the mixture designs themselves, thus making it impossible to conclude about the effect of mixture composition on free shrinkage.

Fig. 3.3 – Free shrinkage strains with time (preliminary optimization study)

Presented in Fig. 3.4 are the results of 28-day compressive strength. It can be seen that, regardless of composition, most of the mixtures tested achieved compressive strength of about 40 MPa and higher, which is often perceived as a desired level of compressive strength for bridge deck concrete (Xi et al. 2001). Similarly, almost all mixtures, irrespectively of the composition, exhibited good resistance to chloride-ion penetration (Fig. 3.5), as indicated by the fact that only one mixture showed RCP test results higher than 1500 coulombs, which is considered more than satisfactory according to most of the bridge decks specifications. Likewise, for none of the mixtures was the 56-days free drying shrinkage (Fig. 3.6) greater than -560 microstrains, which was roughly an expected value based on the of previous studies carried out on high-performance concretes of similar composition (Olek et al. 2002).

Fig. 3.4 – 28-day compressive strength (Series 1 and 2 mixtures)

Fig. 3.5 – 56-day rapid chloride permeability results (Series 1 and 2 mixtures)

When analyzing the data, it was evident that the scatter present in the test results was often as large between the replicate mixtures of one design as between the mix designs themselves. Thus, it was virtually impossible to determine which mixture composition is actually the best, i.e., results in the highest strength, lowest coulomb value, and lowest shrinkage. It appears that the only conclusion that could be drawn from visual examination of the data presented in Fig. 3.4-Fig. 3.6 is that RCP results obtained within the same mixture composition for Series 2 are somewhat higher than those obtained for Series 1 mixtures (Fig. 3.5).

Fig. 3.6 – 56-day free drying shrinkage (Series 2 mixtures)

Therefore, for each of the properties mentioned above statistical analysis was performed and, using the least squares method, the following multiple regression models were developed:

$$STR = 73.9 - 0.040 \cdot CM - 0.258 \cdot AIR^2 \quad (R^2=0.86) \quad \text{Eq. 3.1}$$

$$RCP = 992 - 37.57 \cdot AIR + 18.76 \cdot FA - 124.35 \cdot SF + 341.78 \cdot \text{Series\#} \quad \text{Eq. 3.2}$$

($R^2=0.89$)

$$SHR = 625 - 2.56 \cdot CM - 4.08 \cdot AIR^2 \quad (R^2=0.82) \quad \text{Eq. 3.3}$$

where:

STR – 28-day compressive strength (MPa);

RCP – 56-day rapid chloride permeability (coulombs);

SHR – 56-day free shrinkage (microstrains);

CM – mass of cementitious materials (kg/m³);

AIR – fresh concrete air content (%);

FA – amount of fly ash (% by mass of total binder);

SF – amount of silica fume (% by mass of total binder).

The values of coefficients of determination obtained for the prediction models were not the only parameters used for selection of the best-fitting models. The *t*-test was also used to assess the statistical significance of predictor variables in each of the models, whereas variance inflation factor (VIF) was computed to check for the presence of multicollinearity effect in the models (Kutner et al. 2005). Furthermore, development of any models with reasonable value of R^2 was only possible when several outlying results were removed from the data set. In the case of model for 28-day strength, the results for mixtures 3, 8 and 24 were excluded, for 56-day RCP the results for mixtures 6 and 8 were removed, and for 56-day shrinkage the results obtained for mixture 16 was discarded, as they appeared to contradict the expected trends.

The developed regression models are very useful in terms of identifying which factors, and to what extent, affected the examined parameters. However, due to specific conditions under which the mixtures were prepared and tested, as well as because of variations in composition and properties of raw materials used, the models can be expected to yield only rough estimates of the anticipated concrete properties. It can be seen, that an increase in air content, which was found to be statistically significant in all the developed models, appeared to reduce compressive strength and RCP test results and to increase shrinkage. It is also apparent that an increase in the amount of cementitious materials, or in other words an increase in the volume of paste, reduced compressive strength and increased shrinkage. The model developed for RCP contains further

refinement, and implies that an increase in fly ash content increases the coulomb values, as opposed to silica fume, which results in lower RCP results when used in larger quantities. This is in agreement with findings from the earlier study as reported by Olek and Lu (2004). Finally, the change in source of materials appeared to alter rapid chloride permeability results by approximately 30%, while compressive strength was not found to be affected by this variable.

In order to enable the actual comparison of results obtained from mixtures of different compositions, the values of response functions for evaluated concrete properties (as expressed by Eq. 3.1 through Eq. 3.3) were back-calculated for each of the mixture design and series (if present in the model). The results are given in Table 3.4. The air content used in this process was set at the target value of 6.5 %. In addition, Table 3.4 also provides an estimated unit costs of 1 m³ of concrete mixtures calculated using unit prices of the raw materials acquired from the local ready-mix concrete plant located in Indiana.

Table 3.4 – Back-calculated values of response functions

Criterion	28-day Compressive Strength (MPa)	56-day Rapid Chloride Permeability (coulombs)		56-day Free Shrinkage (μstrains)	Unit cost (\$/m ³)
	Series 1&2	Series 1	Series 2	Series 2	Series 2
R ² for model	0.86	0.89		0.82	-
20FA/5SF	51.5	843	1185	-338	126
20FA/7SF	51.2	594	936	-360	136
30FA/5SF	49.9	1030	1372	-460	128
30FA/7SF	49.5	782	1123	-489	139

3.4 Optimization Analysis

As a last step, an ultimate goal of this study – optimization of mixture composition, was carried out. As it was mentioned previously, due to relatively narrow range of mixture composition of interest, a simple optimization problem has been formulated and all 4 mixture compositions were examined. Since free shrinkage measurements were not performed for Series 1 mixtures, the optimization analysis was conducted using only the results for Series 2 mixtures (see Table 3.4).

The decision variables vector for this optimization problem was defined as:

$$\mathbf{x} = [x_1, x_2]^T \quad \text{Eq. 3.4}$$

where x_1 and x_2 – percentage of FA (20% or 30%) and SF (5% or 7%), respectively.

The objective functions vector has been specified as follows:

$$f(\mathbf{x}) = [f_1(x), f_2(x), f_3(x)]^T \quad \text{Eq. 3.5}$$

where:

$f_1(x)$ – durability function (permeability and shrinkage) – to be minimized;

$f_2(x)$ – mechanical function (compressive strength) – to be maximized;

$f_3(x)$ – economical function (unit cost) – to be minimized.

Since the durability function consists of two components, it can be expressed as:

$$f_1(\mathbf{x}) = [f_{1,1}(x), f_{1,2}(x)]^T \quad \text{Eq. 3.6}$$

where:

$f_{1,1}(x)$ – permeability – to be minimized;

$f_{1,2}(x)$ – shrinkage – to be minimized.

The presented optimization problem is a typical multi-objective problem which, due to conflicting objectives, often prevents simultaneous optimization unless special methods of single objective determination are employed. In this study, the normalized weighted sum method has been employed to select the preferred (optimum) value of the objective functions and corresponding optimum solution (mixture composition). The underlying principle of this method is calculation of value of function f_s for each of the evaluated solutions, i.e. mixture compositions, using a formula which in general has the form:

$$f_s = \sum_{j=1}^J w_j \cdot \frac{f_j(x)}{\max f_j(x)} \quad \text{Eq. 3.7}$$

where:

J – number of objective functions;

w_j – weight of the j -th objective function;

$f_j(x)$ – value of the j -th objective function for solution x .

In general, function f_s is to be minimized. In order to properly account for fact that durability function f_l consists of two sub-functions $f_{l,1}$ and $f_{l,2}$ (representing resistance to chloride ion penetration and shrinkage respectively), as well as for the negative values of shrinkage and the need to maximize the compressive strength, the actual formula used for calculations takes a form:

$$f_s = w_l \cdot \left[\frac{w_{l,1} \cdot \frac{f_{l,1}(x)}{\max f_{l,1}(x)} + w_{l,2} \cdot \frac{f_{l,2}(x)}{\min f_{l,2}(x)}}{\max \left(w_{l,1} \cdot \frac{f_{l,1}(x)}{\max f_{l,1}(x)} + w_{l,2} \cdot \frac{f_{l,2}(x)}{\min f_{l,2}(x)} \right)} \right] - w_2 \cdot \frac{f_2(x)}{\max f_2(x)} + w_3 \cdot \frac{f_3(x)}{\max f_3(x)}$$

Eq. 3.8

The objective functions weights for, respectively, function $f_l(x)$ and function $f(x)$ were specified as follows:

$$w_l = [w_{l,1}, w_{l,2}]^T = [0.5, 0.5]^T \quad \text{Eq. 3.9}$$

$$w = [w_1, w_2, w_3]^T = [0.6, 0.2, 0.2]^T \quad \text{Eq. 3.10}$$

The choice of the weight coefficients assigned to each of the objective functions, however arbitrary, was driven by the intended application of optimized mixture for HPC bridge decks. As selected, the relative weights imply that permeability and shrinkage were equally important and comprised 60% in terms of the overall importance, while the strength and unit cost of mixture were assigned an importance of 20% each.

The calculated values of function f_s for each mixture composition as well as their rank are given in Table 3.5. The lowest rank denotes the best mixture design. As it can be seen, for the selected optimization criteria and specified weight coefficients assigned to them, the optimum mixture is the one that contains 20% of fly ash and 7% silica fume by mass of binder. For that mixture the resulting function f_s equals 0.44. Moreover, it appears that mixture 20FA/5SF which ranked as a second best mixture is not much worse than the optimum mixture (f_s equals 0.46). In addition, the performance of

both mixtures containing 30% of fly ash seems to be poorer as compared to both mixtures containing only 20% of fly ash.

Table 3.5 – Calculated values of function f_s and ranks of the examined mixture designs

Mix Designation	f_s	Rank
20FA/5SF	0.46	2
20FA/7SF	0.44	1
30FA/5SF	0.59	4
30FA/7SF	0.57	3

Specific relationships between the evaluations of all the normalized and weighted objective functions are shown in Fig. 3.7. To help with the interpretation of these graphs, an example is provided using plot “a”, which shows the relationship between the normalized and weighted objective sub-functions $f_{1,1}$ (permeability) and $f_{1,2}$ (shrinkage). Because both functions are being minimized, the ideal value y_{id1} (and corresponding solution) are determined by the x-coordinate of the point having the minimum value of $w_{1,1}f_{1,1}/\max f_{1,1}(x)$ (mixture 20FA/7SF) and y-coordinate of the point having the minimum value of $w_{1,2}f_{1,2}/\max f_{1,2}(x)$ (mixture 20FA/5SF). The preferred (optimum) evaluation y_{p1} is determined using a distance method (Brandt and Marks 1996, Deb 2001, Paczkowski 1999), and it constitutes a point which is closest (geometrically) to ideal point y_{id1} .

Similarly, the anti-ideal evaluation (not shown in Fig. 3.7) would be represented by the point having x-coordinate of the point having the maximum value of $w_{1,1}f_{1,1}/\max f_{1,1}(x)$ (mixture 30FA/5SF) and y-coordinate of the point having the maximum value of $w_{1,2}f_{1,2}/\max f_{1,2}(x)$ (mixture 30FA/7SF). Also, by analogy, the worst solution is mixture 30FA/5SF being the closest to anti-ideal point. Analogical interpretation can be applied to the three other plots showing relationships between functions f_1 , f_2 and f_3 (Fig. 3.7, plots “b-d”). While parameter y_{id1} denotes an ideal evaluation of the objective sub-functions (i.e., permeability and shrinkage), parameter y_{id}

occurring in the plots “b” through “d” is an ideal evaluation of the objective functions in the entire optimization problem (durability, mechanical and economical).

Fig. 3.7 – Evaluations of objective functions for $w_1 = 0.6$; $w_2 = 0.2$; $w_3 = 0.2$

It is interesting to note that mixtures 20FA/5SFA and 20FA/7SF are in all cases so-called “non-dominated solutions” (as indicated by non-dominated evaluations). That outcome implies that at least one of the normalized weighted objective functions reaches the extreme value (minimum for durability and economical function and maximum for strength). Furthermore, as shown in plot “d”, mixture 20FA/5SF is an ideal solution, i.e., it minimizes the unit cost and maximizes the compressive strength at the same time.

The dominating tendency of the mixtures containing 20% of fly ash has been examined further by determining the optimum mixture composition at any combination of values of weights w_1 , w_2 and w_3 . Due to computational constraints, the values of weights $w_{1,1}$ (corresponding to permeability) and $w_{1,2}$ (corresponding to shrinkage) were kept constant at 0.5. The ranges in which the examined mixtures become the optimum solution have been illustrated in the form of a ternary diagram (Fig. 3.8), in which each corner of a triangle denotes 100% weight of the respective objective function. Evidently, as the entire area of the diagram is taken by either mixture 20FA/5SF or 20FA/7SF, none of the two mixtures containing 30% of fly ash becomes an optimum solution, regardless of what weights are applied to the evaluated properties.

Fig. 3.8 – Diagram for optimum mixture selection for a full range of objective functions weights

3.5 Conclusions

On the basis of the work presented in this chapter, the following conclusions can be drawn:

- The proposed approach of statistical analysis of data prior to the actual optimization analysis helped in determining the parameters affecting each of the evaluated response functions and accounted for variability in concrete properties and test results.
- The developed regression models indicate that the increase in air content reduces compressive strength and rapid chloride permeability test results, but at the same time it also increases shrinkage.
- The change in material sources altered 56-day RCP results by approximately 30%, whereas the 28-day compressive strength appeared to be insensitive to change in materials source.
- For the predefined set of optimization criteria and selected weight coefficients, the optimum mixture contains 20% fly ash and 7% silica fume.
- The selection of the optimum mix design depends on the values of weights assigned to the corresponding criteria, but for any configuration of the weights the optimum mixture always contains 20 % rather than 30% of fly ash.

CHAPTER 4 – PART II LABORATORY STUDIES

4.1 Introduction

Several extensive laboratory studies were conducted on the ternary concrete mixture with fly ash (FA) and silica fume (SF). These studies were focused on in-depth analysis of key properties and parameters of HPC used for bridge decks. The purpose of the research presented in this chapter was driven by the experience gained from chapter 3. In attempt to best characterize the ternary mixtures, it was inevitable to capture certain important aspects (not considered in chapter 3) that might have a significant impact on their potential performance. The parameters examined in this chapter include transport-related properties curing, freezing-thawing and scaling resistance, strength development and resistance to shrinkage cracking. Further, for most properties the effect of curing conditions was also examined.

4.2 Experimental Program

4.2.1 Materials

The same materials were used throughout the laboratory studies conducted within the scope of this report. Ordinary ASTM C 150 Type I portland cement (ASTM 2008a) and two supplementary cementitious materials, namely ASTM class C fly ash meeting requirement of ASTM C 618 (ASTM 2008c) and dry commercial grade densified silica fume, were utilized. The physical properties and chemical composition of all cementitious materials used in the study are provided in Table 4.1.

The coarse aggregate used in the study was Indiana #8 crushed limestone (INDOT 2006), with the maximum aggregate size of 19 mm, specific gravity (SSD) of 2.65 and absorption of 1.6%. The fine aggregate used was natural siliceous Indiana #23 sand (INDOT 2006) with specific gravity (SSD) of 2.66 and absorption of 1.4%. The

aggregate correction factor for the air content of the mixture of fine and coarse aggregate used was 0.5%. The gradation of fine and coarse aggregate is shown in Fig. 4.1.

Table 4.1 – Physical properties and chemical composition of cementitious materials used in part II laboratory studies (as reported by the suppliers)

Property	Cement	Class C Fly Ash	Silica Fume
<i>Physical</i>			
Specific Gravity	3.15	2.56	2.20
Fineness – Retained on #325 Mesh	-	20.4	-
– Blaine’s Surface Area (cm ² /g)	3470	-	-
Compressive Strength Tested on Mortar			
Cubes (MPa) – 1 day	16.5	-	-
– 3 day	26.0	-	-
– 7 day	32.2	-	-
– 28 day	-	-	-
<i>Chemical</i>			
Silicon Dioxide SiO ₂ (%)	20.04	41.40	93.07
Aluminum Oxide Al ₂ O ₃ (%)	5.84	19.98	0.62
Ferric Oxide Fe ₂ O ₃ (%)	2.28	5.95	0.41
Calcium Oxide CaO (%)	64.87	16.23	0.66
Magnesium Oxide MgO (%)	1.63	3.72	1.16
Sulfur Trioxide SO ₃ (%)	3.28	0.99	<0.01
Loss on Ignition (%)	1.13	1.05	2.71
Sodium Oxide Na ₂ O (%)	0.14	-	-
Potassium Oxide K ₂ O (%)	0.88	-	-
Total Alkali as Sodium Oxide Na ₂ O (%)	0.72	2.46	0.67
Insoluble Residue (%)	0.47	-	-
<i>Potential Compound Composition (Bogue)</i>			
Tricalcium Silicate C ₃ S (%)	60	-	-
Dicalcium Silicate C ₂ S (%)	12	-	-
Tricalcium Aluminate C ₃ A (%)	12	-	-
Tetracalcium Aluminoferrite C ₄ AF (%)	7	-	-

Three chemical admixtures were used in all mixtures: synthetic air entraining agent, polycarboxylate-based high-range water reducing admixture (HRWRA) and mid-range water reducing/set retarding admixture. The dosages of the first two admixtures varied, and were adjusted individually for each mixture in order to reach the target values of the air content and slump of 6.5% and 150 mm, respectively.

Fig. 4.1 – Gradation of fine aggregate (a) and coarse aggregate (b)

4.2.2 Mixture Proportions

The detailed proportions of mixtures studied are given in Table 4.2. Most of the laboratory studies were focused on evaluation of four ternary mixtures containing different combinations of fly ash (20% or 30% by mass of cementitious materials) and silica fume (5% or 7% by mass of cementitious materials). These mixtures have been already studied during the preliminary optimization analysis presented in chapter 3. The cement content of all four mixtures was the same (231 kg/m^3) and they were all prepared at a constant water-cementitious materials ratio of 0.41. As a result, the paste content of these mixtures varied from 23% to 28% of their total volume.

In addition, for comparative purposes two more mixtures were assessed in the selected studies. The first mixture was made according to INDOT's specification for class C (INDOT 2008). The mixture had slightly higher w/cm compared to the main ternary mixtures of interest (0.43 instead of 0.41), and much higher cement content (390 kg/m^3 compared to 231 kg/m^3). The second mixture was a ternary mixture with 5% of SF and 20% of FA, by total mass of binder, and it was used on the pilot HPC bridge deck on SR 23 over SR 20 near South Bend, IN (see chapter 5). This mixture will be later referred to as "20FA/5SF – SR 23". The mixture composition was selected by the

contractor based on the requirements of the provisional specification (INDOT 2004). It was similar to the composition of the first ternary mixture under evaluation in this study, however the w/cm of 0.40 used (instead of 0.41) and the cement content was 250 kg/m^3 (compared to 231 kg/m^3).

Since each mixture was made many times and the dosage of air entraining agent and high-range water reducer were adjusted individually for each batch, the respective values reported in Table 4.2 are the mean dosages.

To avoid repetitions, it shall be assumed that each of the studies described in sections 4.3-4.9 was conducted on four main ternary mixture only. If any of two (or both) additional mixtures were evaluated, it will be specified in respective section regarding experimental details. The mixing procedure used for all laboratory studies discussed in this chapter has been described in section 3.2.3.

Table 4.2 – Mixture proportions for part II laboratory studies

Mix Designation	20FA/5SF	20FA/7SF	30FA/5SF	30FA/7SF	INDOT Class C	20FA/5SF – SR 23
Water-Cementitious Materials Ratio	0.41	0.41	0.41	0.41	0.43	0.40
Total Cementitious Materials (kg/m^3)	308	317	356	367	390	332
Paste (%)	23	24	27	28	29	25
Cement (kg/m^3)	231	231	231	231	390	249
Fly Ash (kg/m^3)	62	63	107	110	–	66
Silica Fume (kg/m^3)	15	22	18	26	–	17
Fine Agg. (SSD) (kg/m^3)	746	738	706	696	732	785
Coarse Agg. (SSD) (kg/m^3)	1116	1104	1055	1040	965	1035
Water (kg/m^3)	126	123	146	150	168	133
Air Entraining agent – Avg. (ml/m^3)	55	66	109	134	118	70
HRWRA – Avg. (ml/m^3)	2011	2378	928	839	768	2070
Mid-Range Water Reducing/Set Retarding Admixture (ml/m^3)	402	413	464	479	512	436

4.3 Effect of Mixture Composition and Initial Curing Condition on Transport Properties of Ternary OPC/FA/SF Mixtures

4.3.1 Introduction

It is generally accepted that degradation of concrete in harsh environment is mainly driven by the moisture and ion transport mechanisms, namely capillary suction and diffusion. Ever since high performance concrete (HPC) became a product of choice for the structures exposed to corrosive environment, there has been a great need for developing a test method which would allow for objective and relatively quick assessment of its transport-related properties. However, despite extensive research efforts in this area, to-date no universally accepted methods of determining these properties exist.

A bulk diffusion test has been recently developed for determination of chloride-ion diffusion coefficient in the saturated concrete and standardized under ASTM C 1556 designation (ASTM 2004a). While, as will be described later, the test procedure is extremely tedious and lengthy, the bulk diffusion test is one very few which have a well established theoretical background (Nielsen and Geiker 2003, Stanish and Thomas 2003) and provide a transport-related concrete property as an output. The knowledge of the diffusion coefficient is crucial for assessment of predicted time to corrosion initiation (Berke and Hicks 1992, Magee et al. 1996, Val 2006) and service life modeling (Boddy et al. 1999, de Rooij and Polder 2004, Violetta 2002) for reinforced concrete structures. ASTM C 1556 (ASTM 2004a) describes in detail the procedure for obtaining the chloride concentration profiles from specimens soaked in chloride solution and for calculation of diffusion coefficient. The following formula being Crank's solution for Fick's second law is used to compute the apparent diffusion coefficient:

$$C(x, t) = C_s - (C_s - C_0) \cdot \operatorname{erf}\left(\frac{x}{\sqrt{4D_a t}}\right) \quad \text{Eq. 4.1}$$

where $C(x, t)$ is time- and depth-dependent chloride concentration (kg/m^3), C_s is surface concentration, C_0 is background concentration, x is average depth from surface (m) and

D_a is apparent diffusion coefficient. The values of D_a and C_s are obtained using regression analysis (least squares method).

Undeniably, the most common method used for determining resistance of concrete to chloride-ion penetration is rapid chloride permeability (RCP) test. This method has been standardized in the U. S. as ASTM C 1202 (ASTM 2006e) and AASHTO 277 (AASHTO 2006f). Despite its frequent use and common belief that it gives a comparative indication of resistance of concrete to chloride ion penetration (Aïtcin 2003, Bleszynski et al. 2002, Hooton and Titherington), this test method is often criticized (Hooton et al. 2001, Shi 2004) for providing the values of current that reflect concentration of all ions in the solution (not just chlorides). Partially in response to this criticism, a rapid migration test has been recently standardized under designation AASHTO TP 64 (AASHTO 2006b) with the premise of providing better indication of concrete resistance to penetration of chloride ions (Stanish et al. 2000). The rapid migration test is viewed as being less affected by the presence of conductive ions than RCP test due to generally lower voltage applied, which also limits the temperature increase during testing. In addition, the actual ingress of chloride ions (depth of penetration) is evaluated by rapid migration test (Stanish et al. 2000).

Due to their simplicity, the absorption-based tests have always been of particular interest to those concerned with moisture movement in concrete. The initial arguments of lack of theoretical basis for this type of tests have been resolved by adopting the water sorptivity test, which was earlier successfully applied to other building materials and soils (Hall 1981, Philip 1957), and appeared to have solid theoretical basis (Hall 1989, Kehlam 1988, Taha et al. 2001). According to original definition by Philip (1957), sorptivity is a measure of the capacity of the medium to absorb or desorb liquid by capillary suction. A linear relationship was found to exist between the unidirectional water intake through unsaturated surface (absorption I in mm) and the square root of time ($t^{1/2}$), as shown in Eq. 4.2:

$$I = S\sqrt{t} + S_0 \quad \text{Eq. 4.2}$$

where S is sorptivity (rate of absorption in $\text{mm/s}^{1/2}$), t is time in sec., S_0 is a positive intercept with I axis and I is absorption calculated from the formula:

$$I = \frac{m}{a\rho} \quad \text{Eq. 4.3}$$

in which m denotes mass, a is cross-sectional area exposed to water and ρ is the density of water.

An alternative to this simple linear approach was proposed by Martys and Ferraris (1997), who suggested describing the relationship between water intake and time by using an empirical stretched exponential function. Recently, Neithalath (2006) proposed yet another equation, which allows for simultaneous determination of sorption and diffusion. Although both of these approaches provide a good fit to experimental data, the adequacy of the models linking several parameters included in a water intake function to a single variable, such as time, may be questionable.

A natural consequence of a strong interest in sorptivity data is an existence of variety of test procedures that vary in specimen size and method of sample preconditioning. As a result, the obtained sorptivity values often differ by several orders of magnitude. In response to these concerns, the American Society for Testing and Materials recently introduced a standard test method for sorptivity under designation ASTM 1585 (ASTM 2006g). Two parameters are obtained from this standard sorptivity test: initial sorptivity and secondary sorptivity. These correspond to the slopes of a plot of absorption I vs. $t^{1/2}$ for, respectively, the first 6 hours or the period between 1 and 8 days. The interpretation of sorptivity test results is complicated by the fact that, unlike the initial sorptivity, the secondary sorptivity is usually attributed to mechanisms other than capillary suction, including filling of larger pores and voids (Bentz et al. 2001). Even though standardization of the test was hoped to provide more reproducible and uniform results, the data obtained during the present research indicate that the proposed procedure still suffers from several shortcomings. In addition, no commonly accepted ranges of sorptivity values for concretes with different permeabilities exist.

The main purpose of this study was to evaluate transport properties of four selected ternary concrete mixtures incorporating different amounts of fly ash and silica fume. These transport properties were evaluated using several currently available test methods that included: rapid-chloride permeability (RCP) test, rapid-migration procedure (RMP), bulk diffusion test, sorptivity test and absorptivity (ponding water absorption) test. All five of the above test methods were used to evaluate Series I mixtures that were continuously moist cured until testing. Additionally, the results of RCP, sorptivity and absorptivity tests obtained at 56 days for INDOT class C mixture (mixture composition as in Table 4.2) have been provided for comparative purposes.

Only two of these test methods (RCP and absorptivity) were used to evaluate Series II mixtures which had the same composition as Series I mixtures but were subjected to curing procedures that were more likely to lead to internal moisture condition of test specimens that were comparable to those of the field concrete. This approach allowed for the determination of sensitivity of these test methods to changes in properties of HPC resulting from differences in curing.

4.3.2 Experimental Details

4.3.2.1 Curing Procedures

The 102×203-mm concrete cylinders were cast for both series of mixtures but the curing method varied between the series. For the first day after casting all specimens were kept in molds covered with lids except the air dried specimens that were kept without lids. After the first day, all specimens prepared from Series I mixtures were continuously cured in a moist room until the age of testing (28, 56 or 180 days). The specimens from Series II mixtures were subjected to four different curing regimes: (a) air drying (no moisture or curing compound applied), (b) 3 days under wet burlap, (c) 7 days exposure to curing compound (curing compound applied to all surfaces immediately after demolding and removed after 7 days using a wire brush), and (d) 7 days under wet burlap.

Series II specimens were further divided into two subgroups, one which was tested at an early age (14 - 21 days) and the other tested at late age (90 days).

Following the initial curing regime (a - d), all specimens in the early age subgroup were subjected to drying at 23°C and 50% RH for 14 days. Depending on the initial curing regime (a - d) used, this resulted in initiating the testing procedure at 14, 17 or 21 days.

Following the initial curing regime (a - d), the specimens in the late age (90 days) subgroup were all dried at 23°C and 50% RH until the age of 69 day. Subsequently, they were all “re-saturated” by placing them in a moist room (100% RH) for 7 days. After resaturation period, all specimens were air dried for 14 days at 23°C and 50% RH. This procedure was employed in an attempt to obtain comparable internal moisture content and distribution in all of the Series II specimens tested, as these parameters are known to be the critical factors in absorption-type tests (DeSouza et al. 1997, Hall 1989, McCarter et al. 1992, Neithalath 2006). Despite these efforts, some discrepancy in moisture content between the specimens from different testing groups might have not been entirely eliminated (see discussion in section 4.3.3).

4.3.2.2 Test Procedures

Except for bulk diffusion test, all tests performed in this study, were carried out on 52-mm thick disks obtained from 102×203-mm cylinders as shown in Table 4.3. The bulk diffusion test (ASTM C 1556 (ASTM 2004a) was carried out on two 75-mm thick disks cut from the top of 102×203-mm cylinders. The specimens were coated with epoxy such that only the finished face remained uncoated. The specimens were then placed in a sealed container containing solution of sodium chloride (165 g of NaCl per 1 liter of solution) for 90 days. At the end of this exposure period the specimens were allowed to dry at 23°C and 50% RH for about two days. Subsequently, 10 layers of concrete were ground off using a lathe. The thickness of layers was as follows: 0-1, 1-2, 2-3, 3-5, 5-7, 7-9, 9-12, 12-16, 16-20 and 20-24 mm. Additionally, a 20-mm thick specimen cut from the same cylinder as the specimen exposed later to chloride solution was crushed using a jaw crusher for determination of the background chloride concentration. All 11 powdered samples obtained from each cylinder were sieved through 850-µm (#20) sieve and water-soluble chloride concentration was determined in them using automatic

titration. The average values of apparent diffusion coefficient and surface concentration for Series I mixtures will be reported.

The reported RCP test (AASHTO T 277) results for Series I mixtures represent the average obtained from four specimens, whereas for Series II mixtures, the RCP test was performed on two disks.

The rapid migration procedure (RMP) was performed as per AASHTO TP 64, except that no upper portion of the disk was discarded. The reported results constitute the average rate of penetration (in mm/(V-hr)), measured on both of the freshly split-opened surfaces of three disks after spraying them with 0.1 N silver nitrite.

Table 4.3 – Location and number of test specimens

	RCP	RMP	Sorptivity	Absorptivity	Bulk Diffusion
Series I All specimens moist cured
Series II Curing cond. (a), (b), (c) and (d)		—	—		—

Sorptivity test performed for Series I was carried out following ASTM C 1585. The conditioning procedure recommended in ASTM C 1585 is intended to ensure uniform distribution of moisture within the unsaturated test specimens (internal relative humidity between 50 and 70%). The current study examined applicability of this preconditioning procedure to specimens continuously cured at 100% RH.

At the time of testing, the sorptivity specimens were removed from the moist room and subjected to drying at 50°C and 80% RH for 3 days. Subsequently, the specimens were kept separately in a sealed plastic container for 15 days. After that period, the side of each test specimen (disk) was sealed with self-adhesive aluminum tape, while the top surface was covered with a loosely attached polyethylene sheet secured

with a rubber band. The specimens were then supported on spacers in plastic pans so that the water level was about 2 mm above the bottom surface of specimen (see Fig. 4.2(a)). The change in mass of the specimen was recorded (after wiping the exposed surface with dampened cloth) at 1, 5, 10, 20, 30 and 60 minutes after starting the test. Subsequently, the mass was determined every hour up to 6 hours and then every day (except for the 4-th day) up to 8 days.

The absorptivity test, sometimes referred to as ponding water absorption (Neithalath 2006) or ponding sorption test (Bentz et al. 2001), was conducted for Series I and Series II mixtures on two disks sampled from the upper portion of the cylinder (see Table 4.3). The sample conditioning procedure for both series was identical to the one used for sorptivity specimens and was initiated at the predesignated testing time after removing the specimens from the respective environment (moist room for Series I specimens and 50% RH for Series II specimens). In fact, the absorptivity test itself was performed nearly identically to sorptivity test, except for reversing the location of the surface of specimens in contact with water (Fig. 4.2(b)). The ponding dike was prepared at the top of the disk by attaching an additional strip of aluminum tape that extended by about 20 mm beyond the surface of the specimen and sealing it with silicon caulk. During the absorptivity test about 15 mm of water was placed in the dike. Similar test configuration was recently used by Bentz (2001) and Neithalath (2006).

Fig. 4.2 – Schematic of sorptivity test (a) and absorptivity test (b)

4.3.3 Test Results and Discussion

4.3.3.1 Series I Mixtures

The test results obtained from Series I mixtures are summarized in Table 4.4. Even at relatively young age of 28 days, the RCP values of all mixtures tested were very low. The highest result (~1160 coulombs) was obtained for mixture containing 30% of fly ash and 5% of silica fume (30FA/5SF), by the mass of binder. With time, these results reduced even further, and at 180 days none of the mixtures showed a value higher than 500 coulombs. It should be noticed, however, that although at any given age the RCP results obtained for different mixtures appear to be similar, the 20FA/7SF mixture always exhibited permeabilities that were slightly lower than those obtained for other mixtures. Very similar trends, if not more pronounced, can be noticed in the test results obtained from the RMP test (see Table 4.4).

Further, the RCP results of any of the ternary mixtures studied are markedly lower than the value of 4556 coulombs obtained for INDOT class C concrete tested at 56 days, proving much increased resistance to chloride-ion penetration of concrete containing fly ash and silica fume.

Table 4.4 – Summary of test results from Series I mixtures

Age (days)	Mixture	Charge passed (RCP) (coulombs)	Rate of penetration (RMP) (mm/(V-hr))	Initial absorptivity $\times 10^{-4}$ (mm/s ^{1/2})	Secondary absorptivity $\times 10^{-4}$ (mm/s ^{1/2})	Initial sorptivity $\times 10^{-4}$ (mm/s ^{1/2})	Secondary sorptivity $\times 10^{-4}$ (mm/s ^{1/2})	Apparent diffusion coefficient $\times 10^{-12}$ (m ² /s)	Surface concentration (kg/m ³)
28	20FA/5SF	1032 (4.6)	0.0175 (5.1)	7.2 (3.9)	2.2 (64.3)	7.6 (18.6)	3.3 (27.1)	1.87 (21.6)	28.9 (3.6)
	20FA/7SF	917 (2.6)	0.0140 (19.9)	4.8 (-)	1.2 (-)	5.4 (6.6)	2.7 (34.7)	1.66 (15.5)	39.8 (17.3)
	30FA/5SF	1161 (4.8)	0.0198 (18.0)	3.3 (72.9)	0.9 (94.3)	6.9 (15.5)	3.5 (2.2)	1.69 (4.4)	44.4 (4.0)
	30FA/7SF	1120 (2.1)	0.0174 (2.6)	6.1 (16.3)	3.3 (41.3)	5.9 (47.9)	3.5 (52.5)	1.77 (6.2)	34.8 (9.1)
56	20FA/5SF	657 (5.1)	0.0140 (10.1)	3.9 (-)	1.2 (-)	9.1 (9.3)	2.2 (0.0)	1.35 (11.9)	34.7 (0.4)
	20FA/7SF	569 (6.8)	0.0121 (1.5)	4.5 (7.9)	4.3 (18.3)	5.5 (41.1)	4.1 (24.1)	1.34 (3.9)	30.9 (5.4)
	30FA/5SF	742 (4.1)	0.0136 (12.6)	4.2 (20.2)	3.4 (70.7)	6.5 (37.9)	3.0 (14.1)	1.54 (1.9)	36.6 (1.3)
	30FA/7SF	668 (5.3)	0.0118 (12.0)	3.9 (27.5)	2.1 (80.8)	9.5 (12.7)	2.4 (17.7)	1.23 (4.4)	43.4 (1.5)
180	20FA/5SF	488 (2.3)	0.0092 (8.3)	2.8 (-)	3.7 (-)	12.2 (16.2)	3.8 (7.4)	1.73 (4.4)	30.9 (4.5)
	20FA/7SF	396 (7.2)	0.0060 (7.2)	2.9 (12.4)	2.0 (32.6)	8.6 (19.0)	2.9 (9.8)	1.00 (9.7)	32.5 (5.5)
	30FA/5SF	476 (5.1)	0.0087 (13.5)	3.3 (-)	3.7 (-)	34.4 (16.3)	2.1 (3.4)	1.21 (4.6)	36.8 (1.8)
	30FA/7SF	440 (4.7)	0.0100 (11.2)	5.1 (44.4)	4.4 (73.1)	35.1 (52.4)	2.3 (6.1)	1.51 (11.3)	37.5 (5.9)

Note: The values in parentheses denote coefficient of variation (CV) in %. No CV could be provided in cases when one of the replicate specimens were leaking during the absorptivity test.

To better illustrate the influence the of amount of fly ash and silica fume in the mixture and the age of concrete on the RCP and the RMP test results (denoted as RMP), two regression models have been developed. The values of coefficients of determination were 0.98 and 0.94 for the RCP and RMP models, respectively. These prediction models were used to create surface plots of RCP and RMP as a function of fly ash and silica fume content at three testing ages for Series I mixtures (Fig. 4.3). It can be seen that while an increase in silica fume content decreases both RCP and RMP values, the increase in fly ash content increases both of those parameters. Furthermore, the parallelism of the RCP and RMP response surfaces confirms the similarity of the mechanisms underlying these two test methods.

The strong relationship between results obtained using RCP and RMP tests, suggested earlier by Stanish et al. (2000), was further enforced by high coefficient of determination $R^2 = 0.90$ in the linear model linking RCP and RMP (Fig. 4.4).

Fig. 4.3 – Response surfaces for Series I mixtures for: (a) RCP; (b) rate of penetration

The change in initial sorptivity and absorptivity with time is presented in Fig. 4.5. Several interesting and rather unexpected features can be observed. To start with, it should be realized that these results were plotted on the axes scaled differently for absorptivity and sorptivity. That implies that the sorptivity values were initially (at 28

days) comparable with absorptivity ($5.4\text{--}7.6\text{ mm/s}^{1/2}$ and $3.3\text{--}7.2\text{ mm/s}^{1/2}$, respectively), but with time they became roughly 3 to 10 times higher than absorptivity, depending on the fly ash content.

Fig. 4.4 – Relationship between the rate of penetration and RCP

Fig. 4.5 – Initial absorptivity and sorptivity with time

As suggested by several authors (Bentz et al. 2001, Hall 1989), capillary suction is the driving mechanism for the water penetration, while in the case of ponding from the top of a sample (absorptivity test method) the inflow is additionally gravity-driven. Since the magnitude of capillary pressure is several orders of magnitude higher than that of forces of gravity, it follows that the gravity-driven effect is negligible (Hall 1989, Lockington and Parlange 2003). This, in turn, implies that sorptivity and absorptivity should be, in general, of the same order of magnitude. This theory, however, was not confirmed in this study, perhaps due to the refinement of the capillary pore system by mineral admixtures. Therefore, it appears that, at least at the internal RH higher than that required by ASTM C 1585 (ASTM 2006g), the mechanisms of sorptivity and absorptivity may not be the same. In addition, while absorptivity decreases with time for all mixtures (neglecting two points representing averages of two scattered results), the

general trend implies that sorptivity increases with time (see Fig. 4.5). For the two mixtures containing 20% of fly ash, the increase in sorptivity was linear and rather minor. However, for the two mixtures with 30% fly ash there was a very marked increase in the initial sorptivity values between 56 and 180 days. It is believed that this phenomenon can be attributed to changes in pore structure of mixtures containing higher percentage of fly ash resulting from prolonged curing period.

To examine the reason for this sharp increase in sorptivity more closely, the full set of sorptivity data obtained for 30FA/5SF mixture at 56 and 180 days is presented in Fig. 4.6(a) and (b), respectively. Probably due to limited capacity of the pore system, the relationship representing initial I vs. $t^{1/2}$ obtained at 180 days becomes somewhat non-linear toward the end of the initial monitoring period of 6 hours (Fig. 4.6(b)). Accordingly, as suggested by Hall (1989) and Taha et al. (2001), the theory of unsaturated medium becomes inapplicable, and only the linear portion of the plot should be considered.

Fig. 4.6 – Water intake vs. time^{1/2} in the sorptivity test for mixture 30FA/5SF at: (a) 56; (b) 180 days (single but different specimen tested at each age)

The 56-day initial and secondary absorptivity results obtained for INDOT class C (11.4 and 7.1 mm/s^{1/2}, respectively) were substantially higher than the highest

values achieved by 20FA/7SF mixture (4.5 and 4.3 mm/s^{1/2}). Similarly, the 56-day initial and secondary sorptivity values obtained by INDOT class C mixture (20.8 and 14.5 mm/s^{1/2}) were much higher than the highest results attained by any of ternary mixtures.

The least squares method was used to examine the relationships between sorptivity or absorptivity results and RCP or RMP values. The only strong relationship that was found to exist was that between the initial absorptivity (after excluding the two outlying points) and both RCP and RMP results (Fig. 4.7). The observed relationships are linear and yield a relatively high coefficient of determination of 0.82 and 0.81 for initial absorptivity vs. RCP and RMP, respectively.

All mixtures displayed relatively low (i.e. less than 2.0×10^{-12} m²/s) diffusion coefficients at all selected testing ages (Table 4.4). It can be noticed that except for the 180-day diffusion coefficients of 20FA/5SF and 30FA/7SF mixtures, there is an expected decreasing tendency of diffusion coefficient with time. The reason for the two aforementioned unexpected increases of diffusion coefficient with time is unknown. However, it should be realized that according to ASTM C 1556 (ASTM 2004a) the apparent diffusion coefficient results of two properly conducted tests should not differ by more than 39.8% of the mean value. The difference between the minimum or maximum average diffusion coefficient results obtained by any mixture at any testing age (1.00 m²/s and 1.87 m²/s, respectively) is 30.1% of their mean value, which is smaller than the precision of the test method itself. This implies that the precision of bulk diffusion test is relatively low and the migration-based tests (RCP or RMP) are preferred for comparison purpose of diffusivity of various mixtures. In addition, unlike RCP or RMP which provide a resistance of concrete to chloride-ion penetration almost instantaneously, the bulk diffusion test is very time-, cost- and labor-consuming (Berke and Hicks 1992, Castelotte and Andrade 2006).

Additionally, a relatively strong relationship was found to exist between the apparent diffusion coefficient values and RCP results and initial absorptivity (as shown in Fig. 4.8(a) and (b), respectively) as well as RMP results (not shown). However, these relationships were strong only the outlying values, resulting either from discussed

previously unexpected increase in 180-day diffusion coefficient values for two of the mixtures studied or poor repeatability of absorptivity test, were removed.

Fig. 4.7 – Relationship between initial absorptivity and (a) RCP;
(b) rate of penetration (Series I)

Fig. 4.8 – Relationship between apparent diffusion coefficient and (a) initial absorptivity
and (b) RCP (Series I)

4.3.2.2 Series II Mixtures

Based on the fact that, for Series I mixtures, the initial absorptivity was decreasing with time, which is a desirable trend (as opposed to initial sorptivity exhibiting the opposite trend), whereas the time-consuming bulk diffusion test did not yield uniform results and since higher repeatability of RCP over RMP was observed, it was decided to employ RCP and absorptivity tests in the Series II mixtures. This series included testing specimens at two different ages (14 - 21 days and 90 days) after subjecting them to 4 different curing conditions (a - d) as described in section on curing procedures.

Table 4.5 contains a summary of test results. While both mixtures containing 20% fly ash showed similar performance, 20FA/7SF mixture had generally slightly lower RCP and initial absorptivity values than 20FA/5SF mixture. Nevertheless, the performance (particularly at the early age) of 20% FA mixtures was significantly better than that of both mixtures with 30% FA, of which 30FA/7SF mixture exhibited generally slightly lower RCP and initial absorptivity values than 30FA/5SF mixture.

It can also be seen that, especially at the early age, the initial curing condition had a substantial impact on RCP and initial absorptivity of the examined ternary mixtures. As expected, while air drying (no curing) resulted in the highest results (for both tests) than any other curing method, curing under wet burlap for 7 days gave the lowest values. Furthermore, curing using a curing compound for 7 days appeared to give comparable results to 3 day wet burlap curing.

To illustrate the relationship between the composition of concrete, testing age and either RCP or initial absorptivity results, the least squares method was utilized to develop regression models linking these parameters. These models yielded very high values of coefficient of determination ranging from 0.94 to 0.99. The fly ash content and the age of testing were incorporated in all models, whereas the silica fume content was not found to be statistically significant. Furthermore, the interactions between the amount of fly ash and the testing age were present in most of the models.

Table 4.5 – Summary of test results from Series II mixtures

Mixture	Age (days)	Curing condition	Charge passed (RCP) (coulombs)	Initial absorptivity $\times 10^{-4}$ (mm/s ^{1/2})	Secondary absorptivity $\times 10^{-4}$ (mm/s ^{1/2})
20FA/5SF	14	air drying	2800 (1.0)	133.0 (-)	7.5 (-)
	17	3 days burlap	1412 (10.3)	56.2 (6.4)	9.7 (12.5)
	21	7 days cur. comp.	1385 (26.9)	44.2 (10.1)	3.8 (18.6)
	21	7 days burlap	1223 (8.4)	34.7 (2.7)	8.7 (16.3)
20FA/7SF	14	air drying	2524 (3.7)	132.2 (15.2)	6.0 (13.1)
	17	3 days burlap	1350 (4.1)	58.3 (6.1)	11.6 (35.4)
	21	7 days cur. comp.	1081 (5.3)	48.8 (2.3)	2.2 (32.1)
	21	7 days burlap	1093 (8.0)	40.7 (5.2)	9.0 (7.9)
30FA/5SF	14	air drying	5302 (7.2)	150.6 (3.0)	10.0 (39.1)
	17	3 days burlap	2015 (4.0)	74.4 (2.9)	16.6 (12.8)
	21	7 days cur. comp.	1943 (3.3)	98.8 (-)	10.2 (-)
	21	7 days burlap	1487 (1.2)	61.2 (4.3)	10.5 (17.5)
30FA/7SF	14	air drying	4816 (3.6)	179.4 (6.9)	6.5 (25.2)
	17	3 days burlap	1945 (1.5)	67.2 (5.1)	7.5 (6.6)
	21	7 days cur. comp.	1854 (2.8)	91.9 (2.3)	10.0 (30.6)
	21	7 days burlap	1441 (1.7)	63.5 (7.1)	13.8 (22.1)
20FA/5SF	90	air drying	690 (0.8)	43.6 (1.3)	7.2 (5.9)
	90	3 days burlap	432 (5.6)	31.5 (1.3)	22.5 (24.3)
	90	7 days cur. comp.	609 (22.2)	34.5 (5.7)	17.6 (4.4)
	90	7 days burlap	428 (4.4)	22.2 (1.9)	15.9 (5.8)
20FA/7SF	90	air drying	571 (9.6)	50.4 (7.3)	8.0 (0.0)
	90	3 days burlap	413 (2.0)	32.9 (9.9)	20.4 (-)
	90	7 days cur. comp.	407 (1.7)	40.2 (10.0)	16.7 (6.4)
	90	7 days burlap	334 (15.8)	27.2 (0.5)	16.3 (16.5)
30FA/5SF	90	air drying	984 (26.1)	108.9 (5.1)	10.8 (9.9)
	90	3 days burlap	558 (7.2)	58.2 (1.5)	9.0 (24.5)
	90	7 days cur. comp.	630 (7.3)	63.5 (5.0)	14.6 (24.2)
	90	7 days burlap	552 (4.2)	38.7 (11.0)	26.1 (7.9)
30FA/7SF	90	air drying	1421 (9.1)	103.5 (2.3)	12.6 (6.2)
	90	3 days burlap	682 (2.3)	57.4 (3.7)	20.5 (11.4)
	90	7 days cur. comp.	687 (6.9)	59.8 (1.9)	13.2 (5.9)
	90	7 days burlap	567 (11.1)	41.7 (14.8)	21.8 (5.2)

Note: The values in parentheses denote coefficient of variation (CV) in %. No CV could be provided in cases when one of the replicate specimens was leaking during the absorptivity test.

Using the developed regression models, the surface plots were made for RCP and initial absorptivity as a function of age and fly ash content for all four curing conditions examined (Fig. 4.9). It is evident from these plots that any form of curing, if present, will significantly lower the early age RCP when compared to complete absence of curing. However, the initial curing becomes less significant for RCP determined at later ages. The response surface for air drying is located in some distance from the

surfaces for other three curing regimes. The surfaces for all latter cases appear to almost overlap. Examination of the surface plots for initial absorptivity resulted in similar conclusions as these drawn for RCP. However, the negative effect of lack of curing (air drying) was more evident even if the specimens were tested at 90 days.

Fig. 4.9 – Response surfaces for Series II mixtures for: (a) RCP; (b) initial absorptivity

Further statistical analysis was carried out and relatively strong relationship was found between the initial absorptivity and RCP (Fig. 4.10(a)). This relationship appeared to be statistically valid only if the early age (up to 21 days and late age (90 days) test results were treated separately (Fig. 4.10(a)). In other words, the evident separation between the data points for respective testing ages exists, which might be indicative of the fact that absorptivity characteristics do not change with time (and thus degree of hydration) as rapidly as RCP. It might also mean that despite the efforts made to bring all absorptivity specimens to the same moisture content, the 90-days old (late age) specimens still exhibited lower internal RH. Unfortunately, this parameter was not measured in the study. Finally, no strong relationship was found between the secondary absorptivity and RCP (Fig. 4.10(b)).

Fig. 4.10 – Relationship between RCP and (a) initial absorptivity; (b) secondary absorptivity (Series II)

The repeatability of initial absorptivity test has greatly improved for the Series II mixtures, and the average coefficient of variation (CV) was only 5.5%, as opposed to 25.7% obtained on Series I mixtures. This can be attributed to achieving the equilibrium condition (lower internal moisture content and more uniform distribution) by the specimens from Series II mixtures, as opposed to specimens from Series I mixtures. The internal RH of the latter specimens appeared to be much higher, and perhaps the lack of uniform moisture distribution might have induced serious variability in test results. Interestingly, the CV for initial absorptivity in Series II mixtures is actually lower than that for RCP test (7.2%).

4.3.3 Conclusions

The following conclusions have been drawn on the basis of the tests conducted on two series of mixtures:

- In general, all ternary mixtures exhibited incomparably better transport properties (in terms of resistance to chloride-ion penetration and rate of water absorption) than the counterpart INDOT class C mixture.

- Low chloride diffusion coefficients were obtained for all ternary mixtures studied. While the diffusion coefficient generally tended to decrease with curing time, an unexpected increase occurred between the 56th and 180th day in two cases, which questions the efficacy of bulk diffusion test when used for comparative purposes.
- Sorptivity was found to increase with time for the mixtures containing 30% fly ash, potentially reflecting progressive refinement in pore structure resulting in much greater capillary suction forces. However, in the applications such as bridge decks, where ingress of aggressive substances from the bottom is only of minor concern, absorptivity test may be more desirable as it represents field exposure more realistically.
- The preconditioning procedure proposed in ASTM C 1585 does likely not result in achieving the intended internal RH of 50-70% for continuously moist cured specimens, thus making results obtained from such cases somewhat incomparable with the results for partially dried specimens prior to conditioning.
- Migration-type tests had better reproducibility than the absorption-based tests when conducted on continuously moist cured specimens. However, when brought to sufficiently low moisture content, the repeatability of absorptivity test can be as good, or even better, than that of RCP test.
- While a strong linear relationship exists between the RCP and RMP results, the RCP test exhibited better repeatability than RMP at all testing ages.
- Relatively strong relationship appears to exist between RCP and initial absorptivity, but only for the groups of specimens which received the same (or very similar) curing treatment prior to testing.
- Initial curing conditions appeared to have very significant effect on the early age RCP and initial absorptivity test results. With the increase in the curing time these differences gradually diminished.

- Regardless of the amount of fly ash and silica fume in the ternary mixture, curing under wet burlap for 7 days provided the best performance, whereas air drying provided the worst performance in terms of resistance to chloride-ion penetration and absorptivity.

4.4 Effect of Mixture Composition and Initial Curing Condition on Salt Scaling

Resistance of Ternary OPC/FA/SF Mixtures

4.4.1 Introduction

While incorporation of SCM's, particularly fly ash and ground granulated blast furnace slag (GGBFS), in concrete often results in significant enhancements of its various properties (long-term strength, elastic modulus, permeability and chemical resistance), it has also been reported (Bleszynski et al. 2002, Çopuroğlu 2006, Deja 2003, Krishnan et al. 2006; Toutanji et al. 2005, Pigeon et al. 1996) that freeze-thaw (F-T) and scaling resistance of these concretes may not be satisfactory (as determined using the standard laboratory test procedures). This problem is particularly likely to occur when the exposure to low temperature takes place at relatively early age (before the pozzolanic reaction can fully develop).

Although ACI 318 imposes a limit of 35% on the total percentage of fly ash and silica fume in the ternary concrete exposed to deicing chemicals (ACI 2005), until now no clear recommendations have been developed regarding the optimum content of silica fume and the permissible content of fly ash in OPC/FA/SF mixtures that will ensure good resistance to salt scaling.

Many researchers have emphasized the importance of initial curing of concrete, particularly of that containing slowly reacting mineral admixtures such as fly ash and GGBFS (Khan and Ayers 1993, Shafiq and Cabrera 2004). Waktola et al. (2005) showed that a mixture with class F fly ash substantially increased scaling resistance with moist curing durations if “moisture stabilization was ensured prior to the beginning of freeze-thaw cycles”. Similar conclusion were drawn by Krishnan et al. (2006), who also reported beneficial effects of prolonged curing of mixtures incorporating fly ash or blast

furnace slag. However, virtually no data is available on the influence of curing conditions on scaling resistance of ternary mixtures containing both fly ash and silica fume. The in-situ inspection of the pilot HPC bridge deck incorporating fly ash and silica fume conducted after the first winter, described in section 5.6.2, revealed that such ternary system may be relatively sensitive to poor curing practices.

The presented research had several distinctive objectives. The first goal was to determine the optimum contents of fly ash and silica fume in the ternary system with respect to scaling resistance. The other objective of this work was to assess sensitivity of the OPC/FA/SF ternary mixtures to the type of initial curing conditions. Four curing conditions, namely air drying (lack of curing), 3 days under wet burlap, 7 days covered with curing compound and 7 days under wet burlap, were studied. The last objective was to verify whether the late age exposure to deicing salt (simulating the case of e.g. bridge deck constructed during summer) results in significant improvement of scaling resistance compared to the early age exposure (simulating the case of bridge deck constructed in the late fall).

4.4.2 Experimental Details

4.4.2.1 Specimens Preparation

One 76-mm thick slab with the planar dimensions of 360×255 mm was prepared from every mixture for each curing condition. The slabs were cast in one lift and consolidated using a steel rod and a rubber mallet. The surface was finished with several passes of wooden strike-off board. Thanks to the use of high range water reducer and fly ash all mixtures offered very good finishability while, due to incorporation of silica fume and relatively low w-cm ratio, they did not show increased bleeding tendency. After finishing, a single pass of medium-stiffness brush was used to apply texture to the surface. The specimens were demolded 24 hours after mixing. Each slab was dry-cut before testing to obtain two specimens of equal area of 255×180 mm.

4.4.2.2 Curing Procedures

Immediately after completion of finishing operations, the test specimens from each group of mixtures were subjected to the following curing regimes: (a) air drying (storage at 23°C and 50% RH), (b) 3 days under wet burlap and covered with plastic sheet, (c) 7 days covered with curing compound (curing compound applied to all surfaces (except the bottom) immediately after demolding and removed after 7 days using a wire brush), and (d) 7 days under wet burlap covered with plastic sheet. Special care was taken to avoid touching of the surface of freshly placed concrete with wet burlap to prevent local increase in the water-cementitious materials ratio at the specimen surface.

The scaling test specimens were divided into two subgroups: one tested at the early age (14 - 21 days) and the other tested at later age (90 days). After being subjected to one of the initial curing regimes (a - d), all specimens in the early age subgroup were subjected to drying at 23°C and 50% RH for 14 days. Depending on the length of initial curing regime (a - d) used, this resulted in initiating the scaling testing at 14, 17 or 21 days. After completion of one of the initial curing periods (a - d), the specimens in the late (90 days) subgroup were all dried at 23°C and 50% RH until the age of 69 days. Subsequently, they were all “re-saturated” by placing them in a moist room (100% RH) for 7 days. Following this resaturation period, all specimens were air dried for 14 days at 23°C and 50% RH before being exposed to freeze-thaw cycles. This procedure was employed in an attempt to obtain comparable degree of saturation in all specimens tested, as this parameter is known to have strong influence on scaling test conducted according to ASTM C 672 (Jacobsen et al. 1997, Krishnan et al. 2006, Waktola et al. 2005). Despite these efforts, some discrepancy in moisture content between the specimens from different testing groups might have not been entirely eliminated (see “Discussion” in section 4.4.4).

4.4.2.3 Test Procedures

All salt scaling tests were performed following ASTM C 672 (ASTM 2006c) except for the curing method (discussed earlier) and the use of pre-programmed environmental

chamber capable of producing one full freeze-thaw cycle every 24 hours. The freeze-thaw cycle consisted of lowering the temperature from +22.0°C to -8.0°C within the first four hours of the cycle, followed by further reduction of temperature to -18.0°C within next five hours. That temperature was maintained for four hours, after which it was gradually increased to +22.0°C over a period of six hours. The temperature was maintained at that level for five hours. That 24-hours-based freeze-thaw cycle was successfully used in the previous study dealing with scaling resistance of mixtures containing fly ash and GGBFS (Krishnan et al. 2006).

The test specimens were subjected to 50 previously described cycles of freezing and thawing while being ponded with about 6 mm-thick layer of 4.0% calcium chloride solution placed in the dike installed at the top of test specimens. After every 5 cycles, the salt solution was removed from the surface of the slabs, the scaled-off material was carefully collected and the visual rating was conducted (as per ASTM C 672). The collected scaled-off material was dried in the oven at 105°C for about 5 days, weighted and used to determine the cumulative scaled-off mass (in kg/m²). The fresh salt solution was then placed on the top of the slabs and the exposure to freeze-thaw cycles continued.

It should be realized that, while being commonly used for comparative purposes in laboratory studies, the ASTM C 672 procedure is often criticized for being far too severe and thus for providing poor correlation with field performance of concrete with respect to salt scaling (Bleszynski et al. 2002; Boyd and Hooton 2007; Krishnan et al. 2006).

4.4.3 Test Results

The test results, along with the mean values and corresponding coefficients of variation (CV), are given in Table 4.6 and Table 4.7 for early and late age scaling tests, respectively. Due to subjective nature of the visual scaling rating method used in ASTM C 672, the results of the present tests will be thereafter discussed in terms of the cumulative mass loss, which was found to be a more consistent measure of the rate and the extent of scaling. Typically, the 50 F-T cycles cumulative mass loss of 0.80 kg/m² (or higher) is used as a test failure criterion (MTO 1989).

Table 4.6 – Early age (14, 17 or 21 days) scaling test results

FA (%)	SF (%)	Curing Condition	Cumulative Scaling after 50 Cycles			Visual Rating after 50 Cycles						
			Mass (kg/m ²)	Avg.	CV (%)	Rank	Avg.					
20	5	air drying	0.27	0.35	33.4	1	1.5					
			0.43			2						
		3 days burlap	0.77	0.89	18.5	2	2.0					
			1.01			2						
		7 days curing compound	0.29	0.27	8.8	1	1.0					
			0.25			1						
	7 days burlap	0.56	0.42	46.9	2	2.0						
		0.28			2							
		20			7		air drying	0.47	0.47	0.2	2	2.0
								0.46			2	
3 days burlap	0.38		0.44	19.2		1	1.5					
	0.50					2						
7 days curing compound	0.14		0.15	7.4		1	1.0					
	0.16					1						
7 days burlap	0.47		0.31	69.9	2	1.5						
	0.16				1							
	30				5		air drying	1.34	1.24	11.9	3	3.0
								1.13			3	
3 days burlap		1.19	1.17	1.6		3	3.0					
		1.16				3						
7 days curing compound		0.51	0.47	12.2		2	2.5					
		0.43				3						
7 days burlap		0.87	0.95	12.9	3	3.0						
		1.04			3							
		30			7		air drying	1.56	1.46	9.6	3	3.0
								1.36			3	
3 days burlap	1.13		1.15	3.2		3	3.0					
	1.18					3						
7 days curing compound	0.40		0.38	8.7		3	2.5					
	0.35					2						
7 days burlap	0.76		0.90	22.9	3	3.0						
	1.05				3							

The scaling test results exhibit relatively high variability (Table 4.6 and Table 4.7), even though the two replicate test specimens were obtained from one larger slab. It is believed that this relatively high degree of scatter of test results may reflect the non-uniform distribution of aggregate particles in the near-surface region of the slabs. Excessive variability in scaling test results, particularly at low overall level of scaling, was previously reported by Boyd and Hooton (2007), Jacobsen et al. (1997) and Marchand et al. (1996). The specific aspects of the experimental results are discussed in the following sections.

Table 4.7 – Late age (90 days) scaling test results

FA (%)	SF (%)	Curing Condition	Cumulative Scaling after 50 Cycles			Visual Rating after 50 Cycles						
			Mass (kg/m ²)	Avg.	CV (%)	Rank	Avg.					
20	5	air drying	0.95	0.77	32.0	2	2.0					
			0.60			2						
		3 days burlap	0.36	0.33	14.7	1	1.0					
			0.29			1						
		7 days curing compound	0.47	0.44	7.3	2	2.0					
			0.42			2						
	7 days burlap	0.27	0.27	2.0	1	1.0						
		0.28			1							
		20			7		air drying	1.03	0.61	96.3	2	1.5
								0.20			1	
3 days burlap	0.17		0.17	5.5		1	1.0					
	0.18					1						
7 days curing compound	0.22		0.23	5.7		1	1.0					
	0.24					1						
7 days burlap	0.25		0.26	7.8	1	1.0						
	0.28				1							
	30				5		air drying	1.36	0.94	63.0	3	2.5
								0.52			2	
3 days burlap		0.18	0.23	29.1		1	1.0					
		0.28				1						
7 days curing compound		0.35	0.29	28.7		2	2.0					
		0.23				2						
7 days burlap		0.33	0.24	49.1	2	1.5						
		0.16			1							
		30			7		air drying	1.97	1.80	14.1	3	3.0
								1.62			3	
3 days burlap	0.88		0.74	27.7		2	2.0					
	0.60					2						
7 days curing compound	0.83		0.78	9.8		2	2.0					
	0.72					2						
7 days burlap	0.90		0.94	5.8	2	2.0						
	0.98				2							

4.4.4 Discussion

4.4.4.1 Comparison of Scaling Progress Curves

The progress of scaling with the number of F-T cycles is presented in Fig. 4.11 and Fig. 4.12, respectively, for specimens from the early and the late testing age groups. It appears that for the specimens tested at the early age, the scaling was initiated after approximately 8 F-T cycles for mixtures with 20% fly ash (Fig. 4.11(a)-(b)), whereas it started sooner (only after about 5 cycles) for mixtures with 30% fly ash (Fig. 4.11(c)-(d)). This difference reflects potentially the differences in permeabilities of pastes of the ternary systems under evaluation. Further, for 3 out of 4 curing regimes applied, scaling for mixtures with 30% FA occurred at much higher rate compared to the mixtures with 20% FA. This, in turn, probably corresponds to the difference in the tensile strength of

the paste, i.e. both mixtures with 30% fly ash most certainly developed much lower tensile strength compared to the mixtures with 20% fly ash.

Fig. 4.11 – Mass loss of the early age salt scaling mixtures: (a) 20FA/5SF; (b) 20FA/7SF; (c) 30FA/5SF; (d) 30FA/7SF

Fig. 4.12 – Mass loss of the late age salt scaling mixtures: (a) 20FA/5SF; (b) 20FA/7SF; (c) 30FA/5SF; (d) 30FA/7SF

It is further worth noting that within each group of mixtures with the same fly ash content (20% or 30%) similar scaling trends can be observed (Fig. 4.11(a)-(b) and Fig. 4.11(c)-(d)). In the case of both 20% FA mixtures (Fig. 4.11(a) and (b)) the scaling curves for all curing regimes (except “3 days burlap” for 20FA/5SF mixture) seem to level off after about 20-25 cycles with the maximum amount of scaled material of about 0.4 kg/m^2 . Similarly, for both mixtures with 30% FA content (Fig. 4.11(c) and (d)) the

curves corresponding to “7 days curing compound” method appear to stabilize after about 40 cycles with the total amount of material scaled of 0.4-0.5 kg/m². Examination of the results for other three curing conditions also reveals marked similarities in trends of the mass loss rates and the total amounts of material scaled (respectively for each curing regime) between 30FA/5SF and 30FA/7SF mixtures (Fig. 4.11(c)-(d)).

For the specimens tested at the late age, it can be seen from Fig. 4.12 that the scaling curves for mixtures 20FA/5SF (Fig. 4.12(a)), 20FA/7SF (Fig. 4.12(b)) and 30FA/5SF (Fig. 4.12(c)) exhibit very similar trends for the corresponding curing conditions. However, mixture 30FA/7SF (Fig. 4.12(d)) experienced much higher scaling compared to other three mixtures, regardless of the curing condition. It can be also noticed that for all four mixtures, air drying (essentially implying lack of any curing actions) resulted in substantially greater scaling compared to other curing conditions. This indicates that once the initial curing is compromised at the early age, the surface of concrete may not be capable of developing good scaling resistance even after some remedial actions (supply of moisture, e.g. from rain) are taken at later age. The potential explanation for the fact that the air dried specimens tested at the late age underwent even more scaling than those tested at the early age may be related to the differences in the mode of the early and late age scaling. This issue is broadly discussed in section 4.4.4.3.

4.4.4.2 Effect of Mixture Composition and Initial Curing Condition on Scaling

The statistical analysis aimed at detailed examination of the effect of mixture composition and initial curing condition on scaling was performed separately on the data collected for the early and late age subgroups of specimens. All statistical tests have been carried out at significance level $\alpha = 0.05$. Since two variables were present in each data set, namely mixture composition and curing condition, a two-way ANOVA analysis was conducted. The results of the analysis are discussed separately for the early and late age groups of specimens in the following sections.

Scaling of early age group of specimens

The two-way ANOVA analysis revealed that, in addition to both variables being statistically significant (p-value < 0.001), there was also a statistically significant interaction between the mixture composition and curing condition (p-value = 0.001). Thus, it is not possible to properly interpret the main effects of the two variables. Accordingly, all pairwise comparisons between the curing conditions were performed separately for each mixture composition. The results are presented in Table 4.8.

Table 4.8 – Pairwise multiple comparisons of curing conditions (Tukey Test) for early age scaling results

	Air Drying	3 Days Burlap	7 Days Curing Compound	7 Days Burlap
Air Drying	n/a	D N N N	N N D D	N N N D
3 Days Burlap	–	n/a	D N D D	D N N N
7 Days Curing Compound	–	–	n/a	N N D D
7 Days Burlap	–	–	–	n/a

Note: Symbol D (statistically different) indicates that the two curing conditions compared produced statistically different results. Similarly, the symbol N indicates lack of statistically different response. The order of D and N symbols corresponds to the following mixture types: 20FA/5SF, 20FA/7SF, 30FA/5SF, and 30FA/7SF

It can be seen that, as it was apparent from Fig. 4.11(a), for 20FA/5SF mixture only “3 days burlap” curing condition was identified to differ significantly from the other curing conditions (as denoted by letter “D” in the corresponding entry in Table 4.8). Interestingly, for 20FA/7SF mixture there is no statistically significant difference between any of the curing conditions (notice “N” symbol for the corresponding entries in Table 4.8). That result implies that the early age scaling resistance of this mixture may not depend on the initial curing regime. Therefore, the very low amount of scaling observed after 50 cycles, regardless of the curing condition, proves superiority (in terms of durability) of this mixture already observed during preliminary optimization (section 3.4). This is further confirmed by the fact that for 20FA/7SF mixture all scaling curves are grouped in relatively tight bundle (Fig. 4.11(b)). This perhaps would have been also

true for 20FA/5SF mixture, if the specimens cured under wet burlap for 3 days did not exhibit the unexpected and unexplainable higher amount of scaling mentioned previously.

Of the four curing methods, the only method statistically different for the 30FA/5SF mixture was 7 days curing compound method. The same observation also applies to the 30FA/7SF mixture. However, for this particular mixture there was also a significant difference between the air drying and 7 days wet burlap curing methods.

Fig. 4.13 – Cumulative mass loss after 50 cycles of salt scaling at (a) early age; (b) late age

A better illustration of comparison of the effects of the considered variables on the scaling amount for the four examined mixtures has been provided in Fig. 4.13. The clear lack of parallelism for most of the line plots representing different curing conditions in the early age group (Fig. 4.13(a)) confirms the existence of the interaction effects between the mixture composition and curing condition.

Scaling of late age group of specimens

The results of pairwise comparisons of the effect of curing conditions on scaling for the late age group of specimens are given in Table 4.9. For this group of specimens no interaction effect between the mixture composition and curing condition was found by

means of two-way ANOVA analysis. This is illustrated in Fig. 4.13(b) by almost perfect parallelism of line plots corresponding to different curing conditions. Therefore, since the effect of curing conditions is independent of the mixture composition (and vice versa), the effect of each factor can be generalized (as indicated by single entries in the cells of Table 4.9).

The results presented in Table 4.9 further confirm the previously stated observation that air drying led to significantly different results compared with the other three curing regimes, which themselves were not statistically different from each other. Therefore, it appears that for long-term scaling resistance, the type and duration of curing may not matter, as long as some curing actions have been taken initially, i.e. concrete has been not allowed to dry.

Table 4.9 – Pairwise multiple comparisons of curing conditions (Tukey Test) for late age scaling results

	Air Drying	3 Days Burlap	7 Days Curing Compound	7 Days Burlap
Air Drying	n/a	D	D	D
3 Days Burlap	–	n/a	N	N
7 Days Curing Compound	–	–	n/a	N
7 Days Burlap	–	–	–	n/a

Note: Symbol D (statistically different) indicates that the two curing conditions compared produced statistically different results. Similarly, the symbol N indicates lack of statistically different response.

Furthermore, only 30FA/7SF mixture was determined to be different from the other three mixtures which, on the other hand, were found to give statistically insignificant differences in scaling between in each other. This is evident from Fig. 4.12(b), which shows very comparable scaling results for mixtures 20FA/5SF, 20FA/7SF and 30FA/5SF but substantially higher results for 30FA/7SF mixture for each of the curing conditions. Hence, it should be realized that despite poor scaling resistance of 30FA/5SF mixture exposed at the early age, the same mixture, when exposed at the later age, displayed a very good scaling resistance (Fig. 4.13(a) and (b)).

Another important aspect regarding the effect of initial curing regime on scaling deals with application of curing compound. That curing method appeared to provide the best resistance to scaling for all mixtures exposed at the early age and very good scaling resistance of specimens exposed at the late age (Table 4.6). This observation is in agreement with the results of the study by Krishnan et al. (2006) where significant reduction in scaling was observed for specimens treated with the curing compound. This reduction was attributed mostly to blocking of the salt and moisture ingress by a thin layer of curing compound. However, since in the current study the curing compound was removed from the surface after 7 days, the surface was as-finished at the time of testing. It should be noticed that at the late age the curing compound method resulted in amount of scaling very comparable to that exhibited by specimens cured with wet burlap for 3 days and 7 days (Fig. 4.13(b)). This was despite the significant difference between the scaling of specimens cured with a curing compound and all other curing conditions observed at the early age (Fig. 4.13(a)). This suggests that much better (compared to other curing methods) performance of specimens cured with a curing compound and tested at the early age (21 days) might be related to much lower degree of saturation of the pore system in the very top layer caused by the use of curing compound. This, in turn, might have led to reduction of the salt concentration gradient across the specimen resulting in lesser scaling damage. However, additional research is needed to verify this hypothesis.

4.4.4.3 Effect of Exposure Age on the Mode of Scaling

Intuitively, the extended curing period (here represented by late age exposure) should substantially reduce the extent of scaling due to more complete hydration, leading to densification of microstructure of cement paste and thus resulting in reduction in permeability of the system (Saric-Coric and Aïtcin 2002). Additionally, the tensile strength of concrete increases with hydration time, which should also contribute to improvement in resistance to scaling. However, it should be also realized that while the increase in the tensile strength provides better resistance to scaling in terms of time to

initiation, once the scaling is initiated, its extent in terms of mass loss can be as big or even bigger than that observed in cases of weak paste.

The current study showed very distinctive differences in the mode of scaling failure of specimens tested at early and late age. For the early exposure age group of specimens the scaled residue was in the form of thin flakes of paste (Fig. 4.14(a)). On the other hand, the scaled residue from the late exposure group of specimens was in the form of relatively large and thick chips (pop-outs) of paste (Fig. 4.14(b)). Frequently, the depth of these pop-outs extended to the coarse aggregate particles located below the surface of the slabs.

Fig. 4.14 – Typical appearance of scaled residue from: (a) early age test; (b) late age test

The characteristic differences in the appearance of the top surface of the scaling slabs after 50 cycles from the early and late age groups are illustrated in Fig. 4.15 (a) and (b), respectively. It is interesting to note that both specimens were made from 30FA/5SF mixture and subjected to air drying (as a curing regime). In addition, both of these specimens experienced nearly identical amount of scaling (1.34 kg/m^2 and 1.36 kg/m^2 , respectively, for early and late age specimen). However, it can be seen that the

early age scaling occurred in the form of shallow depressions relatively uniformly distributed over the entire surface of the slab (Fig. 4.15(a)). In contrast, the scaling on the surface of the late age specimen (Fig. 4.15(b)) manifested itself in the form of relatively deep pop-outs that appeared to be concentrated only in the specific areas (typically over the underlying coarse aggregate particles). Based on these observations, it is concluded that the early age scaling occurred mainly as a result of the paste failure due to tensile stresses imposed by the brine ice formed on its surface. On the contrary, the late age scaling appeared to be mostly caused by failure at the paste-aggregate interface.

Fig. 4.15 – The 30FA/5SF specimens (subjected to initial air drying) after 50 cycles of freezing and thawing at: (a) early age; (b) late age

The more concentrated and random nature of late age scaling was found to introduce high variability to the test results (Table 4.7). Interestingly, the biggest discrepancy between the replicate specimens was observed for the air dried specimens prepared from 20FA/7SF and 30FA/5SF mixtures. It is, however, not clear whether or not the air drying as a curing regime contributed to the variation in the test results. For instance, for one specimen prepared from 20FA/7SF mixture which underwent air drying as the initial curing regime, the cumulative mass loss after 50 cycles was 1.03 kg/m^2 ,

while for the companion specimen cut from the same slab the mass loss was only 0.20 kg/m².

4.4.5 Conclusions

Based on the experimental results presented, the following conclusions can be drawn regarding the scaling resistance of ternary (OPC/FA/SF) mixtures exposed to freezing and thawing cycles at two different ages after subjecting them to various curing treatments:

- The ternary mixtures containing 20% fly ash and 5% or 7% silica fume (by mass of binder) provided very good resistance to salt scaling at both early and late testing ages. Mixture with 30% fly ash and 5% silica fume (by mass of binder) exhibited poor scaling resistance when tested at the early age but considerably reduced scaling was observed when the same mixtures were tested at the late age. In the latter case, the extent of scaling was very similar to that observed for the mixtures with 20% fly ash. Mixture with 30% fly ash and 7% silica fume by mass of binder experienced substantial scaling irrespective of exposure age.
- Both mixtures with 20% fly ash (but in particular 20FA/7SF mixture) were much less sensitive to initial curing condition than the mixtures containing 30% fly ash. Therefore, based on the results of the current study, ternary mixtures with 20% rather than 30% of class C fly ash are recommended to minimize potential scaling problems.
- Air drying (lack of curing) at the early age may result in reduced late exposure scaling resistance even if some remedial curing actions (i.e. intermittent moisture supply) are taken.
- Only slight improvement in scaling resistance was obtained by extending wet burlap curing period from 3 to 7 days when scaling test was performed at the

early age. When the scaling test was conducted at the late age, this small difference diminished entirely.

- Application of curing compound resulted in the best scaling resistance for all mixtures tested at the early age. However, since comparable scaling results were obtained for specimens cured with either curing compound or burlap at the late age, the strong improvement in scaling observed for early age specimens is most likely a result of significantly lower degree of saturation of the surface layer pores and thus possible reduction in the salt concentration gradient.
- The specimen's age at the time of exposure to freeze-thaw cycles influenced the mode of scaling failure. For specimens exposed at early the age more uniform surface scaling occurred and it was mostly a result of the failure of paste layer. For the specimens exposed to freeze-thaw cycles at the late age, the scaling damage occurred mostly in the form of pop-outs located directly above the coarse aggregate particles.

4.5 Effect of Mixture Composition and Initial Curing Condition on Strength

Development of Ternary OPC/FA/SF Mixtures and Predictive Capability of Maturity

Method

4.5.1 Introduction

Maturity method is an experimental technique that enables relating the compressive strength of concrete to a product of time and temperature experienced by concrete during hydration of its cementitious components. Extensive literature review and description of applicability of the maturity method is available elsewhere (Carino and Lew 2001, Myers 2000). In the last decade, the maturity method has gained a relatively wide interest amongst practitioners and various Departments of Transportations (DOTs) as a tool for determining the time for opening traffic, formwork removal, etc. In the field applications the maturity method reduces the number of specimens needed for strength testing during

the quality control/quality assurance (QC/QA) process and results in labor and cost savings (Ahmad et al. 2006).

The maturity method was originally developed for ordinary portland cement (OPC) concretes which did not contain any supplementary cementitious materials (SCMs) such as fly ash (FA), silica fume (SF) or slag. However, simultaneously with growing interest in utilization of the maturity method to monitor the in-place strength of concrete, the number of projects incorporating high-performance concrete (HPC) with SCMs has also significantly risen. Therefore, there is a need to determine whether the maturity method is suitable for the concrete with mineral admixtures. Relatively limited amount of information regarding the applicability of the maturity method to compressive strength estimation of HPC has been reported in the literature. In the research conducted by Olek et al. (2002), the authors monitored the maturity of ternary concrete mixtures with combination of fly ash, silica fume and slag only up to 7 days. Although frequently one of the main objectives when using the maturity method is to estimate the strength of concrete at early age, the knowledge of later-age strength is very important as well. This is particularly true for concretes incorporating SCMs, as these are known to require longer hydration period compared to plain OPC concrete mixtures.

It was evident from the literature review that in majority of research on ternary (OPC/FA/SF) concrete compressive strength was evaluated for standard (i.e., continuous moist curing) curing conditions only (see Appendix). However, relatively little is known about the effect of curing method on strength development, and thus on strength predictive capability of maturity method. Therefore, the first objective of this study was to examine the influence of initial curing conditions and mixture composition on compressive strength of laboratory-made ternary (OPC/FA/SF) concrete. The second objective was to assess applicability of maturity method for prediction of compressive strength of ternary concrete subjected to various curing conditions.

4.5.2 Experimental Details

All specimens made in the laboratory were 102×203-mm cylinders. These specimens were used for monitoring of either temperature development or strength testing and were subjected to five different curing conditions. In addition to standard (control) water curing regime applied to specimens used for development of maturity function, four other curing regimes were incorporated. Those additional curing regimes were selected to cover a range of possible early-age curing conditions that may occur in the field and included: (a) air drying (no moisture or curing compound applied), (b) 7 days exposure to curing compound, (c) 3 days of curing under wet burlap and (d) 7 days of curing under wet burlap. The details of these curing procedures are provided in Table 4.10. Throughout the entire 90-days testing period all specimens were kept in a room with average temperature of $23.0 \pm 1.0^\circ\text{C}$ and relative humidity of $60 \pm 5\%$.

The concrete temperature was monitored using copper-constantan (type T) thermocouples embedded in the center of the 102×203-mm concrete cylinders at mid-height point. The temperature data were collected at the 0.5-hour intervals using the automated data acquisition system. With few exceptions, two specimens per mixture design and per curing condition were instrumented with thermocouples.

Table 4.10 – Curing procedures for 102×203-mm cylinders used in the study

Time (days) Curing regime	1	2	3	4	5	6	7	--	28	--	90
Water curing (Control)	Top under wet burlap and plastic sheet (while in mold)	Stored in tank in lime-saturated water									
Air drying (a)	Top allowed to dry (while in mold)	Entire specimens (except bottom) opened to air									
7 days curing compound (b)	Top covered with curing compound (while in mold)	Entire specimens (except bottom) covered with curing compound						Curing compound removed using wire brush and entire specimens (except bottom) opened to air			
3 days wet burlap (c)	Top under wet burlap and plastic sheet (while in mold)	Entire specimens (except bottom) covered with wet burlap and plastic sheet		Entire specimens (except bottom) opened to air							
7 days wet burlap (d)	Top under wet burlap and plastic sheet (while in mold)	Entire specimens (except bottom) covered with wet burlap and plastic sheet						Entire specimens (except bottom) opened to air			
TESTING	✓		✓				✓		✓		✓

Since all four concrete mixtures contained substantial amount of supplementary cementitious materials (see Table 1), rather than assuming the value of activation energy as the one suggested in ASTM C 1074 (ASTM 2006d) for plain cement concrete (42.0 kJ/mol), the actual value of activation energy was determined for each of the mixtures studied. This was accomplished following procedures proposed in ASTM C 1074 (ASTM 2006d) using mortar cubes cured at three different temperatures (11.5, 21.0 and 38.0°C). As per recommendations of the same specification, the composition of mortars was such that a fine aggregate-to-cement ratio (by mass) was the same as the coarse aggregate-to-cement ratio of the corresponding concrete mixtures. Furthermore, the relative (with respect to total mass of cementitious materials) content of chemical admixtures used in the mortar mixtures was the same as in the corresponding concrete mixtures. The compressive strength test was performed on two 102×203-mm cylinders at 1, 3, 7, 28 and 90 days (as indicated in the bottom row of Table 2). The test was performed in accordance with ASTM C 39 (ASTM 2006a).

4.5.3 Results and Discussion

4.5.3.1 Activation Energy

The values of activation energy obtained for the mortar mixtures were as follows: 39.7 kJ/mol for 20FA/5SF, 43.4 kJ/mol for 20FA/7SF, 20.3 kJ/mol for 30FA/5SF and 11.6 kJ/mol 30FA/7SF. Surprisingly, the values obtained for the first two mixtures are very close to that suggested for plain cement mixture (42.0 kJ/mol) by ASTM C 1074, in spite of relatively high content of supplementary cementitious materials present in the mixtures. The effect of change in silica fume content from 5% to 7% on the activation energy is unclear since for mixture with 20% fly ash, the activation energy slightly increased, whereas for mixture containing 30% fly ash it decreased by about 50%. However, a substantial decrease in activation energy observed with increasing fly ash content from 20% to 30% is generally in agreement with findings of Ma et al. (1994). Using isothermal calorimetry, these researchers obtained the activation energy of 26.7 kJ/mol

for mixture with 18% fly ash, which was significantly lower than that obtained for plain cement mixture (39.0 kJ/mol).

4.5.3.2 Temperature History

The temperature history from the time of casting up to about 8 days for all four concrete mixtures subjected to previously described curing regimes (a) - (d) is shown in Fig. 4.16. Although the differences observed in Fig. 4.16 can be considered negligible from the standpoint of maturity-based prediction of compressive strength, several distinctive features should be noticed. To begin with, for any given curing condition the maximum temperature of individual mixtures was almost the same, irrespective of mixture composition. This maximum temperature peak occurred at approximately 12 hours after casting. This time roughly coincides with final setting time determined for the same mixtures as a part of another study (see section 4.6.2.3).

Furthermore, it can be seen that during the first 24 hours period the maximum temperature of specimens covered with wet burlap and plastic sheet was about 2°C higher than the maximum temperature of specimens subjected to two other curing conditions (air drying and 7 days curing compound). Considering the fact that the experiment was conducted at room temperature, this difference can be considered significant. If the ambient temperature was substantially lower (as it often happens toward the end of construction season in the northern part of U.S.), covering the concrete surface with burlap and plastic sheets immediately after finishing should be highly beneficial. Not only will it prevent evaporation of water from concrete but it will also provide thermal insulation.

Also noteworthy is the fact that the temperature trends for respective curing regimes are very similar for all four mixtures studied (Fig. 4.16). The 7 days curing compound method caused the temperature to stabilize at room temperature after about 24 hours, which corresponds to the time of demolding and application of curing compound to the specimens. However, a distinct drop in temperature (by about 2°C) can be observed at the time of demolding for specimens subjected to air drying.

Fig. 4.16 – Early age concrete temperature data for mixtures: (a) 20FA/5SF; (b) 20FA/7SF; (c) 30FA/5SF; (d) 30FA/7SF

The same phenomenon can be noticed at 76 and 168 hours for 3 days burlap and 7 days burlap cured specimens. In these cases, the temperature drop corresponds to the end of wet burlap curing period. The observed temperature drop occurred, most likely, as a result of adiabatic cooling upon evaporation of moisture from concrete surface exposed to relatively dry air. In other words, since evaporation process requires some additional energy to mobilize water molecules, the surrounding

environment is deprived of this energy, which in turn leads to cooling effect. Interestingly, the temperature depression has not occurred at 168 hours for either of two mixtures containing 30% fly ash (Fig. 4.16 (c) and (d)). Although not shown, the data collected past the initial 7 days (168 hours) curing period indicate that the temperature of all specimens stabilized at the level of room temperature and remained at this level for the duration of the test.

4.5.3.3 Compressive Strength

Since the compressive strength test results exhibited very good repeatability (the average coefficient of variation was about 2.4%), only the average values obtained from two specimens are shown in Fig. 4.17. In general, the compressive strength was compromised by decreasing relative quality (in terms of moisture loss prevention) of curing. As expected, the control (water cured) specimens achieved the highest strength, whereas the strength of specimens cured using other methods was decreasing in the following order: 7 days burlap, 3 days burlap, 7 days curing compound and air drying.

However, it is evident that the strength development of mixtures with 30% fly ash was impaired by “poor” curing practices to much greater extent than the strength development of mixtures with 20% fly ash. For instance, the specimens from mixtures with 30% fly ash subjected to air drying achieved only about 50% of the ultimate strength of “control” moist cured specimens. This is appreciably lower than about 65% achieved by specimens from mixtures with 20% fly ash. Numerous researchers have previously observed increased sensitivity to inadequate curing of concrete with high fly ash contents (Bentz 2002, Ma et al. 1994, Neuwald 2004, Shafiq and Cabrera 2004).

For comparative purposes, compressive strength of INDOT class C mixture was measured after 28 days of moist curing. This strength obtained by class C concrete was 41.3 MPa which is significantly lower than that obtained by either of the mixture with 20% fly ash, but is very comparable with the 28-day strength of mixtures containing 30% fly ash (Fig. 4.17).

Fig. 4.17 – Compressive strength development for mixtures: (a) 20FA/5SF; (b) 20FA/7SF; (c) 30FA/5SF; (d) 30FA/7SF

4.5.3.4 Strength Prediction Using Maturity Method

The prediction of compressive strength of the mixtures studied was based on the equivalent age at 20.0°C, which was calculated for each mixture and curing condition using Eq. 4.4 and the procedures of ASTM C 1074 (ASTM 2006d):

$$t_e = \sum e^{-\frac{E_a}{R} \left(\frac{1}{T_a} - \frac{1}{T_s} \right)} \Delta t \quad \text{Eq. 4.4}$$

where t_e is equivalent age at specified temperature T_s (days), E_a is activation energy (J/mol), R is universal gas constant (8.314 J/(mol·K)), T_a is average temperature of concrete during time interval Δt (K), T_s is specified temperature (K) (here 293.2 K) and Δt is time interval between the temperature readings (h). The summary of the t_e values for all mixtures and curing conditions used in the study is shown in Table 4.11.

Subsequently, a mathematical function relating compressive strength of “control”, water cured specimens to the corresponding equivalent age at 20°C was established for each mixture individually. In general, the following function was found to fit the data very well:

$$S = a \cdot \ln(t_e) + b \quad \text{Eq. 4.5}$$

where S is the compressive strength (MPa). The values of coefficients a and b , as well as the values of coefficient of determination (R^2), for all of four mixtures are given in Table 4.12. It can be seen that the value of coefficient a decreases with increasing contents of both fly ash and silica fume, which reflects the fact the compressive strength development is adversely affected by increasing contents of these mineral admixtures in the ternary systems studied (see Fig. 4.17). This is in line with findings from preliminary optimization analysis of ternary concrete mixtures containing fly ash and silica fume discussed in section 3.3.

Table 4.11 – Equivalent age t_e (days) at 20.0°C

Test Age (days)	Equivalent Age (t_e) for Four Series of Mixtures Used			
	20FA/5SF	20FA/7SF	30FA/5SF	30FA/7SF
1.0	1.3/1.3/1.4/1.4	1.3/1.2/1.4/1.4	1.1/1.0/1.2/1.2	0.9/0.9/0.9/0.9
3.0	3.5/3.6/3.8/3.8	3.5/3.6/3.7/3.8	3.2/3.3/3.4/3.4	2.9/2.8/2.8/2.8
7.0	8.1/8.2/8.3/8.5	8.2/8.3/8.3/8.6	7.5/7.6/7.6/7.7	6.7/6.6/6.7/6.6
28.0	32.7/32.7/32.8/32.8	32.9/33.0/33.0/33.0	30.4/30.3/30.3/30.3	26.7/26.8/26.7/26.6
90.0	108.0/108.0/108.1/108.0	109.4/109.4/109.5/109.4	98.4/98.4/98.5/98.4	85.4/85.5/85.4/85.5

Note: Designation 1.3/1.3/1.4/1.4 represents equivalent age t_e for, respectively, air drying/7 days curing compound/3 days burlap/7 days burlap

Based on the equivalent ages at 20°C determined using Eq. 4.4 the predicted compressive strength of each mixture was calculated from Eq. 4.5 for the respective curing condition (a) through (d). The comparison of the predicted compressive strength and the corresponding measured values is shown in Fig. 4.18. Generally, the curing method did not affect the temperature history of concrete significantly (Fig. 4.16). As a result, the equivalent ages for a particular mixture remained very comparable, regardless of curing condition applied (Table 4.11). It therefore follows that the observed differences in compressive strength associated with the different curing conditions (shown in Fig. 4.17) must be mostly attributed to differences in the amount of moisture supplied to the specimens. This conclusion is confirmed by strong dependency of the age at which the predicted strength starts to deviate from the equality line in Fig. 4.18 on the type of initial curing used. Specifically, the curves for air dried specimens start to deviate first, followed by the curves for 7 days curing compound, 3 days burlap and finally for 7 days burlap.

Table 4.12 – Values of constants a and b

Mixture	a	b	R^2
20FA/5SF	9.77	13.61	0.99
20FA/7SF	8.77	12.00	0.99
30FA/5SF	8.42	10.97	0.99
30FA/7SF	7.44	11.89	0.99

Regarding the effect of mixture composition on the accuracy of strength prediction by the maturity method, generally the increase in fly ash content from 20% to 30% caused the curves for respective curing conditions to deviate at earlier age (Fig. 4.18). The exception to this rule was 7 days burlap curing regime. It can be seen that for all four mixtures studied a comparable deviation for 7 days burlap curing occurred between the 28th and 90th day. This means that when the quality of initial curing is satisfactory, the concretes containing high amount of fly ash are fully capable of achieving the strength level of continuously moist cured (control) concrete. This, in turn,

implies that the maturity method can be successfully employed for strength prediction as long as the curing practices are satisfactory. However, if the quality of curing is substandard, the reliability of the maturity method drops drastically with the increase in FA content. For instance, the curves representing the results for air dried specimens with 30% FA content (Fig. 4.18 (c) and (d)) start to substantially deviate from the equality line already at 7 days, while, at the same age, they remain relatively close to the equality line for the mixtures with 20% FA (Fig. 4.18 (a) and (b)).

Fig. 4.18 – Predicted vs. measured compressive strength for mixtures: (a) 20FA/5SF; (b) 20FA/7SF; (c) 30FA/5SF; (d) 30FA/7SF (numbers in parentheses denote testing age)

The increase in amount of silica fume from 5% to 7% was beneficial, in terms of improvement of predictive capability of the maturity method, as the curves for all curing conditions (except for discussed above 7 days burlap) for 5% SF concretes depart from the equality line more than the curves for 7% SF (at constant fly ash content). In other words, the vertical distance from the individual data points to the equality line (representing the prediction error) is smaller for the mixtures containing 7% SF than for the mixtures with 5% SF.

4.5.4 Conclusions

Based on the results of the study on the effect of mixture composition and curing conditions on compressive strength development and strength predictive capability of maturity method, the following conclusion could be drawn:

- The strength of ternary concretes subjected to different curing conditions decreased in the following order: 7 days burlap, 3 days burlap, 7 days curing compound and air drying.
- In general, compressive strength appeared to decrease with an increase in FA and SF content (and thus a paste content) of the mixture. Therefore, for a given curing regime, the strength was generally decreasing in the following order: 20FA/5SF3, 20FA/7SF, 30FA/5SF and 30FA/7SF.
- The accuracy of strength prediction generally decreased with decreasing relative quality of curing regime (in terms of moisture supply). The negative effect of compromised initial curing was much more pronounced for the ternary mixtures containing 30% fly ash than for the mixtures containing 20% fly ash. An increase in silica fume content from 5% to 7% resulted in only slight improvement in reliability of the method.

- Maturity method should not be used for compressive strength prediction of in-situ ternary OPC/FA/SF concrete, unless concrete is cured under wet burlap for a minimum of 3 days and the age of concrete does not exceed 28 days.

4.6 Effect of Mixture Composition and Initial Curing Conditions on Shrinkage and Shrinkage Cracking Resistance of Ternary OPC/FA/SF Mixtures

4.6.1 Introduction

Next to corrosion of reinforcement, salt scaling and freeze-thaw (F-T) damage, shrinkage cracking is considered to be the most significant type of distress frequently encountered in concrete bridge decks (Russell 2004). In an attempt to address these durability problems, high performance concrete (HPC) has been developed several decades ago. While HPC has, in general, proven to be effective in minimizing the risk of corrosion of reinforcement, scaling and F-T damage, the common downside of this material is its relatively high propensity to shrinkage cracking. Increased susceptibility of HPC to shrinkage cracking is generally attributed to relatively low water-cementitious materials ratio and incorporation of very fine pozzolanic materials, such as silica fume. As a result, superior performance of bulk concrete (in terms of high resistance to chloride-ion penetration) is frequently hindered by presence of shrinkage cracks. On one hand these cracks create a direct path for chloride-ions to access the reinforcing steel and on the other they may lead to spalling problem due to freeze-thaw action.

It is well established that the propensity of concrete to shrinkage cracking is not a function of a single parameter, but depends on the combination of several factors, such as rate and amount of shrinkage, creep, elastic modulus and tensile strength (Radlinska 2008, Russell 2004, Xi et al. 2003). However, up to now little research has been conducted to determine the shrinkage behavior and cracking susceptibility of concrete incorporating both fly ash and silica fume.

Similarly, relatively little is known about the impact of curing quality on drying shrinkage and, most importantly, on susceptibility to shrinkage cracking. This research was undertaken to specifically investigate the influence of both mixture

composition and curing quality on the magnitude of shrinkage and shrinkage cracking resistance of ternary (OPC/FA/SF) concretes.

4.6.2 Experimental Details

4.6.2.1 Mixture Proportions

In addition to four ternary OPC/FA/SF mixtures studied (20FA/5SF, 20FA/7SF, 30FA/5SF and 30FA/7SF), the plain cement (INDOT class C (INDOT 2008)) mixture was included for reference purposes. The detailed proportions of all mixtures used in the study are shown in Table 4.2.

4.6.2.2 Curing Conditions

In general, the influence of the following four curing regimes was evaluated: air drying (lack of curing), 3 days under wet burlap, 7 days under wet burlap and 7 days covered with curing compound. However, due to time- and equipment-related limitations not all of the possible mixture-curing condition combinations have been evaluated. The detailed experimental matrix used in this study is shown in Table 4.13. In total, 16 different types of mixtures were prepared and tested.

Table 4.13 – Experimental matrix for study on shrinkage and cracking resistance

	Air Drying	7 Days Curing Compound	3 Days Burlap	7 Days Burlap
20FA/5SF	✓	✓	✓	
20FA/7SF	✓	✓	✓	✓
30FA/5SF	✓	✓	✓	
30FA/7SF	✓	✓	✓	✓
INDOT Class C	✓			✓

4.6.2.3 Specimens Preparation and Test Procedures

For each of 16 mixtures made according to Table 4.13, the following specimens were cast:

- 1 specimen for determination of setting time;

- 2 restrained shrinkage specimens (subjected to curing one of the curing methods shown in Table 4.13)
- 3 “standard” free shrinkage specimens (demolded at 23.5 hours and cured for 28 days);
- 3 “nonstandard” free shrinkage specimens (demolded at final set and cured using one of curing methods according to Table 4.13)

The restrained shrinkage test was conducted in accordance with AASHTO PP 34 (AASHTO 2006a) to accommodate larger ($d_{\max} = 25$ mm) aggregate size than that allowed by ASTM C 1581 ($d_{\max} = 12.5$ mm) (ASTM 2004b). The geometry of the restrained shrinkage specimens is shown in Fig. 4.19. The strains developing in each of the four strain gages attached to the ring were automatically recorded every 30 minutes. The length of the strain monitoring period varied depending on the outcome of the test. If cracking occurred, the test was terminated within few days after cracking; otherwise the strains were monitored until no change was observed over time (up to 196 days).

The specimens were prepared by casting concrete in a space between internal steel ring and outside cardboard mold in three equal layers, each rodded using a 16-mm diameter steel rod. After finishing with a steel trowel, the specimens were placed in the room with controlled temperature (23.0°C) and relative humidity (50%). For the first 24 hours the top surface of the specimens was subjected to one of the curing conditions listed in Table 4.13. At 24 hours, the specimens were demolded by removing the outside cardboard ring. This was followed by sealing of the top of ring specimens using self-adhesive aluminum tape. Subsequently, the exposed side of each specimen was subjected to the respective curing condition for a given period of time (see Table 4.13). Once the time required for a given curing condition expired, circumferential drying was allowed (Fig. 4.19). If curing compound had been applied to the specimens, it was removed from the surface at 7 days using a wire brush. The specimens cured under wet burlap were also covered with plastic sheets to prevent evaporation of water.

Fig. 4.19 – Geometry of restrained shrinkage specimen

Two sets of three prismatic (76×76×286 mm) specimens were cast for monitoring of free shrinkage. Both sets were cast the same way (following AASHTO T 160 (AASHTO 2006c) standard) but they were handled differently thereafter. The first set of shrinkage specimens was conditioned following AASHTO T 160 procedure. This involved transferring the freshly cast specimens to a room with temperature of 23°C and relative humidity of 50%, where they were covered with moist burlap and plastic sheet. The specimens were demolded after 23.5 hours and submerged in lime-saturated water for about 45 minutes. After wiping of the surface of specimens with a damp cloth, the initial length measurements were taken using a length comparator. Subsequently, the specimens were moist cured in lime-saturated water for a period of 28 days. The specimens were then moved to be stored in a room with temperature of 23°C and relative humidity of 50%. The shrinkage measurements were taken after 1, 4, 7, 14, 28, 56, 112, 224, and 448 days and the average shrinkage of three specimens will be reported.

The second set of free shrinkage specimens was cast to monitor free shrinkage of concrete subjected to the same curing regime as the restrained shrinkage specimens. Also, the AASHTO T 160 procedure is often criticized for not accounting for autogenous shrinkage. Therefore, the standard procedure was modified to capture the autogenous deformation, most of which occurs at very early age.

After casting, the specimens from the second shrinkage group were placed in a room with temperature of 23°C and relative humidity of 50% where their top surface was immediately subjected to one of the curing conditions shown in Table 4.13. To account for autogenous shrinkage, the specimens were demolded at the time of final set of concrete. The setting time was determined according to AASHTO T 197 (AASHTO 2006e) on the mortar wet-sieved from the concrete mixture. The final setting times are provided in Table 4.14. Immediately after demolding the initial length measurements were taken using a length comparator (AASHTO 2006c). Afterwards, the specimens were subjected to the respective curing regime. In the case of “7 days curing compound” method the entire surface of the specimens was covered with curing compound which was removed with a wire brush at 7 days. The specimens to be cured under wet burlap were instead wrapped with wet burlap plastic sheet. Following the completion of their respective curing regime, the specimens were allowed to dry in a room with the temperature of 23°C and relative humidity of 50%. The shrinkage measurements were taken 1, 4, 7, 14, 28, 56, 112, 224, and 448 days after completion of the respective curing regime and the average shrinkage of three specimens will be reported.

Table 4.14 – Final setting times (h)

	Air Drying	7 Days Curing Compound	3 Days Burlap	7 Days Burlap
20FA/5SF	9.0	8.5	8.5	–
20FA/7SF	11.0	9.5	10.5	10.5
30FA/5SF	12.0	9.5	12.0	–
30FA/7SF	13.0	11.0	11.0	11.5
INDOT Class C	10.0	–	–	10.0

Note: All setting time specimens were treated the same way (i.e., covered with plastic lid).

4.6.3 Results and Discussion

4.6.3.1 Fresh Concrete Properties

The fresh concrete properties are shown in Table 4.15. It can be seen that both the air content and slump obtained for all mixtures were within a relatively small range of 6.1-7.3% and 100-180 mm, respectively.

Table 4.15 – Fresh concrete properties (air content (%)/slump (mm)) for study on shrinkage and shrinkage cracking

	Air Dry	7 Days Curing Compound	3 Days Burlap	7 Days Burlap
20FA/5SF	7.3/170	6.9/100	6.8/120	–
20FA/7SF	6.7/180	6.6/100	7.1/120	6.0/120
30FA/5SF	7.0/110	6.7/140	6.7/165	–
30FA/7SF	6.2/150	6.1/100	7.2/165	7.0/160
INDOT Class C	6.7/160	–	–	6.7/170

4.6.3.2 Free Shrinkage

The free shrinkage results obtained for the group of “standard” (AASHTO T 160) specimens are presented in Fig. 4.20. It should be noted that within the graphs (a) through (e), the individual curves were obtained for nominally the same composition and standard curing mode but from several batches made to produce restrained specimens and the nonstandard free shrinkage specimens (Table 4.13). Therefore, the results represent the variability of free shrinkage test when the specimens are demolded at 23.5 hours and cured according to AASHTO T 160. It can be seen that, in several cases, the differences between the replicate mixtures were as high as 120 microstrains.

Interestingly, when comparing only the ternary mixtures, it seems that the mixtures with 30% fly ash (Fig. 4.20(c) and (d)) appear to exhibit more uniform results than the 20% fly ash mixtures (Fig. 4.20(a) and (b)). Nevertheless, the amount of free shrinkage of all four ternary mixture studied appears fairly comparable and clearly lower than the shrinkage of class C mixture. These results suggest that the use of ternary OPC/FA/SF mixtures may be advantageous in terms of minimizing shrinkage and hence shrinkage cracking.

Fig. 4.20 – Free shrinkage of standard cured specimens for mixtures:
 (a) 20FA/5SF; (b) 20FA/7SF; (c) 30FA/5SF; (d) 30FA/7SF; (e) INDOT Class C

Since paste content of the mixtures used in the study was not constant (see Table 4.2), an attempt was made to separate the effect of binder composition on shrinkage from that of the paste content by normalizing the results by aggregate content (with respect to OPC mixture). According to Pickett (1956), the effect of aggregate content on shrinkage can be expressed as follows:

$$S = S_0 (1 - g)^\alpha \quad \text{Eq. 4.6}$$

where S is shrinkage of concrete, S_0 is shrinkage of paste, g is volume of aggregate per volume of mixture and α is a coefficient that can vary from 1.2 to 1.7, depending on aggregate stiffness (L'Hermite 1960). The α of 1.4, which was recently suggested by Moon (2006), was assumed in this study. Using Eq. 4.6, to account for different aggregate content in each ternary mixture compared to class C mixture, the normalized (with respect to aggregate content) shrinkage S_{norm} can be calculated as:

$$S_{norm} = S \left(\frac{1 - g_{OPC}}{1 - g_i} \right)^\alpha \quad \text{Eq. 4.7}$$

where g_{OPC} and g_i are aggregate volume fractions for class C mixture and ternary mixtures, respectively. The resulting values of multiplier $[(1 - g_{OPC})/(1 - g_i)]^\alpha$ used to normalize the shrinkage of 20FA/5SF, 20FA/7SF, 30FA/5SF and 30FA/SF mixtures were, respectively, 1.283, 1.240, 1.084 and 1.041.

The average normalized shrinkage for standard-cured specimens is shown in Fig. 4.21. It is evident that even after adjusting for aggregate content, class C concrete still displayed significantly higher shrinkage than any of the ternary mixtures studied. Therefore, lower aggregate content (higher paste content) of plain cement concrete explains higher shrinkage of this mixture only to a limited extent. It is likely that the apparent reduction in the total shrinkage of ternary mixtures (when compared to plain mixture) was the results of higher autogenous shrinkage of the former. Unfortunately, autogenous shrinkage was not measured during this part of the study as it is not specified by AASHTO T 160 test protocol.

Fig. 4.21 – Average shrinkage of standard-cured specimens normalized per paste content of INDOT Class C mix

Fig. 4.22 – Relationship between Cement Replacement Level and Normalized Shrinkage

Interestingly, after normalizing, 30% fly ash mixtures showed the lowest shrinkage. These results imply that shrinkage (particularly at later age) of concrete subjected to standard curing procedure (28 day storage in lime-saturated water) decreases with an increase in cement replacement level, as illustrated in Fig. 4.22. The reduction of shrinkage with cement replacement level may be a result of reduction in carbonation shrinkage, for which the pessimum conditions occur at RH of 50% (Neville 1995). The replacement of cement by FA and SF not only results in less calcium hydroxide produced to begin with, but it also reduces calcium hydroxide content over time by means of pozzolanic reaction. This further emphasizes the benefit of utilizing ternary cementitious systems.

Fig. 4.23 shows free shrinkage results for the group of specimens subjected to four different, previously described non-standard curing conditions. It can be seen that if no curing was applied (concrete was allowed to dry), the resulting ultimate shrinkage appears to be about the same (~550 microstrains) for all four ternary mixtures, despite each having different binder composition and different amount of paste (Fig.

4.23(a)-(d)). Further, the amount of free shrinkage of all four ternary concretes subjected to air drying is clearly lower than that of class C mixture. However, after the results were adjusted for aggregate content with respect to class C mixture (using the same procedure as previously described for standard curing conditions), the shrinkage appeared to be a function of cement content and (as in the case of standard-cured specimens) it dropped with an increase in cement replacement level (results not shown).

It is also important to notice the effect of mixture composition on early age shrinkage of air dried concrete. The 1-day shrinkage was gradually reduced from -150 microstrains for 20FA/5SF mixture to -20 microstrains for OPC mixture (Fig. 4.23(a) through (e)). These results likely reflect the differences in the pore structure in these mixtures resulting from various degree of hydration, as well as differences in autogenous shrinkage (not measured).

Compared with air drying, 7 day curing under wet burlap resulted in reduced shrinkage (in terms of total amount). This reduction is mainly due to initial expansion, which occurred during wet burlap curing period (Fig. 4.23(b), (d) and (e)). Although 3 days wet burlap curing resulted in comparable expansion to that for 7 days burlap curing, the ultimate shrinkage was reduced by about 80 microstrains (compared to air dried specimens) in the case of 20% FA mixtures, whereas it was increased by about 65 microstrains for 30% FA mixtures.

Further, the amount of free shrinkage for 20% fly ash mixtures (Fig. 4.23(a)-(b)) was comparable for air drying and 7 days curing compound curing regimes. This was in spite of the fact that 1-day shrinkage of specimens treated with curing compound was actually lower than that of the air dried specimens. In contrast, 1-day shrinkage of 30% FA concrete protected by curing compound was comparable to that of air dried specimens, whereas the ultimate shrinkage for 7 days curing compound was about 150 microstrains higher than for air drying (Fig. 4.23(c)-(d)).

Fig. 4.23 – Free shrinkage of specimens subjected to nonstandard curing conditions for mixture: (a) 20FA/5SF; (b) 20FA/7SF; (c) 30FA/5SF; (d) 30FA/7SF; (e) INDOT Class C

It should also be noted that the free shrinkage for ternary concrete subjected to more realistic (from the standpoint of field exposure) curing conditions (Fig. 4.23) was about 250 microstrains higher than for the AASHTO T 160 procedure (Fig. 4.20). Therefore, in order to adequately determine potential free shrinkage, the procedure consisting of demolding specimens at final set (rather than at an arbitrary time of 23.5 hours) and subjecting them to curing conditions which are anticipated in the field should be used.

4.6.3.3 Restrained Shrinkage

The results of restrained shrinkage measurements for specimens subjected to different curing conditions are presented in Fig. 4.24. For a given curing condition, the cracking potential of ternary mixtures generally increased with an increase in the content of supplementary cementitious materials (and thus an increase in paste content). This can be attributed to reduction in strength with an increase in the paste content (Fig. 4.17).). It appears that plain cement (i.e., INDOT class C) concrete was more prone to cracking than any of the ternary mixtures evaluated. Particularly, the mixtures with 20% fly ash exhibited in general good resistance to shrinkage cracking (Fig. 4.24(a)-(b)) likely due to their relatively low paste content.

Regarding the effect of curing regime on cracking resistance, curing using wet burlap was in most cases found to have positive effect on cracking resistance of ternary mixtures, but not in all Fig. 4.24, Specifically, while both specimens from 20FA/7SF mixture subjected to air drying remained uncracked, one of the specimens cured for 7 days under burlap cracked. However, curing under wet burlap was observed to delay time to cracking of class C concrete when compared to air drying (Fig. 4.24(e)).

Fig. 4.24 – Restrained shrinkage strains of subjected to various curing conditions for mixture: (a) 20FA/5SF; (b) 20FA/7SF; (c) 30FA/5SF; (d) 30FA/7SF; (e) INDOT Class C
(Note: (1) and (2) in legend denote, respectively, ring 1 and 2)

Even though the results of the other studies conducted within this research project suggested that substituting wet burlap curing with an of application of curing compound results, in general, in sufficiently good performance of ternary concrete in terms of compressive strength development, scaling resistance and transport-related properties, the results from the current study imply that curing compound may have detrimental effects on resistance to shrinkage cracking of evaluated ternary mixtures. Specifically, the only two specimens from 20FA/5SF mixture series that cracked within the monitoring period were the ones treated with curing compound (Fig. 6(a)). Similarly, for 20FA/7SF mixture series, the only two specimens that cracked (in addition to previously mentioned specimen cured under wet burlap) were those treated with curing compound (Fig. 6(b)). Interestingly, for both of 30% fly ash mixtures the effect of application of curing compound was not clear, i.e., neither the improvement nor the reduction in resistance to cracking was observed when compared with air drying (Fig. 4.24(c) and (d)).

In summary, it seems that the curing compound treatment increased the cracking tendency of ternary mixtures and therefore its use on bridge decks containing such mixtures is not recommended. These findings are in line with recommendations put forward by Aïtcin (1998), who stated that curing compound membranes are only acceptable for concretes with high water-binder ratio (0.50 to 0.70), as the total deformation exhibited by such concretes is primarily associated with drying, rather than autogenous, shrinkage.

The results for “age at cracking” are summarized in Fig. 4.25. It can be seen that repeatability of the test is relatively poor. Fig. 4.25 also indicates that the repeatability of the restrained shrinkage test is relatively poor. Differences in “age at cracking” higher than 100% have been previously reported by other researchers who conducted extensive studies on silica fume concrete for bridge decks (Whiting et al. 2000) and on the use of internal curing to prevent concrete bridge deck cracking (Mack 2006). Cracking of only one of two rings tested, which in the present study took place in three cases, was also not uncommon in several studies involving the restrained shrinkage tests

(Brown et al. 2007, Mack 2006, Whiting et al. 2000, Nassif et al. 2008). Nonetheless, as conceptually illustrated in Fig. 4.25, it appears that the highest resistance to shrinkage cracking can be attained by reducing paste content (note that the mixtures are grouped in the order of increasing paste content) and by applying wet burlap curing for 3 or 7 days.

Fig. 4.25 – Age at cracking of restrained shrinkage specimens (Note: 33\94 = 1st specimen cracked after 33 days and 2nd specimen cracked after 94 days)

4.6.3.4 Probability of Cracking

An analysis was performed to determine whether the free shrinkage results (from the modified test) could be used to predict the probability of cracking of the ring specimens. Fig. 4.26 shows a relationship between the free shrinkage after 56 days and the probability of cracking throughout the entire monitoring period (not just during the first 56 days) for all mixtures studied. The age of 56 days was selected based on the fact that by this time about 75% of long term (~450 days) shrinkage occurred in all of the specimens tested (Fig. 4.23). In addition, most (15 of 19) of the restrained shrinkage specimens also cracked within 56 days (Fig. 4.24).

For all the data points shown, the probability of cracking takes a value of either 0 (none of the rings cracked), 0.5 (one ring cracked, the other did not), or 1 (both rings cracked). It can be seen that, although the data points appear to be rather scattered, the plot could be divided into several zones (using dashed vertical lines) according to the increasing average probability of cracking as a function of the range of free shrinkage values. As a result, it was possible to construct the approximate cumulative probability of cracking curve (shown as thick black line) by subsequent inclusion of the data points contained in the respective zones using the following procedure. The x-coordinates of the points on the cumulative curve (grey circles) represent 56-day shrinkage values corresponding to the beginning (i.e., lower boundary) of the respective zone. The y-coordinates of those points represent the average probability of cracking for all data points lying within each zone (excluding the data point coinciding with the upper zone boundary, which was arbitrarily assigned to the beginning of the next zone).

Fig. 4.26 – Relationship between 56-day free shrinkage and probability of cracking

It appears that in order to entirely prevent the evaluated bridge deck concrete from cracking, a 56-day shrinkage limit of 310 microstrains should be applied. Shrinkage of about 430 microstrains will result in the probability of cracking of about

0.64, while 56-day shrinkage higher than 620 microstrains would certainly result in cracking. These results are in a reasonable agreement with findings of Radlinska (2008) who, through a series of Monte Carlo simulations, examined the probability of cracking of concrete with the long-term elastic modulus of 30.0 GPa and splitting tensile strength of 3.7 MPa. It was suggested that to achieve a low probability of cracking (less than 10%), the long-term shrinkage of concrete should be limited to 290 microstrains. However, for shrinkage higher than 410 microstrains, the probability of cracking exceeded 90%.

The above discrepancy in the threshold shrinkage values for a given level of probability of cracking can be explained by several assumptions made when performing Monte Carlo simulations, of which the most important was a degree of restraint. For simulation purposes the degree of restraint was assumed to be 100%, whereas AASHTO PP 34 test provides only about 70% of restraint. Thus, lower degree of restraint would allow for higher shrinkage values to obtain the same probability of cracking. On the other hand, 56-day free shrinkage strain comprising 70-90% of the ultimate shrinkage (Fig. 4.23) was used in this work, while 365-day shrinkage data were used in the simulations. However, clearly the net outcome of the above two differences is such that, according to present study, the threshold shrinkage is consistently shifted to higher values for a given probability of cracking.

While the above analysis was performed for a range of concrete mixtures with different composition and, more importantly, subjected to very different initial curing conditions, clearly, more experimental data are needed to further validate the proposed approach. Further, the probability of cracking pertains to laboratory ring specimens, and not to field concrete structure which may be subjected to different environmental conditions and degree of restraint. Nonetheless, the developed relationship may prove useful in providing guidelines for selection of bridge deck concrete mixture proportions to minimize the risk of shrinkage cracking. It should be also recognized that the only input needed to implement the proposed approach is the 56-day free shrinkage value, which is relatively easy to determine compared to other

techniques used for assessment of susceptibility of concrete to shrinkage cracking (including rather expensive AASHTO PP 34 test setup).

When attempting to make recommendations regarding the ternary mixture composition and curing method based on the threshold shrinkage values obtained from Fig. 4.23, it appears that only 20FA/7SF concrete cured for 3 days under wet burlap ensures likelihood of cracking equal 0. Further, only 20FA/5SF concrete cured for 3 days under wet burlap as well as 7-day wet burlap cured 20FA/7SF concrete exhibited shrinkage below 400 microstrains, which according to Fig. 4.26, should give a probability of cracking of 0.50 or less. Therefore, as concluded from Fig. 4.25, to avoid shrinkage cracking issues, it is recommended that ternary concrete with fly ash content of 20% or less, silica fume of 7% or less and paste content of 24% or less are used. Also, it appears that in order to minimize cracking, concrete should be moist cured for at least 3 days.

4.6.4 Conclusions

The following conclusions have been drawn on the basis of this research which focused on the effect of mixture composition and initial curing conditions on free shrinkage and resistance to shrinkage cracking of high-performance concretes:

- Application of curing compound not only did not reduce free shrinkage of ternary concrete, but it also resulted in reduced resistance to shrinkage cracking compared with the resistance of ternary concrete subjected to air drying.
- Curing under wet burlap resulted in lower total free shrinkage for plain cement concrete and ternary mixtures with 20% FA (and 5% or 7% SF) than air drying. However, the same curing condition resulted in higher ultimate shrinkage for ternary mixtures with 30% FA and 5% or 7% SF. Nevertheless, of all curing methods evaluated, curing using wet burlap provided the highest resistance to shrinkage cracking.
- All ternary OPC/FA/SF mixtures studied appeared to exhibit higher resistance to shrinkage cracking than INDOT class C mixture.

- The susceptibility of ternary concrete to shrinkage cracking increased with increasing paste content. As a result, of the four ternary mixtures studied, concrete with 20% fly ash and 5% silica fume is recommended as it displayed high resistance to cracking when properly cured.
- The relationship established between 56-day free shrinkage and probability of cracking can be used as a guide in selection of bridge deck concrete mixture possessing potentially high shrinkage cracking resistance.

4.7 Examination of Resistance to Plastic Shrinkage Cracking

Resistance of concrete to plastic shrinkage cracking was assessed using the ASTM C 1579 test method (ASTM 2006f). The testing setup consisted of two concrete slabs placed in an environmental chamber and exposed to the following conditions: the temperature of $36\pm 3^{\circ}\text{C}$, relative humidity of $30\pm 10\%$ and wind velocity of about 4.7 m/s. Both slabs were cast from a single batch in special molds with stress risers at the bottom. The geometry and dimensions of the test specimens and stress risers are provided in Fig. 4.27, whereas the actual specimens immediately after casting and during the testing are shown in Fig. 4.28(a) and (b), respectively. In addition to plastic shrinkage specimens, a single 150 mm diameter specimen for determination of setting time (conducted as per AASHTO T 197 (AASHTO 2006e)) was placed in the chamber. The plastic shrinkage test was terminated at the final setting time of concrete.

Fig. 4.27 – Plastic shrinkage specimen and stress risers geometry (from ASTM (2006f))

Fig. 4.28 – Plastic shrinkage specimen after casting (a) and during testing (b)

The test was first performed on the 30FA/7SF mixture which was assumed to be most prone to plastic shrinkage cracking of all four ternary mixtures studied due to the highest paste and silica fume contents. However, when the test was concluded, none of two replicate specimens cracked. Subsequently, the test was performed on INDOT Class C concrete which had a composition as shown in Table 4.2. Again, cracking did not occur. Finally, 30FA/7SF mixture was tested once again, this time using the harshest conditions that could be generated in the environmental chamber used. These included raising the temperature to 44°C and lowering the relative humidity to 20%. Even under these extreme conditions, the plastic shrinkage cracking did not take place. Therefore, it was concluded that the ternary mixtures studied were not prone to plastic shrinkage cracking.

4.8 Comparison of Freezing-Thawing Resistance of OPC/FA/SF Mixtures

4.8.1 Introduction

It was previously reported (Toutanji et al. 2005) that concrete containing supplementary cementitious materials (SCMs), such as fly ash, silica fume and slag, may not be resistant to freeze-thaw (F-T) action. The unsatisfactory performance in AASHTO T 161 (AASHTO 2006d) F-T test of concrete containing SCMs was mainly attributed to early testing age of 14 days specified by that standard. Therefore, several researchers (Russell and Ozyildirim 2006, Toutanji et al. 2005) suggested that F-T test of concrete containing SCMs should not be commenced at the age of 14 days due to somewhat lower rate of pozzolanic reaction (of fly ash and slag in particular) compared to hydration of ordinary portland cement (OPC). It is often argued that by the age of 14 days, the OPC concrete achieves greater level of degree of hydration than concrete with mineral admixtures, which automatically makes the comparison “unfair”. Further, since in real-life applications severe frost action will typically not take place at such early age of concrete, it was proposed that the testing age for concrete incorporating SCMs should be 28 or 56 days (Russell and Ozyildirim 2006).

The main purpose of this study was to evaluate freeze-thaw resistance of ternary concrete containing fly ash and silica fume. The second objective was to verify whether extending curing period from 14 to 56 days indeed results in increased F-T resistance of ternary concrete.

4.8.2 Experimental Details

4.8.2.1 Mixture Proportions

Freeze-thaw resistance of six different mixtures was evaluated. In addition to four ternary mixtures (i.e., 20FA/5SF, 20FA/7SF, 30FA/5SF and 30FA/7SF) studied, two more mixtures were assessed. In addition to the four ternary mixtures studied, for comparative purposes, two more mixtures were made and subjected to F-T test. The first mixture was made according to INDOT Class C concrete while the second mixture was a ternary mixture used on the bridge deck on SR 23 over SR 20 near South Bend, IN (see chapter 5). The mixture composition is given in Table 4.2.

4.8.2.2 Test Procedures

From each mixture, two 76×102×406 mm prismatic specimens were per each testing age. Class C INDOT concrete was only tested at 14 days, whereas all other five mixtures were tested at both 14 and 56 days. The testing procedure of AASHTO T 161 was followed. After about every 35 cycles of freezing and thawing the specimens were removed from the F-T machine. After wiping with a damp cloth, the specimens were weighed and tested using impulse excitation method according to ASTM C 1259 (ASTM 2008d).

The relative dynamic modulus of elasticity was determined from the following formula:

$$P_c = (n_I^2/n^2) \times 100 \quad \text{Eq. 4.8}$$

where:

P_c – relative dynamic modulus of elasticity after c cycles of freezing and thawing (%);

n – fundamental transverse frequency at 0 cycles of freezing and thawing (kHz);

n_I – fundamental transverse frequency at c cycles of freezing and thawing (kHz).

The results of F-T test will be presented in a form of durability factor (DF) which was calculated as per AASHTO T 161 from the following formula:

$$DF = PN/M \quad \text{Eq. 4.9}$$

where:

DF – durability factor of the test specimen (%);

P – relative dynamic modulus of elasticity at N cycles (%);

N – number of cycles at which P reaches the specified minimum value for discontinuing the test (60%) or the specified number of cycles at which the exposure is to be terminated (300), whichever is less;

M – specified number of cycles at which exposure is to be terminated (300).

The reported results comprise the average obtained for two specimens. Due to lack of clear trends, the mass change will not be reported.

4.8.3 Results

The air content and slump values obtained are listed in Table 4.16. The air-void system parameters were not determined. The relative dynamic modulus of elasticity with number of F-T cycles for the specimens tested at 14 and 56 days is shown in Fig. 4.29(a) and (b), respectively. At the testing age of 14 days (Fig. 4.29(a)), it can be seen that the curve for the class C INDOT concrete is relatively flat, indicating good F-T resistance of that mixture. On the other hand, 20FA/5SF and 30FA/5SF mixtures experienced a gradual decrease in relative dynamic modulus from the very beginning of the test. The curves for three other ternary mixtures, i.e., 20FA/5SF, 30FA/7SF and 20FA/5SF – SR23 mixtures run parallel to each other in between the curves for class C concrete and previously mentioned ternary mixtures. The deviation of the curves for these three mixtures from the curve for class C mixture is clear after about 100 cycles. From that point on, the curves run parallel to the curves for 20FA/5SF and 30FA/5SF mixtures.

Table 4.16 – Fresh concrete properties for study on freeze-thaw resistance

Mixture	14 days		56 days	
	Fresh Air Content (%)	Slump (mm)	Fresh Air Content (%)	Slump (mm)
20FA/5SF	7.3	170	6.8	140
20FA/7SF	7.0	145	6.6	100
30FA/5SF	6.3	130	6.5	165
30FA/7SF	6.5	150	5.9	125
INDOT C mix	6.8	150	-	-
20FA/5SF – SR 23	7.9	165	7.9	165

Surprisingly, this situation has not been the same for the specimens tested at 56 days (Fig. 4.29(b)). However, both curves for mixtures with 30% FA tested at 56 days drop dramatically from the beginning of the test and the relative dynamic modulus from both mixtures fell below only 60% after about 130 F-T cycles. Therefore, no clear relationship between the composition of the ternary mixtures and the testing age and the relative dynamic modulus of elasticity can be deduced from Fig. 4.29.

Fig. 4.29 – Relative dynamic modulus for the specimens subjected to F-T cycles at: (a) 14 days; (b) 56 days

For an unexplained reason, both curves for mixtures with 30% FA tested at 56 days drop dramatically from the beginning of the test and the relative dynamic modulus of both fell below 60% after about 130 F-T cycles only. Therefore, no clear relationship between the composition of the ternary mixtures and the testing age and the relative dynamic modulus of elasticity can be deduced from Fig. 4.29.

The values of durability factor (DF) calculated from Eq. 4.9 are shown in Table 4.17. The INDOT class C concrete achieved very high DF of 92%. None of the ternary mixtures studied obtained the DF of at least 60% at both testing ages. In general, the best performing ternary mixtures appeared to be the ones with 20% FA and 5% SF, potentially implying that the susceptibility of concrete to damage caused by F-T action increases with the increase in the total amount of mineral admixtures incorporated in concrete. This is in agreement with findings of Toutanji et al. 2005.

Table 4.17 – Summary of durability factor (DF) values

Mixture	DF (%)	
	14 days	56 days
20FA/5SF	56	64
20FA/7SF	67	46
30FA/5SF	51	40
30FA/7SF	67	35
INDOT C mix	92	-
20FA/5SF – SR 23	64	54

4.8.4 Discussion

It should be realized that the test results exhibited significant variability within a set of two specimens cast from mixture. It was observed that the abrupt failure of several specimens (regardless of the composition) was caused by failure of coarse aggregate, often visible at the surface of the specimens. The failure of individual aggregate particles at the surface of concrete, referred frequently to as pop-outs, occurred in some case as early as only after 30 cycles of freezing and thawing. The example of chert particles popped out from one face of the test beam is illustrated in Fig. 4.30. It can be seen, that

apparently, the stress exerted by both aggregate particles failing close to each other on the same side of the specimen led to extensive cracking and ultimate disruption of concrete.

Fig. 4.30 – Example of aggregate failure in the freezing-thawing test

It appeared that the failure of concrete specimens subjected to cyclic F-T action may be primarily attributed to F-T resistance of aggregate. In other words, whether a given specimen performed well in the F-T test was likely controlled mainly by the quantity and location of the deleterious aggregate particles. However, it should be noted that class C INDOT concrete mixture achieved very high durability factor (92%) and the discussed above aggregate failure experienced by the ternary mixtures, was not observed for the class C mixture.

It is unclear whether the superior F-T resistance of class C INDOT mixture was attributed to its higher paste content compared to the ternary mixtures (and therefore reduced likelihood of inclusion of deleterious aggregate particles) or rather to the composition of the paste. The first argument does not seem to be valid, since the paste content of the 30FA/5SF mixture was 28% which is almost identical to that of class C mixture (29%). Regarding the paste composition, there are two major issues which should be considered, as discussed below.

As mentioned previously, the amount of cement in class C mixture is far greater compared to all ternary mixtures studied (390 kg/m^3 vs. 231 kg/m^3 , respectively). Pigeon et al. (1986) suggested that the compromised F-T resistance of concrete containing silica fume may be caused by substantial refinement of pore structure compared to plain cement paste, as conceptually illustrated in Fig. 4.31. Accordingly, the air-void system becomes less efficient since the number of pores, despite smaller cross-section, which do not intercept the entrained air voids is greatly increased (Fig. 4.31(b)). The flow of super-cooled water to the air voids is therefore hindered, which may result in generation of considerable stresses leading to microcracking.

Fig. 4.31 – Conceptual illustration of effect of refinement of pore structure on efficiency of air voids in protection against freezing and thawing

In addition, the pore structure of cement paste containing supplementary cementitious materials is known to become depercolated at significantly earlier ages compared to plain OPC pastes. Since the standard procedure for curing of the F-T specimens requires continuous moist curing until the testing time, the paste not only becomes saturated to the highest possible level (under normal conditions) but the water is also trapped in the depercolated pore system. As a result, although the air voids are present in the system, they may not provide adequate protection against freeze-thaw action to significant amount of those depercolated pores.

This explanation, if true, gives rise to the following questions. Does concrete with SCMs require more entrained air to ensure F-T resistance? If so, what are the parameters of air-void system to provide satisfactory resistance of HPC to freeze-thaw action? To the best knowledge of the author, up-to-date no data published dealing with the above questions exists, hence more research is needed to determine whether F-T resistance can be improved by providing more effective air-void system in concrete. This issue will be resolved in the following section.

4.8.5 Conclusions

Due to large impact of aggregate failure on freeze-thaw test results, the interpretation is somewhat difficult complicated. However, the following conclusions could be drawn:

- Class C concrete clearly performed better than any of the ternary (OPC/FA/SF) mixtures, which themselves generally did not perform satisfactorily, regardless of content of FA and SF.
- Freezing and thawing resistance appeared to decrease with an increase in the content of fly ash in the mixture. The influence of SF content on the F-T resistance is not clear.
- Extended curing period from 14 to 56 days did not result in significant improvement of F-T resistance, as suggested by several researchers.

4.8.6 Final Remarks

It should be realized that the pore structure of the ternary concrete is very different from that of the plain cement concrete in that it likely contains higher percentage of pores which become depercolated with time. These pores trap water that can expand upon freezing. The possibility of water entrapment is certainly high when specimens are continuously moist cured for 14 days, as prescribed by AASHTO T 161. As a result, higher susceptibility to F-T damage is observed. The validity of the above hypothesis is further confirmed by the fact that (as will be shown in section 4.9.3) the ternary concrete

subjected to more realistic (lack of continuous moisture exposure) curing conditions perform significantly better than continuously moist cured concrete.

4.9 Influence of Air Content and Initial Curing Condition on Freezing-Thawing Resistance and Transport Properties of OPC/FA/SF Concrete

4.9.1 Introduction

This study was driven mainly by the problem with maintaining the appropriate amount of entrained air in the fresh concrete placed in several lots of Phase 1 of the bridge deck on SR 34 over US 20 near South Bend, IN (broadly discussed in chapter 5).

Most of the current specifications for important structures, such as bridge decks, are very rigorous on the required air content in fresh concrete. Frequently, this is the only parameter describing the air-void system to be used on the particular structure, regardless of the composition of concrete. The downside of this somewhat arbitrary approach is that depending on their composition, different concretes may require very different air content to provide protection against freezing and thawing action. For example, the most commonly specified air content of 6.5% for bridge deck concrete exposed to freezing and thawing action is a historic number which was proposed several decades ago for plain cement concrete. However, based on the freeze-thaw test results discussed in section 4.8, it is not clear that the same content of entrained air is also needed for HPC. In addition, the use of fresh concrete air content as a sole indicator of the predicted freezing-thawing resistance is often questioned, as it poorly reflects the quality of air-void system of hardened concrete (Klieger 1994, Radlinski et al. 2008).

The current testing procedure for determining resistance to freezing and thawing (AASHTO T 161) requires that the specimens are continuously moist cured until the age of testing. This has brought up many arguments that some structures, e.g. bridge decks, will never experience such a severe exposure. Interestingly, it has been shown that the initial moisture content plays a crucial role in performance of concrete in AASHTO T 161 test (Gagne et al. 2003). The damage due to F-T action will not be as severe if the specimens are not continuously moist cured prior to testing, although the

uptake of water still takes place during the F-T cycles. Although this phenomenon is not yet fully understood, it can potentially be attributed to the fact that some air voids are being filled with water when continuously moist cured, but they will not be filled to the same extent in concrete with lower than the maximum moisture content, even if water ingress takes place during the test itself. Nevertheless, the test performed in accordance with AASHTO T 161 represents the worst case scenario, which, as it was mentioned before, discriminates concrete which may never be subjected to this type of condition given particular application it is utilized for.

There were several objectives of this research, of which the most important ones were to determine how the initial air content in fresh concrete influences the following:

- Freeze-thaw resistance of hardened concrete;
- Air-void system parameters of hardened concrete;
- Transport-related and mechanical parameters of hardened concrete, which could potentially affect its performance in F-T test.

The final goal was to examine the F-T resistance of concrete specimens subjected to various curing regimes likely to be encountered in the field.

4.9.2 Experimental Details

4.9.2.1 Mixture Proportions and Target Air Content

The testing was conducted on one preselected mixture which contained 30% fly ash and 5% silica fume (by mass of binder). Of the four ternary FA/SF mixtures studied, this mixture was believed to represent to worst case scenario in terms of potential damage resulting from F-T action, as in the preliminary optimization study (Chapter 3) it developed the lowest strength and the lowest resistance to chloride-ion penetration (under standard curing conditions). A total of four batches of concrete were prepared. The initial amount of all but one (air entraining agent dosage) ingredients in these batches was

kept constant. The dosage of air entraining admixture was adjusted to obtain the fresh concrete air contents of, respectively, 6.5%, 5.5%, 4.5% and 3.5%.

4.9.2.2 Specimens Preparation and Curing Conditions

Eight freeze-thaw specimens were cast from each of four mixtures prepared in this study. Each pair of specimens was subjected to one of four curing conditions evaluated. Preparation and testing procedures of freeze-thaw specimens was identical as in the previous study and is described in section 4.8.2.

Sixteen companion 102×203 mm cylinders were also cast from each mixture. Each group of four specimens was subjected to one of four curing conditions. Two 102×203 mm specimens were tested for splitting-tensile strength. The other two specimens were cut using a water-cooled diamond saw and three 51×102 mm disks were obtained from each cylinder. Only about 2 mm was cut off from the top of the specimens. The top disks were used for testing for absorptivity, the middle ones were tested for RCP and the bottom ones were used for determination of permeable porosity.

In addition to standard moist curing as specified by AASHTO T 161, three other curing regimens were employed in the study. These included curing under wet burlap and plastic sheets for 3 or 7 days, and curing using a curing compound applied over the entire surface of test specimen for 7 days. In the case of freeze-thaw specimens, following the initial curing conditions the specimens were allowed to dry at 50% RH until resaturation time. Regardless of the type of initial curing regime, all specimens were “resaturated” by placing them in moist room for 48 hours prior to testing time, which in all cases (including the standard moist cured specimens) took place at 14 days.

4.9.2.3 Test Procedures

The absorptivity test was performed using a method similar to that described in ASTM C 1585. However, the standard method was modified to allow for the inflow of water from the top of the test specimen rather than from the bottom as required by the standard. The details regarding absorptivity test were previously discussed in section 4.3.2.2.

The rapid chloride permeability test was conducted according to AASHTO T 277, whereas the permeable porosity was determined following the modified ASTM C 642 (ASTM 2006b) method. The modification involved obtaining the saturated state of the specimen using vacuum-saturation method (following the procedure used for conditioning RCP specimens described in AASHTO T 277) rather than through boiling of specimens for 5 hours. The premise of the above modification was that the vacuum saturation provides higher saturation level than the boiling method for the specimens tested 24 hours after saturation is initiated (Safiuddin and Hearn 2005). However, in the present study the SSD mass and mass after immersion were checked periodically and were determined after no mass gain was observed. The period required for the mass of saturated specimen to stabilize was substantial and frequently was longer than one month. This has two implications. One is that it is difficult to saturate concrete incorporating SCMs due to very fine capillary pores. The second is that, as a result of this lengthy procedure, the obtained permeable porosity values do not reflect the true porosity at the age of 14 days when drying was initiated. This is because during saturation hydration was still progressing.

In addition, for each mixture the air-void system parameters were determined on rectangular specimens obtained by cutting the 76×152 mm cylinder along the longitudinal plane. The modified point-count method was used as per ASTM C 457 (ASTM 2008b).

4.9.3 Results and Discussion

The measured slump values obtained for the mixtures prepared in this study ranged from 110 mm to 180 mm. The air-void system parameters for all four types of mixtures determined using a modified point-count method are given in Table 4.18. It can be seen that, with the exception of the concrete with the air content of 4.2%, the hardened concrete air content was found to be lower than the fresh concrete air content by about 1% or more. Interestingly, the void frequency appears to be the parameter affected by the air content the most. In fact, the specific surface was almost identical for all four

concretes. The values of the spacing factor of all but one (C4 with 3.4% of air) concretes were slightly below the specified 0.20 mm, which is the value commonly specified for F-T resistant concrete (Hover 1994).

Table 4.18 – Air-void system parameters

Concrete Type	C1	C2	C3	C4
Fresh Air Content (%)	6.3	5.3	4.2	3.4
Hardened Air Content (%)	5.2	3.8	4.2	2.5
Void Frequency (voids/mm)	0.34	0.27	0.28	0.16
Specific Surface (mm^2/mm^3)	25.9	28.7	26.6	26.0
Spacing Factor (mm)	0.18	0.18	0.19	0.25

The changes in the relative dynamic modulus of elasticity as a function of number of F-T cycles and curing conditions are shown in Fig. 4.32. In the case of concrete with 6.3% (Fig. 4.32(a)) the curve for moist cured specimens gradually descends from the very beginning of the test and falls below 60% after about 230 cycles. The curves for the other three curing conditions all fall into a single group. They start to decline after about 100 cycles and drop to the level of roughly 80% of the initial value of the elastic modulus by the end of 300 cycles.

The durability factors (DF) obtained by each mixture subjected to various curing regimes are provided in Table 4.19. It can be seen that although the DF obtained by the concrete with highest air content (6.3%) was the highest, none of the moist cured specimens had a DF above the commonly used acceptance limit of 60%. The lack of clear difference in performance in F-T test between the concretes with different air content was attributable to the fact that the parameters of the air-void system in hardened concrete were very similar. The performance of moist cured specimens in the F-T test in all cases but one was significantly lower than that of specimens subjected to any other curing condition. This was in spite of the fact that the latter groups of specimens were “resaturated” for 48 hours prior to testing. No systematic trends have been observed for the specimens cured using either curing compound or wet burlap.

Fig. 4.32 – Relative dynamic modulus of concrete with fresh concrete air content of:
(a) 6.3%; (b) 5.3%; (c) 4.2%; (d) 3.4%

Table 4.19 – Durability factors for different air contents and curing conditions

Concrete Type	C1	C2	C3	C4
Fresh Air Content (%)	6.3	5.3	4.2	3.4
Hardened Air Content (%)	5.2	3.8	4.2	2.5
Moist cured	51	42	46	43
7 days curing compound	74	74	91	72
3 days burlap	87	71	70	59
7 days burlap	82	39	71	76

Fig. 4.33 shows the relationship between durability factor and either fresh or hardened concrete air content as well as void frequency and spacing factor. Unfortunately, none of these four parameters appear to give a reasonable indication of freeze-thaw resistance. The only distinctive trends for all four parameters can be observed for “3 days burlap” group of specimens. However, the plots illustrate very well the impact of curing mode applied to the specimens prior to testing on the durability factor.

Fig. 4.33 – Relationship between durability factor and: (a) fresh concrete air content; (b) hardened concrete air content; (c) void frequency; (d) spacing factor

It can be seen that, in general, the continuous moist curing, which provides the most unfavorable curing condition, results in a durability factor lower by about 30% than that obtained by “7 days curing compound specimens”. This apparent parallelism of the plots these two curing regimes implies that the effect of curing condition is independent of the quality of air-void system, represented either by air content, void frequency or spacing factor. The line plots for specimens cured under wet burlap for 3 or 7 days are typically between the two aforementioned plots, which is an indication of strong dependence of freeze-thaw test on the moisture content of specimens at the beginning of the test. Therefore, it can be concluded that the 48-hour “resaturation” of the specimens subjected to different curing conditions was not sufficient to bring the saturation level of these specimens to comparable the levels of continuously moist cured ternary concrete.

The results of additional tests performed were found to be affected by the air content (Table 4.20). In general, as the air content decreased, the splitting tensile strength increased, whereas the RCP, initial rate of absorption and permeable porosity were reduced. Evidently, not only does the entrained air have some influence of freeze-thaw resistance (in this study to a limited extent) and strength of concrete, but it also significantly impacts the transport-related properties. Therefore, increasing the air content even above the current target value of 6.5% appears advantageous, as it may result in further enhancement of transport-controlling properties of concrete and lead to increased freeze-thaw resistance.

Also, while the effect of curing conditions on the splitting tensile strength are not clear, permeable porosity was affected by curing conditions only to a limited extent (Table 4.20). In contrast, the curing regimes had a marked impact on RCP and initial absorptivity values. Specifically, likely due to higher saturation level at the time of testing, the continuously moist cured specimens exhibited the highest RCP test results and the lowest initial absorptivity values. Among the three other (nonstandard) curing conditions, the RCP values did not vary significantly, whereas initial absorptivity was consistently the lowest for 7 days wet burlap curing.

Table 4.20 – Results of test performed on cylinders companion F-T specimens

Fresh/Hardened Concrete Air Content (%)	Curing Condition	Splitting Tensile Strength (MPa)	RCP (coulombs)	Permeable Porosity (%)	Initial Absorptivity $\times 10^{-4}$ (mm/s ^{1/2})	Secondary Absorptivity $\times 10^{-4}$ (mm/s ^{1/2})
6.3/5.2	moist cured	3.6	2656	18.7	5.6	2.6
	7d curing compound	3.7	2013	18.6	66.2	10.2
	3d burlap	3.8	1698	18.9	54.7	14.0
	7d burlap	4.5	1583	18.8	24.4	9.9
5.3/3.8	moist cured	3.5	3032	18.2	7.2	2.1
	7d curing compound	2.9	2231	18.7	58.9	7.0
	3d burlap	4.3	1926	17.9	63.7	13.3
	7d burlap	3.6	2040	18.3	36.2	9.3
4.2/4.2	moist cured	3.7	4627	18.3	5.6	1.6
	7d curing compound	4.1	2992	18.4	54.7	7.2
	3d burlap	4.3	3195	17.9	69.7	15.8
	7d burlap	3.9	3029	17.8	47.0	8.3
3.4/2.5	moist cured	4.5	3366	17.1	10.4	3.1
	7d curing compound	3.3	2753	16.6	49.3	4.9
	3d burlap	4.1	2735	17.0	66.4	11.3
	7d burlap	4.4	2562	17.3	42.4	7.7

4.9.4 Conclusions

The freeze-thaw resistance of ternary concrete selected in this study and subjected to continuous moist curing was unsatisfactory even though the air-void system parameters were well within the recommended ranges for freeze-thaw resistance concrete. This may be due to several reasons including relatively high fly ash content (30%), presence of deleterious aggregate particles (cherty limestone) and potentially lower efficiency in protection against F-T damage of air bubbles generated by superplasticizer.

The specific effect of quality of air-void system parameters on freeze-thaw resistance could not be properly interpreted from this study, since the parameters of the air-void system in hardened concrete were very similar despite very different air content. Thus, more research is needed to determine whether higher air content would result in satisfactory F-T resistance. It would also be very beneficial to find the “threshold” values of fresh air content as well as that of air-void system parameters necessary to provide HPC with good freezing and thawing resistance. In addition, the increase in air content appeared to be advantageous in terms of enhancement of transport-controlling properties of ternary concrete.

The obtained results indicated that the “standard” continuously moist cured specimens perform significantly worse than those subjected to other curing conditions. This was despite 48-hours long resaturation of “nonstandard” cured specimens prior to testing (as recommended by AASHTO T 161 method for the specimens obtained from field). However, this procedure has not proven to be efficient for the ternary mixtures tested. In fact, this approach introduced an extra variable (in a form of moisture content at the beginning of test), thus making the interpretation of effect of curing condition on freeze-thaw resistance virtually impossible.

CHAPTER 5 – PILOT STUDY OF FIELD HPC INSTALLATION ON SR 23 BRIDGE DECK

5.1 Introduction

This chapter deals with the research conducted in order to evaluate the field performance of the ternary concrete used in pilot HPC bridge deck on SR 23 over SR 20 near South Bend in Indiana. The description of events related to the bridge deck construction, as well as evaluation of the performance of laboratory-made mixture of the same composition, is provided. This chapter contains also the results of failed material investigation carried out through both field and laboratory studies and the observations from the recent inspection of the condition of bridge deck are discussed. In addition, this chapter discusses the applicability of maturity method for prediction of compressive strength of field concrete. Further, the observations from the site visit and evaluation of the condition of the bridge deck is discussed. Finally, the development of maturity method for prediction of resistance to chloride-ion penetration along with the proposed applications of the method has been described.

5.2 Installation of HPC on SR 23 Bridge Deck

As a part of this research project, the investigation associated with the recent construction of HPC concrete bridge deck on SR-23 in South Bend, Indiana was undertaken. The construction was divided into two phases, each consisting of two lanes. Phase 1 bridge deck construction (BDC-1) took place on November 3rd, 2004, whereas Phase 2 construction (BDC-2) was conducted on May 17th, 2005. From the technological point of view, the only difference between the two construction phases was that the BDC-1 concrete was placed using a pump (Fig. 5.1(a)), while a crane and bucket (in some portions) (Fig. 5.1(b)) or conveyor belt (Fig. 5.1(c)) were used to place BDC-2

concrete. The deck constructed during either phase was cured for 7 days using wet burlap covered with polyethylene sheets.

Fig. 5.1 – Bridge deck construction using pump (a) during Phase 1, crane and bucket (b) during Phase 2 and conveyor belt (c) during Phase 2

As it will be discussed broadly in the course of this chapter, the placement technique had a large impact on the quality and performance of concrete. The use of

pump during construction Phase 1 resulted in significantly compromised air-void system of fresh concrete and resulting freezing-thawing resistance.

Prior to the actual construction, the key properties of the proposed ternary concrete mixture were established during the field trial batch demonstration (FTB) carried out by the ready-mix concrete producer that was selected to supply concrete for the actual bridge deck construction. In fact, two field trial batch demonstrations took place, one on September 16th 2004 and one on October 5th 2005, however since the first trial batch was not successful, the analysis of results will be mostly focused on the results obtained from the second trial batch.

During both construction phases and the field trial batch demonstration, apart from acceptance specimens collected as per standard INDOT's QC/QA procedures, additional specimens were cast in order to assess various properties of concrete. Furthermore, to verify the quality of in-situ concrete, two sets of cores were retrieved from BDC-1 phase.

The concrete mixture of the same composition was also produced in a laboratory (LTB) and the monitored properties attained by that mixture were compared with the properties of field-made concrete (from both construction phases and field trial batch). This comparison has been presented in the following section of the report.

5.3 Evaluation of Field- and Laboratory-Made Concrete

5.3.1 Experimental Details

5.3.1.1 Mixture Composition and Materials Used

The criteria related to the concrete mix design (CMD) set out on for the SR-23 project were as follows (INDOT 2004):

- The paste volume of total cementitious material and water shall not exceed 28% of concrete volume design value.
- The cement content in the ternary system shall be at least 231 kg/m³ of concrete.
- Class F or C fly ash shall be used as part of the total cementitious materials content in the ternary binder system. Fly ash shall constitute 20 to 30 percent by mass of total cementitious materials content in the mix design.

- Silica fume shall constitute 5-7 % of the total cementitious materials content in the mix design.
- The water-cementitious materials ratio shall be no less than 0.38 and not exceed a maximum of 0.42.
- The CMD target air content shall be set at either 6.5% or 7.0%, with a 100% pay factor range of 5.9-8.9 %.
- The slump shall be within a range of 100 mm to 190 mm.
- The target compressive strength at 28-days shall be a minimum of 33.8 MPa.

Table 5.1 – Physical properties and chemical composition of cementitious materials used in field study

Property	Cement	Fly Ash	Silica Fume
<i>Physical</i>			
Specific Gravity	3.15	2.67	2.20
Fineness – Passing 325 Mesh	99	9.6	–
– Blaine’s Surface Area (cm ² /g)	3620	–	–
Compressive Strength Tested on Mortar	2310	–	–
Cubes (psi) – 1 day	3890	–	–
– 3 day	4870	–	–
– 7 day	6060	–	–
– 28 day			
<i>Chemical</i>			
Silicon Dioxide SiO ₂ (%)	20.60	36.16	93.07
Aluminum Oxide Al ₂ O ₃ (%)	4.70	20.32	0.62
Ferric Oxide Fe ₂ O ₃ (%)	2.60	7.58	0.41
Calcium Oxide CaO (%)	64.90	23.94	0.66
Magnesium Oxide MgO (%)	2.60	5.47	1.16
Sulfur Oxide SO ₃ (%)	2.50	1.91	<0.01
Loss on Ignition (%)	1.31	0.40	2.71
Total Alkali as Sodium Oxide Na ₂ O (%)	0.53	1.35	0.67
Insoluble Residue (%)	0.34	–	–
<i>Potential Compound Composition (Bogue)</i>			
Tricalcium Silicate C ₃ S (%)	65	–	–
Dicalcium Silicate C ₂ S (%)	10	–	–
Tricalcium Aluminate C ₃ A (%)	8	–	–
Tetracalcium Aluminoferrite C ₄ AF (%)	8	–	–

The physical properties and chemical composition of the cementitious materials are provided in Table 5.1. The selected mixture proportions are presented in Table 5.2. The binder system contained ordinary portland cement (ASTM C 150 Type I) combined with 20 % of class C fly ash and 5 % of densified silica fume, by weight of

total cementitious materials (332 kg/m^3). The nominal water to cementitious materials ratio (w/cm) was 0.40, while the paste content was 24% by volume.

All four series of mixtures described in section 5.2 were produced using the same source of materials. The only exception was high-range water reducing admixture (HRWRA) used in LTB mixture, which was provided by a different manufacturer than the HRWRA used in the field mixtures. The coarse aggregate used was crushed limestone with specific gravity (SSD) of 2.71 and absorption of 1.14 %. The fine aggregate used was natural siliceous sand having specific gravity of 2.63 and absorption of 1.24 %. Both fine and coarse aggregate were stockpiled separately for this project, and essentially the same material was used in both construction phases as well as in field and laboratory trial batches. For the LTB mixture the coarse aggregate was sieved and recombined to ensure the same gradation as that of the stockpiled material.

Table 5.2 – Mixture composition of field concrete

Material	Cement	Fly Ash (Class C)	Silica Fume	Fine Aggregate (SSD)	Coarse Aggregate (SSD)	Water
Content (kg/m^3)	249	66	17	778	1062	133

5.3.1.2 Tests Conducted and Testing Methods Used

Standard QC/QA tests, such as slump, air content, and unit weight were performed on fresh concrete. To evaluate the resistance of concrete to chloride ion penetration, the rapid chloride permeability (RCP) test (ASTM T 277) was used in the study. The reported results are based on the average obtained from four 52 mm thick disks, two sampled from the top part of the 102×203 mm cylinder (after discarding the top 6.5 mm) and two sampled from the bottom part of a cylinder (114-165 mm from the top). The RCP test results reported for the cores retrieved from the bridge deck (and tested at 149 and 303 days) were obtained from the 51×95 mm disks after removal of the top 12 mm.

The free shrinkage test was performed on three 76×76×286 mm prismatic specimens following the procedure of AASHTO T 160 (AASHTO 2006c). The restrained shrinkage test, frequently called the ring test, was conducted in accordance

with AASHTO PP 34 (AASHTO 2006a), except that the non-standard ring geometry was used (Fig. 5.2). The test method involved casting of concrete ring, with cross-sectional dimensions of 76×76 mm, in a cardboard mold placed around 10 mm thick steel ring of 305 mm external diameter, which was instrumented with four strain gages placed symmetrically at the mid-height. 24 hours after casting the outer cardboard mold was removed and the top and bottom surfaces of the ring were sealed with aluminum tape. As a result, only the outer circumference of the ring was undergoing drying. Two replicate specimens were cast in the case of FTB and BDC-1 mixtures, while only one ring was fabricated for LTB. A third ring was also cast to monitor the temperature developing in BDC-1 concrete, as shown in Fig. 5.2(b).

Fig. 5.2 – (a) Restrained shrinkage test setup; (b) BDC-1 ring test specimens after casting; (c) LTB ring test specimen

For comparative purposes, shrinkage data collected directly from the bridge deck poured during Phase 1 construction will be also presented. These data were obtained from two vibrating wire strain gages located in the deck; one gage was placed on the reinforcing bar located in the mid-span of the deck at equal distance from the

girders, and the second gage was embedded in the surrounding concrete (Frosch and Aldridge 2006).

Compressive strength tests were carried out on three cylinders at each testing age. 102×203 mm cylinders were utilized in the case of BDC-1, BDC-2 and LTB, while for FTB 152×305 mm specimens were used.

5.3.1.3 Curing Conditions

The effect of curing conditions on the examined properties of HPC was of major interest in this work. Focus was made on the impact of the early age temperature on permeability and compressive strength for BDC-1 and BDC-2, as well as on the influence of moisture availability on the strength development for the FTB. The detailed description of curing regimes for respective tests is provided in Table 5.3.

The BDC-1 compressive strength specimens were exposed to the simulated field conditions until the testing time. The curing procedure used for these specimens was intended to mimic the procedure used for the bridge deck itself. Accordingly, the specimens were left at the construction site for the first 24 hours. They were then brought to the laboratory at Purdue University (located about 100 miles south of the construction site) and removed from molds. For the first 6 days after demolding the specimens were covered with wet burlap and plastic sheet and stored in styrofoam containers (simulating the effect of heat of hydration experienced by the deck concrete during the curing period). Once the wet burlap curing was terminated, the specimens were removed from the containers and were continuously exposed to ambient environment.

RCP specimens, however, were field exposed until the 35th day, at which time they were split into three groups. The first group of specimens remained exposed to the in-situ environment, the second group was moved to the lab (23°C and 50% RH) and the last group was moved to the moist room (23°C and 100% RH). The ring specimens, in turn, remained exposed to the field conditions throughout the entire testing period.

In the case of BDC-2, both compressive strength and RCP specimens were divided into two groups from the very beginning. The first group underwent standard

(continuous) moist curing and the other one was subjected to field exposure after 7 days of curing under wet burlap covered with polyethylene sheet (at ambient temperature).

Table 5.3 – Curing conditions for each test and study case

Source of specimen Test	BDC-1	BDC-2	FTB	LTB
RCP	1) 7 days of wet burlap and plastic sheet followed by field exposure*	1) 7 days of wet burlap and plastic sheet followed by field exposure	Lab moist cured	Lab moist cured
	2) As in 1), from 35th day air cured @ 23.0°C and 50% RH	2) Lab moist cured		
	3) As in 1), from 35th day air cured @ 23.0°C and 100% RH			
Compressive strength	7 days of wet burlap and plastic sheet followed by field exposure	1) 7 days of wet burlap and plastic sheet followed by field exposure	1) Lab moist cured	Lab moist cured
		2) Lab moist cured	2) Lab cured @ 23.0°C and 50% RH (air drying)	
			3) Partially lab moist cured (on days 2, 5 and 9 for 6 hours)	
Free shrinkage	Lab moist cured, from 7th day @ 23.0°C and 50% RH	Lab moist cured, from 7th day @ 23.0°C and 50% RH	Lab moist cured, from 7th day @ 23.0°C and 50% RH	Lab moist cured, from 7th day @ 23.0°C and 50% RH
Ring test	4 days of wet burlap and plastic sheet followed by field exposure	-	Lab cured with wet burlap and plastic sheet, from 7th day @ 23.0°C and 65% RH	.

*Note: The term field exposure refers to 24-h storage of specimens at the job site followed by outdoor storage in the courtyard of concrete laboratory of Purdue University (about 100 miles south-east from the job site). Due to very similar air temperature cycles recorded at both locations (Fig. 5.3), the exposure conditions in the courtyard were assumed to be comparable to that experienced at the construction site.

FTB and LTB specimens for RCP test continuously moist cured at 23°C until testing. While LTB specimens for compressive strength were cured in a this standard way as well, the FTB specimens were subjected to three different curing regimes, namely moist curing, air drying (23°C, 50% RH), and intermittent moist curing (saturation on day 3, 5 and 9 for six hours, otherwise air drying). The purpose of the last curing mode was to investigate whether sporadic supply of moisture to high performance

concrete (simulating natural precipitation) can offset the negative impact of absolute lack of curing (in terms of moisture supply).

The specimens for restrained shrinkage test were cured under burlap for 7 days, followed by storage at about 65% RH and 50% RH, for FTB and LTB respectively. Free shrinkage specimens were in all cases moist cured for 7 days, which corresponded to the curing period of the actual structure, followed by exposure to 50% RH.

5.3.2 Results and Discussion

5.3.2.1 Temperature History Data

Presented in Fig. 5.3 is the in-place temperature history for the BDC-1 concrete as well as the ambient temperature at the job site and at the storage (after transferring specimens from the job site). It can be noticed that the ambient temperature for specimens transferred from the job site to the courtyard of concrete laboratory at Purdue University was very similar to the actual ambient temperature at the job site. This supports a claim of previously mentioned similarities of field exposure (see note under Table 5.3).

Fig. 5.3 – Temperature history for BDC-1 concrete (Frosch and Aldridge 2006) throughout the period of initial 90 days

The data also indicated that throughout the first 10 days after casting the concrete temperature was significantly higher than the ambient temperature due to heat liberated during early stages of hydration. Coincidentally, after about 10 days, the temperature in the deck dropped below 0°C and from that time-on remained slightly below the ambient temperature. This low overall temperature history experienced by BDC-1 as well as relatively early (21 days after construction) application of deicing salts might have led to scaling problem observed already after the first winter (several months after construction). This will be further discussed in section 5.6.2.

5.3.2.3 Fresh Concrete Properties

The unit weight and air content obtained by INDOT staff and slump obtained by the contractor for all sublots of bridge deck construction Phase 1 and 2 are listed in Table 5.4. Some problems were encountered with providing air content within the recommended range of 5.9-8.9% (Table 5.4) during the first phase of construction, and in one of the sublots sampled the air content was determined to be 3.1%. This was a result of pumping, which dramatically altered the air content throughout the entire Phase 1 construction.

Fig. 5.4 shows the comparison of test results obtained for fresh concrete; however for BDC-1 and BDC-2 concretes only the results obtained for the subplot from which concrete was sampled for casting of specimens have been used. It can be seen, that in all cases the measured slump fell well within the required range of 100-190 mm (Fig. 5.4). The measured unit weight was in all cases within 1% of the theoretical value (calculated based on the measured air content), indicating relatively accurate batching of mixing water. This is also evident from the opposite trend in the air content and unit weight results (Fig. 5.4).

Table 5.4 – Fresh concrete properties of BDC-1 and BDC-2

Lot #	Sublot #	Construction Phase	Slump (mm)	Unit Weight (kg/m ³)	Air Content (%)
1	1	1	160	2388	3.1*
	2		150	2380	4.4
	3		160	2363	4.6
	4		150	2364	4.3
	5		150	2326	5.5
	6		140	2376	3.9*
2	1	1	160	2342	5.0
	2		150	2286	7.0
	3		160	2302	6.5
	4		165	2340	4.8
	5		130	2284	7.5
	6		n/a	2303	6.7
3	1	2	n/a	2267	8.0
	2		130	2283	7.0
	3		140	2279	7.5
	4		150	2299	7.0
	5		130	2287	7.5
	6		160	2288	7.5
4	1	2	140	2340	7.0
	2		150	2316	6.5
	3		130	2334	5.5
	4		150	2298	6.1
	5		150	2276	6.9

Note: n/a – not available; * denotes result failing the subplot acceptance limit

Fig. 5.4 – Fresh concrete properties for field study

5.3.2.4 RCP Test Results

The data presented in Fig. 5.5 clearly illustrate large variation in the RCP results obtained for different cases studied. The best results were obtained for the BDC-2 concrete cured under field conditions. Moist cured laboratory trial batch concrete achieved similar

coulomb values at a given age, while moist cured BDC-2 concrete yielded slightly higher penetrability. The FTB concrete, also moist cured until the testing time, achieved relatively high RCP value at 28 days (about 2500 coulombs), but significant drop occurred by the 56th day, and a commonly used for bridge decks requirement of maximum 1000-1500 coulombs at 56 days (INDOT 2004) was satisfied. However, BDC-1 exposed to field conditions exhibited very high penetrability even after 150 days. Very low RCP results (average of 520 coulombs) were obtained only after 303 days for additional cores retrieved from the bridge deck. Also, noteworthy is the fact that transferring specimens to either laboratory or to moist room at the 35 days resulted in a very substantial coulomb reduction already at 56th day (Δc_{lab} and Δc_{moist} respectively). This reduction can most likely be attributed to increased hydration rate.

Fig. 5.5 – Rapid chloride permeability test results for field study

5.3.2.5 Compressive Strength Test Results

Despite having the highest air content of all concretes tested (see Fig. 5.6), and therefore being in the position to have its strength most adversely affected (Hale et al. 2005), concrete produced under laboratory conditions developed the highest strength (Fig. 5.6).

This might be explained by more precise batching and more thorough mixing taking place in the laboratory.

It is also important to notice that, unlike resistance chloride-ion penetration, compressive strength was not as sensitive to low temperature curing, and therefore attaining desired levels of 28-day strength by BDC-1 concrete did not cause difficulties (Fig. 5.6). Similar findings were obtained by Khan and Ayers (1993) and Bentur and Jaegermann (1991), who concluded that the adverse effect of inadequate curing conditions at the early age is more pronounced in the case of concrete permeability than in the case of strength.

Fig. 5.6 – Compressive strength development for BDC, FTB and LTB concrete

It should be noted that with an exception of FTB concrete compressive strength was tested on 102×203 mm cylinders. Since, for FTB mixture, 152×306 mm cylinders were used, a correction of about 5-10% of strength at a given age should be considered in order to directly compare the test results (Plante et al. 2000). Such correction would result in the upward shift of the entire strength curve of FTB concrete shown in Fig. 5.6.

Further, the FTB specimens cast for compressive strength were divided into three groups, each subjected to different curing regime. The intermittent moist curing, being between the two extremes, such as moist curing and air drying, represents a situation in which concrete is subjected to non-continuous moisture supply. It can be seen that up to three days the curing regime did not affect strength at all (Fig. 5.7). However, starting from the fifth day a progressive relative loss of strength with respect to moist cured specimens can be observed especially in the case of specimens subjected to air drying. For this treatment, the time-strength curve flattens down very rapidly, and no strength gain was encountered after 14 days. These results indicate that in order for hydration to progress, certain supply of water is necessary. Furthermore, even relatively limited amount of moisture provided during the early age, can help to develop strength comparable with that of continuously moist cured concrete.

Fig. 5.7 – Compressive strength development for FTB as a function of moisture availability

Similar conclusion can be drawn on the basis of previously discussed Fig. 5.6 clearly illustrating the influence of moisture availability on compressive strength. It can be noticed that specimens which were moist cured under wet burlap for seven days

only, developed similar compressive strength to those which were continuously moist cured. Negligible difference which occurred after 56 days became slightly more pronounced at 90 days. This proves that, with respect to compressive strength, 7 days of moist curing is sufficient and due to formation of relatively impermeable and discontinued pore structure, further curing does not result in significant increase in compressive strength.

5.3.2.6 Free Shrinkage Test Results

Similar shrinkage characteristics were obtained for BDC-1, FTB and LTB concretes (Fig. 5.8). However, much lower strains were developed in concrete produced during the second phase of construction (BDC-2). This unusual departure from data collected for the other cases can be explained by the fact, that as mentioned previously, concrete was placed in May on a relatively warm and sunny day. Even though shrinkage prisms were covered with wet burlap and plastic sheet, water might have evaporated quickly, which resulted in premature drying of the specimens. Upon transferring them from the job site to the laboratory (approximately 24 hours after casting) there were no visible signs of moisture on the surface. Using the standard (AASHTO T 160) procedure, the specimens were soaked in saturated lime water about 45 minutes before taking the initial length measurements, which might have not resulted in effective re-saturation. As a result, the initial readings were not taken in the state of sufficient moisture content, and accordingly the readings taken after subsequent 6 days of moist curing yielded an abnormal expansion (about $145 \mu\epsilon$).

It should be noticed that if the shrinkage curve for BDC-2 was shifted down by about 120 microstrains, the strains at 119 days would be 430 microstrains which comparable to results obtained for other cases. This shows that lack of moisture provided during the initial 24 hours period may potentially lead to misleading results and false conclusions.

Fig. 5.8 – Free shrinkage over time

5.3.2.7 Restrained Shrinkage Test Results

The restrained shrinkage test carried out for BDC-1, FTB and LTB concrete showed several interesting features (Fig. 5.9). Due to combination of insufficient restraint level provided by the non-standard steel ring and relatively low shrinkage level, none of the rings cracked within the testing period. Also, the maximum strain levels attained reflects the relative humidity at which the specimens were stored during the test, i.e., 50% for LTB, about 65% for FTB, and 77% (on average) for BDC-1. Furthermore, considerable fluctuation in the strains can be observed for BDC-1 concrete which occurred due to ambient temperature and relative humidity changes.

It can be also seen that the ring test results for BDC-1 are relatively close to the actual strains in the bridge deck measured with two strain gages, one located on the rebar and another embedded in the concrete. Therefore, it can be concluded that the restrained shrinkage test results represent strain level which is very comparable with that which might occur in the actual structure, provided that the test specimen is subjected to similar relative humidity.

Fig. 5.9 – Restrained shrinkage over time, BDC-1 (rebar and embedded) data from Frosch and Aldridge (2006)

5.3.3 Conclusions

The following conclusions have been drawn on the basis of this study:

- In the ternary cementitious system investigated, the early age curing temperature had strong influence on resistance to chloride ion penetration. Over time, low-temperature cured concrete developed resistance to chloride-ion penetration which was comparable to that of normal temperature cured ternary mixtures.
- The ternary concrete placed and cured at low temperature was capable of developing sufficient 28-day compressive strength.
- No significant difference in shrinkage characteristics was observed between the mixtures prepared using different production methods.
- Mixture produced and cured under laboratory conditions exhibited only slightly better performance than the mixtures produced in the field.
- Intermittent moist curing of field trial batch concrete specimens (simulating the effects of natural precipitation) was found to have a very beneficial effect on strength development and, consequently, on the accuracy of strength prediction

using the maturity method. Thus, even occasional supply of moisture to ternary concrete, especially at early age, may significantly minimize the potential strength losses.

Since the concrete placed in the bridge deck on SR-23 during construction Phase 1 did not possess adequate air content in the fresh state, the extensive investigation has been undertaken in order to evaluate the quality of that concrete as well as to determine potential reasons for air loss due to pumping. The outcome of this investigation will be broadly discussed in the section 5.5.

5.4 Predicting Compressive Strength of Field Trial Batch and Bridge Deck Concrete

Using Maturity Method

5.4.1 Introduction

The data presented in a section 4.5 demonstrated that the maturity method can be used to effectively predict compressive strength of ternary (OPC/FA/SF) concretes. It should be realized that the research described in the former section was conducted under well-controlled laboratory conditions, with careful control of batched quantities of materials. In addition, the specimens were stored at constant (room) temperature throughout the entire strength and temperature monitoring period.

The purpose of this study was to verify whether the maturity method is applicable to ternary concrete made and placed under field condition subjected to ambient temperature and moisture fluctuations. The data discussed in this section pertains to concrete used in construction Phase 1 of bridge deck (BDC-1 concrete). Furthermore, the field trial batch concrete was used to develop the maturity curve and to evaluate the effect of curing regimes on precision of maturity method.

5.4.2 Experimental Details

Due to similarity in composition of the mixture used to produce FTB and BDC-1 concretes to one of the laboratory mixtures, the same activation energy (39.7 kJ/mol) was assumed when calculating the equivalent age values.

In the case of FTB concrete, one 152×305-mm cylinder per each curing condition was instrumented with the temperature recording device (I-button[®]). For BDC-1 concrete, the I-buttons were used to monitor the in-situ temperature of the deck, while a corresponding 152×305-mm cylinder, was instrumented with a thermocouple. In all cases, the temperature was recorded every 10 minutes.

Since, as mentioned previously, the BDC-1 concrete specimens were transported (after 24 hours) to the location approximately 100 miles away from the bridge construction site, the inherent assumption while making the prediction of compressive strength of the actual bridge deck concrete was that the temperature history of these two concretes was comparable. As already discussed in section 5.3.2.1 (and shown in Fig. 5.3), the temperature history of the actual structure (denoted as “Bridge deck”) and the temperature recorded at the location of the specimens (denoted as “Cylinder field exposed”) match well.

5.4.3 Results and Discussion

The compressive strength development of FTB and BDC-1 has already been shown in Fig. 5.6. As previously discussed, despite very low curing temperature, the BDC-1 concrete achieved higher strength than the FTB concrete at, respectively, testing ages of 7 or 28 days. This apparent discrepancy can be explained by markedly lower air content of BDC-1 than that of FTB (3.9% vs. 6.7%) and will be discussed broadly in the following section of the report.

Regarding the prediction of strength of the FTB and BDC-1 concrete, the same type of mathematical function as used earlier to relate the strength to the equivalent age t_e for laboratory-made concrete (Eq. 4.5) was also employed for FTB and BDC-1 concretes. The values of coefficients a and b were, respectively, 9.29 and 9.28 with $R^2 = 0.99$. As previously shown (Fig. 5.3), the temperature experienced by the BDC-1 concrete was relatively low, thus leading to reduction in the rate of strength development. As a result, the actual testing ages of 7, 28, 56 and 90 days corresponded, respectively, to the equivalent ages of 4.7, 14.7, 23.0, and 33.0 days.

The comparison of predicted and measured compressive strengths of FTB and BDC-1 concretes is shown in Fig. 5.10. The accuracy of the prediction of both air cured and intermittently moist cured FTB concrete specimens was adequate up to 14 days (Fig. 5.10(a)). The prediction of 28-day strength of specimens subjected to “air drying” curing condition was considered to be unacceptable, whereas the prediction of intermittently moist cured specimens still appeared to be satisfactory.

Fig. 5.10 – Predicted vs. measured compressive strength for: (a) FTB; (b) BDC-1
(numbers in parentheses denote testing age)

Fig. 5.10(b) shows that the predicted compressive strength of field exposed BDC-1 specimens (denoted as “BDC – Field exposed (uncorrected)”) is approximately 10 MPa lower than the actual strength determined for those specimens. Interestingly, this occurs irrespective of testing age. The limited accuracy of this prediction was considered to be unacceptable and was subject to further investigation. A thorough analysis of all factors revealed that the observed strength discrepancies were the result of the difference in air content between the FTB concrete (used for maturity function development) and the BDC-1 concrete. As shown in Fig. 5.4, the air content of FTB concrete was 6.7%, which is much higher than 3.9% obtained for BDC-1 concrete.

In order to properly account for a different amount of entrained air in BDC and FTB concretes, a correction factor was established as described below. It is commonly estimated that each percent of air added to concrete reduces its strength by about 5-10% (Aïtcin 1998, Hale and Russell 2000). However, from described in section 3.3 preliminary optimization analysis the following equation estimating the 28-day strength of ternary mixtures containing fly ash (20-30%) and silica fume (5-7%):

$$STR = 72.854 - 0.040 \times CM - 0.258 \times AIR^2 \quad \text{Eq. 5.1}$$

where STR is 28-day compressive strength (MPa), CM is mass of cementitious materials (kg/m^3) and AIR is fresh concrete air content (%).

By utilizing the “0.258” coefficient associated with the air content in, the generalized form of strength-equivalent age relationship can be expressed by Eq. 5.2:

$$S = a \cdot \ln(t_e) + b + 0.258 \cdot (A_0^2 - A_{actual}^2) \quad \text{Eq. 5.2}$$

where S , a , b and t_e have been defined earlier (see section 4.5.2.2), while A_0 is reference air content of concrete used for development of maturity function (%) and A_{actual} is actual air content of concrete for which the compressive strength is estimated (%). was used to re-calculate the predicted strength values of compressive strength. These values are denoted in Fig. 5.10(b) as “BDC – Field exposed (corrected)”.

It can be seen that the strength prediction for field exposed specimens was greatly improved by using Eq. 5.2 to correct for the actual air content. Furthermore, in order to verify validity of the proposed approach, the measured 7-day and 28-day compressive strength of standard moist cured BDC-1 specimens (at 23.0 °C) obtained from the contractor have been compared with the values predicted from Eq. 5.2. These values are labelled in Fig. 5.10 (b) as “standard lab. moist cured”. It can be observed that the prediction of 7-day strength was excellent (Fig. 5.10(b)). Although the prediction of 28-day strength was less accurate, it is still considered satisfactory. Incidentally, it is evident that the 28-day compressive strength of standard moist cured specimens was about 5 MPa higher than that of field exposed concrete tested at 90 days. This illustrates the adverse effect of low concrete temperatures on its strength development.

5.4.4 Conclusions

The strength prediction of field ternary OPC/FA/SF concrete (used for the construction of the bridge deck on SR 23) was found to be very accurate even up to 90 days, but only after a suitable correction for the difference in air content of actual specimens and specimens used for maturity function development was implemented. The accuracy of the prediction was quite good despite very low concrete temperature throughout the entire monitoring period (up to 90 days) resulting from late-season construction of the bridge deck.

5.5 Failed Material Investigation

Due to the fact that the air content of concrete in two sublots (#1 and #6 (see Table 5.4)) in the Phase 1 bridge deck (BDC-1) was low (below 4%), the entire lot 1 was subjected to failed material investigation, as stipulated by contract specification (INDOT 2004). As mentioned earlier, the low air content (when measured at the point of discharge from the boom) was caused by unexpectedly high losses experienced during pumping. It should be mentioned that as soon as contractor noticed that the air content of fresh concrete was getting below 4%, a substantial amount of air-entraining admixture was added to the truck at the construction site. As a result, the air content measured at the point of discharge from the truck (before being placed in the pump) was often as high as 11%. However, as seen in Table 5.4, even with this high initial air value of air content it was still difficult to achieve the target value of 6.5%.

In order to examine the possible reasons for and consequences of low air content in BDC-1, the previously mentioned failed material investigation was undertaken. The investigation involved testing of cores obtained from the BDC-1 itself, as well as laboratory investigation of air loss during the simulated pumping operation. In addition, an extensive literature review of the issue of air loss during pumping has been completed.

One of the objectives of the failed material investigation was to determine whether the surface scaling observed during the visual inspection of the BDC-1 deck conducted in March and April 2005 (only about six months after construction) and

November 2006 (as discussed in section 5.6.2) might have been attributable to poor quality of the air-void system.

5.5.1 Field Testing Data

As a part of field phase of failed material investigation, several cores were obtained from the construction Phase 1 bridge deck. The first group of cores obtained on March 29th and April 8th of 2005) were used for microscopical examination of air-void system, determination of resistance to chloride-ion penetration (RCP as per AASHTO T 277 (AASHTO 2006f)) and for determination of chloride concentration profiles. Additional cores for microscopical examination of air-void system, determination of resistance to chloride-ion penetration and freeze-thaw resistance were obtained in July, 2005. The results of each test will be discusses in the following sections.

5.5.1.1 Air-Void System Analysis

The air-void system analysis was performed according to ASTM C 457 (ASTM 2008b) using the modified point-count method and stereoscopic optical microscope set at magnification of 60×. Some specimens were tested at Purdue, some by INDOT staff in Indianapolis and some by both. Although the primary objective of this study was to determine the quality of the air-void parameters of failed sublots of Phase 1 (lot 1, subplot 1 and 6), several specimens from Phase 1 having relatively high fresh concrete air content (lot 2, subplot 3 and 5) and Phase 2 (lot 2, subplot 6; lot 4, subplot 4 and 5) were tested for comparative purposes. However, the testing of concrete from Phase 2 was conducted on the cast specimens, rather than cores obtained from the actual structure. Examination of the results of the air-void system analysis listed in **Error! Reference source not found.** reveals several interesting features. Regarding the uniformity of the results, it can be seen that (with the exception of specimen “5P”) the results of all three parameters (void frequency, specific surface area and spacing factor) appear to be relatively consistent within the respective sampling locations (lot and subplot).

Table 5.5 – Results of air-void system analysis of field concrete

Construction Phase	Lot #	Sublot #	Fresh Concrete Air Content (%)	Specimen #	Hardened Concrete Air Content (%)	Void Frequency (1/mm)	Specific Surface Area (mm ² /mm ³)	Spacing Factor (mm)
1	1	1	3.1	1P	5.0	0.14	11.2	0.41
				1IN	5.3	0.17	12.6	0.38
				2IN	5.4	0.16	12.1	0.33
				3IN	4.5	0.17	14.8	0.33
				4IN	6.4	0.16	10.2	0.36
				5P	4.2	0.10	9.2	0.63
				6P	5.2	0.09	6.7	0.74
		6	3.9	1P	4.9	0.13	10.6	0.41
				1IN	4.3	0.17	14.3	0.33
				2P	5.2	0.17	12.9	0.35
				3P	6.5	0.17	10.6	0.37
		2	6.5	1IN	6.3	0.20	12.9	0.34
				2IN	6.0	0.29	19.1	0.18
				3P	4.9	0.19	15.6	0.30
				4P	5.9	0.28	18.6	0.23
		5	7.5	1P	6.3	0.16	9.9	0.44
				2P	7.5	0.20	10.5	0.39
2	4	6	6.7	1IN	8.0	0.50	25.2	0.10
		4	6.1	2IN	7.4	0.45	24.3	0.13
				1IN	10.5	0.61	23.2	0.10
		5	6.9	2IN	7.8	0.52	26.4	0.10
				1IN	8.1	0.58	28.5	0.10
				2IN	7.8	0.56	28.9	0.10

Note: “P” indicates specimens tested by Purdue, “IN” indicates specimens was tested by INDOT

The hardened air content of the specimens from both failed sublots (1 and 6) from lot 1, appear to be significantly higher than the respective fresh air contents (Table 5.5). The typical values recommended for these parameters are as follows (ASTM 2008b, Hover 1994): (a) void frequency of $(0.04-0.06) \times \text{air content}$; (b) specific surface of 16-24 mm²/mm³; (c) spacing factor of 0.20 mm. As can be seen, the actual values of these parameters were: 0.09-0.17 1/mm for void frequency, 6.7-14.8 mm²/mm³ for specific surface and 0.33-0.74 mm for spacing factor. Therefore, the air void-system parameters are not sufficient from the standpoint of freeze-thaw resistance. A slight improvement of the air-void system parameters is visible in the specimens from lot 2 (construction Phase 1). However, even though the fresh concrete was well within the acceptance limits, these parameters still can not be considered good enough from the freeze-thaw resistance viewpoint.

In contrast, all specimens obtained from the construction Phase 2 exhibit excellent air-void system parameters (Table 5.5). It is evident that the void frequency was about three to four times higher, the specific surface area is approximately two to three times higher and the spacing factor is about three to four times lower than the respective values for specimens from construction Phase 1. When comparing the results for specimen 2P from lot 2, subplot 5 (Phase 1) and 2IN from lot 2, subplot 6 (Phase 2), it is clear that even though these specimens have almost identical hardened air content (7.5% and 7.4%, respectively), their air-void system parameters were very different. For instance, the spacing factor in the former case was 0.39 mm, or almost twice as high as the minimum spacing factor believed to be required to ensure freeze-thaw resistance (0.20 mm). On the other hand, the spacing factor of 2IN specimen was 0.13 mm, which is about half of the minimum recommended value. Therefore, it is apparent that the placement of the bridge deck concrete using a pump (Phase 1), rather than a conveyor belt or crane and bucket (Phase 2) had a negative impact not only on the air content but also on the parameters of the air void system.

5.5.1.2 Freeze-Thaw Resistance

In order to evaluate the effect of compromised air-void system parameters on freeze-thaw (F-T) resistance of concrete placed during Phase 1 construction, several specimens were obtained and subjected to freeze-thaw test. However, since the reinforcing mat was present at the level of 76 mm (2.5 in.) from the top surface of the bridge deck, cutting the standard freeze-thaw specimens (76×102×406 mm) was not feasible. Instead, the 22 mm-thick cylindrical specimens were obtained from previously cored 95 mm cylinders. The specimens were obtained from lot 1, sublots 1 and 6 (representing the sublots with the lowest fresh concrete air content) and lot 2, subplot 3 (two specimens) and subplot 5. Prior to testing, the specimens were resaturated in lime-saturated water for 48 hours. The specimens were subjected to freeze-thaw test as per AASHTO T 161 (procedure A). The relative dynamic modulus was monitored approximately every 36 cycles up to 300 cycles through monitoring the resonant frequency obtained using the impulse excitation method following the procedure described in the special annex pertaining to testing of disk-

shaped specimens in ASTM C 1259 (ASTM 2008d). In addition, the mass change was also recorded.

The freeze-thaw test results are shown in Fig. 5.11. It can be seen that the relative dynamic modulus could be obtained throughout the entire period of 300 cycles only for one of the specimens from lot 2, subplot 3 (Fig. 5.11(a)). The resulting durability factor for that specimen is 85%.

Fig. 5.11 – Results of freeze-thaw test: (a) Relative dynamic modulus of elasticity and (b) mass loss vs. number of freeze-thaw cycles

The resonant frequency measurements for other disks were not possible after 133 F-T cycles due to insufficient stiffness (the samples were, however, kept in the F-T machine until about 300 cycles were completed). By extrapolation of the curves obtained until 133 cycles (Fig. 5.11), it is evident that the relative elastic modulus of the specimens from lot 1, subplot 1 and 6, as well as the specimen from lot 2, subplot 5, would have fallen well below 60% which is considered a threshold value for acceptance of concrete based on the freezing-thawing test. Moreover, this claim is enforced by the fact that both specimens from lot 2, subplot 3 experienced a negligible mass loss, while all three other specimens (particularly from lot 1, subplot 1) underwent a significant mass loss as shown in Fig. 5.11(b).

The extent of damage of the specimens after 300 freeze-thaw cycles is visible in Fig. 5.12. Clearly, only two specimens from lot 2, subplot 3 retained their original shape. All other specimens were broken into smaller pieces. The evidently better performance in freeze-thaw test of the specimens from lot 2, subplot 3 can be attributed to significantly lower spacing factor of concrete from that location compared with the spacing factors of concrete from all three other sublots (Table 5.5). Therefore, the above results further emphasize the necessity for adequate quality of air-void system in order to ensure freeze-thaw resistance of concrete.

Fig. 5.12 – Extent of damage of freeze-thaw specimens after 300 cycles

5.5.1.3 Resistance to Chloride-Ion Penetration

As a part of the failed material investigation, several cores were obtained to determine the overall quality of concrete. The AASHTO T 277 (AASHTO 2006f) rapid chloride permeability test was conducted on the specimens obtained from the cores extracted from the bridge deck 150 and 303 days after construction. As seen from the results presented in Table 5.6, a very dramatic reduction in the RCP values took place between 150 and

303 days. This reduction is attributable to additional hydration of the ternary mixture used in the bridge deck that took place during the summer following the construction.

Table 5.6 – RCP results (coulombs) obtained for the cores from BDC-1

Testing age (days)	Lot 1, Sublot 1	Lot 1, Sublot 6	Lot 2, Sublot 3	Lot 2, Sublot 5	Average
150	2642	-	2733	-	2688
303	478	493	591	538	525

5.5.1.4 Chloride Concentration Profiles

Two cores were extracted from Phase 1 deck after the first winter (about 6 months after placement) and used for determination of chloride concentration profiles. The first core was retrieved from the driving lane and the second core was extracted from the gutter zone located along the temporary concrete barrier separating Phase 1 and Phase 2 decks. The water-soluble chloride contents obtained from these specimens (BDC-1 (Lane) and BDC-1 (Gutter)) is shown in Fig. 5.13.

Fig. 5.13 – Chloride concentration profiles (Fort Wayne data from Olek et al. (2003))

The relatively high level of chloride near the surface, that occurred after one winter season only, is attributed to relatively high permeability of the ternary concrete mixture which was exposed to deicing salt only 21 days after construction. The average ambient temperature during that period was about 7°C (Fig. 5.3). For comparative purposes, also shown in Fig. 5.13 are the data obtained for other bridge decks also located in the northern area of Indiana, near Fort Wayne (Olek et al. 2003). OPC concrete of $w/c = 0.45$ with no mineral admixtures was used for these structures. Chloride profiles were obtained from those bridge decks at 3, 4, 5, 7 and 9 years. Noteworthy is the fact that despite high surface chloride content, the chloride concentration at lower levels in the ternary concrete (BDC-1 Lane and BDC-1 Gutter) remained very low.

For comparative purposes, diffusion coefficient and surface concentration were determined for the analyzed chloride profiles (Table 5.7) using Fick's second law and the least squares method (ASTM 2004a). The effects of other phenomena affecting the chloride transport, such as, wicking action, sorption, and permeation have been omitted in the analysis. Although the results indicate that the diffusion coefficients were higher in the relatively young ternary concrete, one can reasonably expect that they will become substantially lower upon further hydration.

Table 5.7 – Diffusion coefficient and surface concentration values for various bridge decks

	Bridge Deck Location						
	BDC-1 - Lane	BDC-1 - Gutter	Fort Wayne (IN) Area				
Exposure Time (years)	0.34		3	4	5	7	9
C_s (kg/m ³)	16.50	12.33	10.57	12.33	11.53	11.21	13.78
D_a (10 ⁻¹² m ² /s)	3.766	5.274	3.607	2.096	2.089	2.941	1.579

5.5.1.5 Conclusions

The following conclusions have been drawn on the basis of the failed material investigation conducted in cooperation with INDOT:

- The air-void system parameters of concrete samples from lot 1 have been considered substandard. This resulted in unsatisfactory freeze-thaw resistance of lot 1 concrete, compared to much better freeze-thaw resistance of lot 2 concrete which also possessed satisfactory air-void system parameters.
- While scaling observed during the visual inspection of Phase 1 deck was likely mainly caused by low curing temperature and relatively early-age exposure to deicing salts, it might have been further aggravated by poor air-void system parameters found in the Phase 1 deck concrete.
- Despite low early-age curing temperature and early-age exposure to deicing salts, the investigated concrete achieved very high resistance to chloride-ion penetration. It is expected that due to reduction of permeability upon further hydration (as demonstrated by low RCP values shown in Table 5.6), the risk of further salt scaling or F-T damage resulting from poor air-void parameters should remain low. As such, adequate long-term durability of the deck is anticipated.

5.5.2 Laboratory Investigation into Air Loss due to Pumping

5.5.2.1 Introduction

From the results presented in the preceding section, it was concluded that pump placement of concrete during Phase 1 construction had a very adverse effect on the quality of air-void system and resulting freeze-thaw resistance during the first few months after construction. Therefore, it was desirable to determine the possible causes of the dramatic air loss experienced by BDC-1 concrete. To do so, an extensive literature review was performed and a laboratory investigation on the stability of entrained air was also undertaken.

5.5.2.2 Literature Review

Nowadays, more and more improvements are made to the modern pumps in order to enhance the pumping process. However, considering the fact that very often some extreme operation parameters, comprising typically of both high pressure and long pump lines, are selected by pump operators in order to increase efficiency, it can be imagined

that some detrimental effects on air stability in concrete might be expected. There are many different factors and mechanisms involved in the phenomenon of air loss due to pumping of concrete. The three main mechanisms believed to take place when concrete is pumped and air loss problems are experienced are discussed below.

Drop of concrete from a certain height: it was found that an impact of falling concrete essentially leads to air bubbles collapse (Nelson et al. 2002, NRMCA 1992, Sellers 2002, Scholer and Grossman 1998, Yingling et al. 1992). Two cases should be distinguished at this point.

The first one occurs when weight of concrete, in conjunction with its loose consistency and low viscosity, is sufficient to overcome frictional resistance and results in a slug of concrete sliding uncontrollably the pipe. This results in an accelerated rate of fall of concrete through the line and it may result in larger amount of air being lost when concrete drops down. Secondly, though not less importantly, a vacuum can be created at the points of discontinuity of concrete in a line and it may make the air bubbles bigger and more prone to bursting as concrete hits the horizontal surface. Accordingly, the height of fall is determined by the length of vertical or near-vertical downward pipe (from the end of the boom to the bottom of the form, reinforcing bars, or o wall of an elbow if one is used).

The second case takes places when concrete is being held continuously in a pump line and free fall through the entire height of a vertical section is not allowed. Obviously, concrete falls freely once it is released by the rubber hose and the distance from its end to the bottom of the form has been also found to be a key factor. As a matter of fact, 20-40 % of air loss was reported to occur due to concrete impact (Scholer and Grossman 1998).

High pressure in a pump line: it leads to air bubbles bursting and therefore air content loss (Nelson et al. 2002, Scholer and Grossman 1998). High pressure forces air bubbles to become smaller and (when concrete is released by the hose) such bubbles do not entirely recover (Scholer and Grossman 1998). Loss of air is attributed in this case to dissolution of air bubbles. The dissolution is more likely to occur in the case of which affects smaller, rather than bigger, bubbles, as these have smaller surface area.

Dissolution was found to take place in the very first seconds after the concrete is subjected to pumping (Scholer and Grossman 1998).

Bursting of air voids caused by vacuum: this is likely to occur preferentially in the case of highly flowable or very stiff concrete mixes which develop discontinuities when pumped through the pump line (Scholer and Grossman 1998). Therefore, vacuum is created in the places of discontinuity which makes the air bubbles to be sucked out of the system. Accordingly, the vacuum causes great expansion of the bubbles and subsequently high stresses are created on their surface resulting in bursting. It should be noted that pressure method used for air content determination becomes increasingly less accurate as the bubbles become smaller (Hover and Phares 1996).

Typically, air content change is divided into two categories: changes due to the concrete composition and changes due to placement. The factors associated with mix composition are as follows:

Concrete consistency: highly flowable mixes, with a slump of over 150-180 mm, tend to lead to discontinuity occurring within a pipe. Furthermore, flowable mixes slide down the pipe easier. As a result, a vacuum, causing bursting the air bubbles, is created.

Cement characteristics: high cement content was found to have adverse influence on air content stability (NRMCA 1992). Moreover, variability in quality and properties of cement, including high alkali content, fineness and content of contaminants may also contribute to this phenomenon (Nelson et al. 2002, Whiting and Stark 1983).

Presence of SCM's: supplementary cementations materials such as silica fume, fly ash or blast furnace slag are known to cause destabilization of air void system in fresh concrete mix, even when it is not subjected to pumping (Khayat and Assaad 2002, Du and Folliard 2005). High air loss during pumping may be encountered especially in cases of high levels of replacement of cement by e.g. silica fume (Holley et al. 1999) or fly ash (Whiting and Stark 1983).

Presence of chemical admixtures: High range water reducers (HRWR), especially in very high dosages (very flowable, self-compacting concrete) were found to destabilize entrained air-bubbles during transport (Khayat and Assaad 2002, Plante et al.

1989, Pleau et al. 1995, Whiting and Stark 1983). Due to the fact that such a mix is only slowly agitated, one could expect even much greater impact of the pumping process, where much more vigorous action due to high pressure and velocity takes place. Particularly, polycarboxylate-based superplasticizers are believed to entrain air which is in general not stable in its nature. It should be noted that air content is typically directly related to slump loss, may become an issue in highly flowable concretes. Moreover, splitting dosages of superplasticizer was reported to have adverse effect on stability of air void spacing factor in concrete which was not pumped. However, redoing of superplasticizer, which commonly takes place at the job sites, was perceived as a factor not influencing the stability of air void system (Plante et al. 1989). Other chemical admixtures affecting the air content are: set retarders, accelerators and obviously air entrainers. Noteworthy is the fact that the stability of entrained air (along with stability of slump) should be always verified by making trial mixes. If either slump or air content tend to be unstable, one of the chemical admixtures (air-entrainer or water reducer) may need to be replaced due to incompatibility.

Aggregate characteristics: total aggregate volume, fine-to-total aggregate ratio, gradation of coarse and fine aggregate (Singh et al. 2004), maximum size of aggregate and content of impurities all seem to influence the air content. Specifically, poor gradation of fine aggregate, especially gap-graded fine aggregate, was found to destabilize air-void system as the entrained air bubbles cannot be supported by fine aggregate particles within the system.

Water content: high content of water in concrete mixture typically causes increase in air content. However, this sometimes becomes an issue as the air present in such mixtures tends to be unstable. Therefore, it should also be noted that an amount of water added at the site or the amount of rain water also need to be taken into consideration (Nelson et al. 2002, Sellers 2002).

The factors associated with placement of concrete are as follows:

Pump configuration: boom length and angle of inclination (the steeper the boom the more air is lost (Hover and Phares 1996, Sellers 2002)), pump set-up (A-frame, Z-type) and length of a vertical sections, diameter of a rubber hose, use of any devices

allowing reduction of air loss, pump size, number of pump segments, method of loading of the concrete into the pump hopper (Scholer and Grossman 1998, Sellers 2002).

Weather conditions: extreme temperatures, both low and high, lead to general instability of entrained air. Furthermore, rain will result in an increased water content and hence also increased air content in a mix.

Some individual cases of air loss and general remarks are described in the following sections of this report.

The study performed by the Washington Aggregates and Concrete Association (1991) indicated that when there was a negative pressure in the line, or when concrete was let to fall freely, air content loss could be observed. Whereas when pump line was kept full (which could be attributed to less flowable mixes), there was a little change, or even slight increase, in the air content.

Some other important studies clearly revealed that air-void system parameters are also greatly altered by pump setup (Lessard et al. 1996). Two vertical setups (of which one had a reduced vertical section) and one horizontal setup were incorporated in the described work. It was shown that, despite some air content loss, the horizontal pumping resulted in no change in spacing factor and specific surface area. Nevertheless, when concrete was pumped through vertical pipes without reduced section, obtaining of recommended values of spacing factor ($L < 230 \mu\text{m}$) and specific surface ($S_s > 25 \text{ mm}^{-1}$) which would meet the requirements from a frost resistance standpoint, was not possible. However, no significant loss in freeze/thaw resistance of the resulting concretes was observed despite the alteration of air-void system parameters.

Another relevant aspect pointed out by many researchers is the fact that freeze/thaw durability does not necessarily need to be altered by change in air content of fresh concrete. This is because often bigger air bubbles, which have been found to contribute little to the improvement of freeze/thaw resistance anyway, are removed from the system and only smaller bubbles remain in place. Hover and Phares (1996) concluded that even significant losses in air content resulting in less than 4% of air in fresh concrete may not translate to losses in frost resistance.

The next issue, which is particularly of importance to concrete durability, is the actual size of air bubbles which are lost due to pumping. The ACI 304.2R report (ACI 1997) suggests that pumping leads to loss of bigger air bubbles only, as these are mostly affected by the impact caused by drop of concrete. Also, a vacuum may be created within a pump line would first burst bigger, rather than smaller and therefore stronger, air bubbles. Contradictory to this theory are the results obtained by Pleau et al. (1995) which clearly show that air loss comes from loss of smaller air bubbles. An explanation for that observation was related to creation of high pressure, which causes dissolution of smaller air bubbles as these have relatively small surface area ($100 \mu\text{m}^2$) and smaller initial volume. This may obviously have a detrimental effect on freeze/thaw resistance.

Extensive investigation of air loss due to pumping was also conducted by Yazdani et al. (2000). In total, 71 concrete mixes pumped on different job sites (bridge decks in particular) in Florida were involved in the study. An average air content loss reported by authors was 1% while maximum air loss was 3%. These numbers are in line with other investigations. However, it should be remembered that the composition of the concrete mixtures subjected to pumping was not known. Furthermore, results shown in the above mentioned reference (Yazdani et al. 2000) are presented only in terms of relative levels of change. As a result, it is not known in what ranges of air content and slump of the concrete mixes studied was.

Similar conclusions about the air content change due to pumping were drawn by Nelson et al. (2002). The average of 1% of air loss was found to be a typical value at 31 different tested sites. The following factors were reported to be the most influential: dropping distance, boom slope, boom length, amount of air added as an admixture and pump loading method. The other possible influential factors listed as possibly contributing to the air loss were: weather conditions, amount of water added on site and pump size. Noteworthy is the fact that authors did not record any negative influence of slump and water-cement ratio on the air content change. Similar conclusions with respect to lack of influence of w/c, as well as change in type of admixtures, were reported by Scholer and Grossman (1998).

A short article written by Gorsha (1992) appears to be contradicting the recent findings. The author emphasizes that high air content reduction is caused by an impact of free falling concrete as opposed to a vacuum in a line which, in author's mind, may not exist. The big concern, actually motivated this author to investigate an issue of air loss due to the pumping observed during a certain construction project. Specifically, a substantial amount of air content (9-12 %) at the end of the truck chute dropped to about 3.5 %. Various factors were considered to be causing such a dramatic change in air content, including the type of an air-entrainer and superplasticizer. Eventually, an angle of concrete pump boom was found to be a key factor. Accordingly, if the boom was elevated and the end pointed down, which happens to be the most commonly used pump setup these days, it was impossible to achieve the air content within an acceptable range. However, when the boom was positioned either flat out and straight or angled straight up, and when the line was laid flat out from the end of the boom, the target value of 6 % could then be obtained.

The difficulty in maintaining an average spacing factor lower than $230\ \mu\text{m}$ has led the Québec Ministry of Transportation in Canada to prohibit the placement of high-performance concrete with a pump on all high-performance concrete bridges built before 1996 (Aïtcin 1998).

To prevent the air loss during pumping, several solutions are recommended:

- Limit the impact force: this can be achieved in number of ways, e.g. by inserting four 90 degree elbows just before rubber hose (Nelson et al. 2002, NRMCA 1992, Scholer and Grossman 1998, Sellers 2002, TDOT 2003, Yingling 1992) or using a "j" shaped configured section or loop attached to the discharge hose (Nelson et al. 2002, TDOT 2003). Another optional techniques are using a slide gate at the end of the rubber hose to restrict discharge to provide resistance (NRMCA 1992) or using a 1.5 m (6 ft.) diameter loop in the rubber hose with the extra section of rubber hose (NRMCA 1992) or laying 2.5 m (10 ft.) or 5.0 m (20 ft.) of hose horizontally on the deck pours which has been proven to be a great help in overcoming the problem (Nelson et al. 2002, NRMCA 1992, Yingling et al. 1992).

- Reduce the rubber hose size from 127 mm (5 in.) to 102 mm (4 in.): a transition pipe may be needed to avoid blockages (NRMCA 1992). The idea is to slow down the speed of concrete fall through a rubber hose. At the same time, the concrete is squeezed and hence prevented from becoming discontinuous.
- Use concrete of a slump within the range 50-150 mm (2-6 in.) (ACI 1997): both too flowable and too stiff mixes will lead to vacuum being generated within the pipe and therefore contribute to bursting of the air bubbles. Moreover, mixes which are too stiff require use of high pressure, thus contributing to air bubbles dissolution.
- Keep the pressure at the lowest possible level (preferably up to 3.5 MPa (500 psi)) (Scholer and Grossman 1998).
- Limit the distance of concrete drop up to 60 cm (2 feet) (Scholer and Grossman 1998).
- Keep the boom as close as possible to the horizontal position (Hover and Phares 1996, Yingling et al. 1992).
- Appropriately place the air release valve: if pumping downward 50 feet or more is required, the valve should be placed in the middle of the top bend of the pump line to prevent either vacuum or air build-up (Scholer and Grossman 1998).
- Avoid pumping in rain: pumping in rain will result in excessive water presence which, apart from increased air content, can lead to much more difficulties with finishing.

In order to verify the air content change due to pumping on the site, test the air content of concrete should be determined after pumping. The sample should be taken directly from the point of placement (e.g., bridge deck). Pumping concrete directly to the bucket or wheelbarrow should be avoided as it may result in obtaining non-representative sample due to change in boom position, pump configuration and pumping rate. Also, trial pumping (full-scale testing) is recommended to avoid potential problems at the site, either with a pump set-up or the concrete mix itself. If air loss is observed, one of the mitigating techniques mentioned above should be tried before adding more air entraining agent. This may result in an excessive amount of air in concrete whenever

the pump changes its position (boom angle, distance etc.) and other undesirable side effects, such as increased slump (Yingling et al. 1992), segregation etc.

It should be also realized that pumping does not always lead to reduction in the air content. In many cases, the air content remains the same or even increases (ACI 1997, Hover and Phares 1996). Noteworthy is the fact that even if the air content is unaffected by pumping or changes only slightly, specific surface and spacing factor may still be greatly affected. Thus, obtaining just a single number (air content) from either fresh or hardened concrete is of limited usefulness in terms of estimation of the performance of pumped concrete. An increase in the air content, in turn, can be caused by two factors: initial pressure in pump and adverse weather (rain). Furthermore, other methods of placing concrete, such as using buckets or direct chute from the truck may also result in air loss. However, in those cases the air loss is mostly controlled by the impact and thus it can be expected to be smaller than due to pumping.

5.5.2.3 Experimental Details

The main purpose of this study was to determine whether the ternary concrete used for construction of the bridge deck of interest was susceptible to air loss. Therefore, a series of mixtures with different initial air content and slump (adjusted by different dosages of air entraining agent and superplasticizer, respectively) were prepared. The composition of all mixtures was the same and the same materials as those used in the bridge deck construction. In addition, for comparative purpose, the class C INDOT concrete was prepared and examined. Thus, three variables included in this research were: initial air content (varied from 4.5% to 11.0%, slump (varied from 110 to 250 mm) and composition of concrete (ternary with 20% FA and 5% SF or class C concrete).

Based on the literature review presented in section 5.5.2.2 it was concluded that the main two mechanisms responsible for loss of entrained air during pumping are the impact of dropping concrete and bursting of air bubbles under vacuum. Accordingly, the air loss of concrete was simulated by two methods: (a) vacuum suction and (b) vertical drop.

The general order of measurements was as follows. Immediately after mixing, the measurements of the initial slump and air content were taken. Next, about 10 minutes after the end of mixing, the new portion of concrete was placed in two aluminum desiccators, each with an approximate volume of 5 L. The desiccators were covered with glass lids, sealed and subjected to vacuum suction for 150 seconds at a pressure of about 1 mm Hg). The same vacuum pump that is normally used for conditioning of specimens for AASHTO T 277 RCP test was employed (Fig. 5.14(a)). The concrete was then quickly removed from both desiccators, combined and its air content was measured. Subsequently, about 20 minutes after the end of mixing, the upper hopper of compacting factor apparatus (see Fig. 5.14(b)), was filled with a new portion (about 10 L) of concrete. The apparatus consists of two conical hoppers mounted above one another. The hoppers have hinged doors at the bottom. Since the bottom hopper was not used, its door was open throughout the duration of the tests. After the door of the top hopper was opened, the concrete was instantaneously dropped vertically from the distance of 95 cm. The concrete collected in the bucket was then tested for the air content.

Fig. 5.14 – Concrete subjected to (a) vacuum suction; (b) vertical drop

Since substantial amount of entrained air is being lost by concrete with time, especially during first 30 minutes after mixing, the appropriate correction for the air content needed to be made in order for the results to be meaningful. Accordingly, several mixtures with different air content were made and the air content loss was determined after 10 and 20 minutes. The results were incorporated into a statistical function, from which the air content at 10 minutes (when subjected to vacuum suction) and 20 minutes (when dropped) was estimated based on the initial air content.

5.5.2.4 Results and Discussion

The results from the vacuum suction experiment are presented in Fig. 5.15. It should be mentioned that two vacuum pumps were used in the study (“old” and “new”). This was because the “old” pump broke about half way through the experiment. Since it is evident from Fig. 5.15 that the new pump resulted in air loss which was by far greater than for “old” pump, the results from the two pumps will be analyzed separately. However, regardless of which pump was used, the results have been divided into two groups depending on the initial slump of the concrete tested. Accordingly, the “low slump” group represents the values of slump of about 130 mm, which is on the lower side of the range of slump recommended (INDOT 2004), whereas “high slump” stands for a group of highly flowable concretes with a slump higher than the recommended values (about 250 mm).

The results obtained using the “old” pump indicate a strong correlation between the amount of air lost, which can be as low as 0.8% and as high as 3.3%, and the initial air content (Fig. 5.15(a)). Accordingly, it seems that neither the initial slump nor the presence of the mineral admixtures influenced the amount of air lost.

Fig. 5.15 – Air loss due to vacuum suction vs. initial air content for (a) old pump and (b) new pump

The air loss obtained for the “high” slump mixtures using the “new” pump seem to be much greater than those for the “old” pump (compare Fig. 5.15(b) and (a)). Depending on the initial air content, the air loss was as low as 4.5% and as low as 7.5%. It is worth noting, that the equation for the trend line obtained for this group of mixtures suggests that the air content after vacuum suction is almost the same, i.e. 2.0-2.5%, irrespective of the initial air content. This means that probably all entrained air is being lost in the case of highly flowable mixtures subjected to negative pressure for extensive period of time. Interestingly, the magnitude of air loss of “low” slump mixtures was much lower (2-3%), and similar to that obtained using the “old” pump. Nevertheless, it is clear that the excessive slump can lead to dramatic air loss.

The air loss resulting from the vertical drop for is shown in Fig. 5.16. It can be seen that dropping of concrete from a height of nearly 1 m can result in air loss in the range of 0.5% to almost 2.0%. The amount of air lost appears to be a weak function of the initial air content, whereas statistical analysis showed that it is independent of slump.

Fig. 5.16 – Air loss due to vertical drop vs. initial air content

5.5.2.5 Conclusions

On the basis of the results of the experimental study aimed at simulating of loss of air due to pumping the following conclusions have been drawn:

- The presence of mineral admixtures in the ternary mixture appeared not to have adverse effect on air loss.
- Air loss due to vertical drop of concrete was independent of slump of concrete. It was also not found to be strongly correlated with the initial air content.
- Subjecting air-entrained concrete to vacuum suction may lead to substantial loss of air. The amount of air lost depends mostly on the initial air content, however for concrete with excessive slump this problem is aggravated.

5.6 Observations from Site Visits

5.6.1 Introduction

As previously discussed in section 5.5, two site visits on the bridge deck on SR 23 over SR 20 near South Bend, IN were conducted in March and April of 2005. The main purpose of those visits was to collect the cores from Phase 1 bridge deck for failed material investigation. Subsequent visit was conducted on Nov. 10th, 2006, the purpose

of which was to compare the extent of scaling and cracking (if presents) of all sections of the bridge deck, as shown in Fig. 5.17 and listed below:

- bridge deck – Phase 1 (constructed in Nov. 2004) (A)
- bridge deck – Phase 2 (constructed in May 2005) (B)
- approach slab – Phase 1 eastern side (constructed in Nov. 2004) (C)
- approach slab – Phase 2 eastern side (constructed in May 2005) (D)
- approach slab – Phase 1 western side (constructed in Nov. 2004) (E)
- approach slab – Phase 2 western side (constructed in May 2005) (F)

Fig. 5.17 – Schematic illustration of sections of the bridge deck inspected

Additional field inspection took place on Dec. 19, 2006 during which a detailed crack mapping was performed. The average crack widths were also measured using a handheld optical ocular at a magnification of 12×. The observations of the extent of scaling and cracking at each of the aforementioned locations of the structure (from the first site visit) as well as crack map (from the second inspection) are provided in the respective sections below.

5.6.2 Scaling

The inspection of the construction Phase 1 bridge deck undertaken over a year earlier revealed that significant amount of scaling occurred already after one winter season. The current site visit revealed that the extent of scaling is very similar, which implies that scaling probably seized to progress. The scaling (or possible combination of scaling and abrasion) manifested itself in several forms. Firstly, the edges of the tinning lines were

affected, such that particles of fine aggregate were clearly visible. This damage of tines appeared to be relatively uniform over the entire Phase 1 bridge deck but was not observed on either of the approach slabs (see Fig. 5.18) constructed using INDOT class C concrete.

Fig. 5.18 – Damaged edges of the tinning lines on Phase 1 bridge deck

Signs of scaling also existed in a form of long lines (about 50 cm wide) in few locations at the deck (Fig. 5.19). The lines were several meters long and it is not clear whether they occurred as a result of abrasion or potentially due to placement of temporary barrier accumulating salt. The construction Phase 2 bridge deck displayed superior performance (in terms of scaling) to that observed for Phase 1 bridge deck (see Fig. 5.20).

Fig. 5.19 – Longitudinally-oriented scaled/abraded sections of Phase 1 bridge deck

Fig. 5.20 – General appearance of Phase 2 bridge deck

5.6.3 Cracking

The approximate number of cracks (in both directions) at various location of the bridge deck is given in Table 5.8. The western approach slab poured during Phase 1 construction had no transverse cracks, while 2 longitudinal cracks spanning the entire slab length were present. The western approach slab constructed during Phase 2 had 1

transverse crack, while 5 longitudinal cracks were noticed. Only one of those cracks spanned the whole slab, whereas 3 others were almost as long as the slab.

Table 5.8 – Approximate number of cracks observed in various sections of the bridge deck

Section	Longitudinal Cracks	Transverse Cracks
Phase 1 Approach slab – West	2	0
Phase 2 Approach slab – West	5	1
Phase 1 Approach slab – East	5	3
Phase 2 Approach slab – East	2	2
Phase 1 – deck	7	0
Phase 2 – deck	30	41

The western approach slab poured during Phase 1 construction had no transverse cracks, while 2 longitudinal cracks spanning the entire slab length were present. The western approach slab constructed during phase 2 had 1 transverse crack, while 5 longitudinal cracks were noticed. The eastern approach slab poured during construction Phase 1 had 5 longitudinal cracks, of which only one spanned the whole slab. Also, 3 transverse cracks were observed, none of which spanned the entire slab width. Total of 2 longitudinal and 2 transverse cracks were observed on the eastern approach slab constructed during Phase 2. Only one longitudinal crack spanned the whole slab.

The section of bridge deck poured during Phase 1 construction had a total of 7 longitudinal cracks. 3 of those cracks were very short (< 0.5 m) (see for example Fig. 5.21), 3 were about 1-2 m long and one was very long (> 10 m long) and was running along the center girder close to the curb dividing the construction Phase 1 and 2 sections of the deck (Fig. 5.22). No transverse cracks were noticed in Phase 1 bridge deck.

The bridge deck constructed in Phase 2 developed much more extensive cracking compared to the Phase 1 deck. The total number of longitudinal cracks was 30, whereas the number of transverse cracks was 41. The longitudinal cracks varied in length from 50 cm to several meters. Vast majority of longitudinal cracks were observed in the vicinity of the approach slabs (on both ends of the deck), as shown in Fig. 5.23.

Fig. 5.21 – Example of short longitudinal crack in Phase 1 bridge deck

Fig. 5.22 – Long longitudinal crack running over the center girder on Phase 1 bridge deck

Transverse cracks occurred generally in two regions. Approximately 7 relatively short (about 2 m long) transverse cracks initiated at the connection of the Phase 1 and 2 decks and extended toward the eastern end of the deck (Fig. 5.24). They were

spaced at about 1 m interval. The second location of occurrence of the transverse cracks was observed from about a mid-span of the deck toward east. These cracks were also spaced at about 1 m or less. However, they were much longer than the ones mentioned above and spanned at least one driving lane. Two of them spanned both lanes, and 2 spanned the entire Phase 2 deck width (both lanes and shoulder).

Fig. 5.23 – Short longitudinal cracks on Phase 2 bridge deck

Fig. 5.24 – Example of transverse cracks on Phase 2 bridge deck

A detailed crack map from the second inspection of the bridge deck along with the average crack widths (for Phase 2 construction) is shown in Fig. 5.25. At the time of crack mapping Phase 1 bridge deck was 776 days old, whereas Phase 2 bridge deck was 397 days old. Comparing the crack patterns for the two phases, it is evident that the Phase 2 deck is more severely cracked than the Phase 1 deck. The cracking pattern for the Phase 2 deck suggests that the cracking occurred most likely as a result of shrinkage (as opposed to mechanical load-induced cracking). It should be noted that although, in principle, cracking of Phase 2 bridge deck seems more severe, most of the cracks are relatively tight (about 0.1-0.2 mm). Since for the crack width of about 0.1 mm chloride ingress occurs at relatively low rate (Ismail et al. 2008, Zhou et al. 2007), it may be inferred that the observed cracking may only slightly contribute to potential deterioration, i.e., increased risk of corrosion of reinforcement. Nevertheless, the observed differences in severity of cracking for Phase 1 and 2 bridge decks are somewhat perplexing and deserve more careful examination. The following four potential reasons have been identified:

- Improper curing. It is possible that curing of the Phase 2 deck was not adequate. For instance, curing may have been applied too late to prevent development of plastic shrinkage cracks. Further, duration of moist curing may not have been sufficient, thus possibly increasing the early-age drying shrinkage as well as compromising the drying shrinkage resistance of concrete.
- Use of different materials. As seen in Fig. 5.24, the color of Phase 2 concrete (to the right of the construction joint) is darker than that of Phase 1 (seen between the construction joint and the yellow line). This suggests that the composition (e.g., cement or silica fume content, water-binder ratio, etc.) of Phase 1 and 2 bridge deck concretes might have been different. However, since the properties of Phase 1 and 2 construction concrete were generally comparable (see section 5.3.2) the likelihood of compositional differences is limited. Rather, this would suggest that different cement source was used, which might have also resulted in different shrinkage (plastic, drying or thermal) cracking tendency.

Fig. 5.25 – Crack map of SR 23 bridge deck (Note: 0.1 indicates a width of 0.1 mm or less)

- Additional thermal stresses due to different ambient temperature during construction. The ambient temperature on the day of pour of Phase 1 deck (9/3/2006) was 1-8°C, whereas it was 13-21°C during the pour of Phase 2 deck (5/17/2005). Shown in Fig. 5.26 is the development of strain inside of Phase 1 bridge deck due to temperature variation. Temperature data are not available for Phase 2 deck.

Fig. 5.26 – Development of strain due to temperature variation in Phase 1 bridge deck
(from Frosch and Aldridge 2006)

It is clear that following the period of first 14 days (when shrinkage due to drying is dominant), the strain fluctuates with changes in temperature. Using the data shown in Fig. 5.26, , it was estimated that the change in strain due to variation in temperature is about $3 \mu\epsilon/^{\circ}\text{C}$. Taking into account the aforementioned differences in ambient temperature at the time of construction, it can be assumed that around the time of setting, the temperature of BDC-2 concrete was about 20-30°C higher than that of BDC-1 concrete. Accordingly, the maximum strain in Phase 2 bridge

deck generated upon the temperature drop during winter, along with the strain due to drying shrinkage (about $40 \mu\epsilon$), likely added up to about 100-130 $\mu\epsilon$. When compared with maximum strain of 70 $\mu\epsilon$ experienced by Phase 1 bridge deck (Fig. 5.26), it becomes evident that Phase 2 bridge deck was subjected to much higher thermal stresses due to higher ambient temperature during construction.

- Additional restraint of Phase 2 deck from Phase 1 deck at construction joint. It is visible in Fig. 5.25 that most of the transverse cracks initiated at the construction joint between Phases 1 and 2 (shown by a dash-dotted line) and propagated toward either lane or even a shoulder. Therefore, it appears that bonding of concrete at the joint as well as the reinforcing bars extending from Phase 1 into Phase 2 deck (shown in Fig. 5.27) provided additional restraint (apart from that from underlying concrete girders and metal stay-in-place forms) of Phase 2 bridge deck. This additional restraint likely lead to higher tensile stress development in Phase 2 deck as a result of drying shrinkage and differential thermal shrinkage (from heat of hydration). Most likely majority of both the drying and the thermal shrinkage occurred relatively soon after (within first 2-3 months) construction of Phase 2 bridge deck. Incidentally, the additional restraint of Phase 2 deck may as well explain occurrence of longitudinal cracks at the ends of the deck, although these may be of minor significance.

While it is difficult, if not impossible, to determine which of the above were predominantly responsible for the construction Phase 2 deck cracking, it is rather evident that the last two issues most certainly contributed to it, whereas the first two causes are hypothetical in their nature. Nonetheless, it would be useful to extract cores from Phase 2 deck to determine if there are differences in chloride-ion concentration in the cracked concrete compared to uncracked areas.

Fig. 5.27 – Construction joint between Phase 1 and 2 bridge deck (the frame indicates reinforcing bars providing additional restraint of Phase 2 bridge deck)

5.6.4 Summary

The inspection of the bridge on SR 23 over SR 20 near South Bend, IN took place on Nov. 10, 2006. It was observed that although the surface of construction Phase 1 bridge deck was moderately scaled, the extent of scaling did not appear to change since the last inspection which took place over one year earlier. Very few longitudinal cracks and no transverse cracks were observed on that section of the deck. The Phase 2 bridge deck revealed exactly opposite performance. The surface appeared to be in excellent condition in terms of scaling, but numerous cracks in both longitudinal and transverse directions were present. This cracking was mostly attributed to development of stresses due to thermal strains combined with additional restraint at the construction joint induced by Phase 1 deck. Coincidentally, the performance of both approach slabs made out of class C concrete constructed individually shortly (within a week) after the construction of each phase appeared to be similar to that of Phase 2 bridge deck. As such, almost no scaling was observed but the slabs developed significant amount of cracks.

5.7 Predicting Resistance of Concrete to Chloride-Ion Penetration using Maturity Method

5.7.1 Introduction

Chloride-induced corrosion of reinforcement has been frequently reported to be the main cause of premature deterioration of concrete bridge decks (Hooton et al. 2001, Russell 2004). As a result, each year an enormous budget is spent for highway repairs and maintenance (Kirkpatrick et al. 2002, Liu and Weyers 1998, Tanesi and Meininger 2007). One of the common methods to offset the problem of reinforcement corrosion, and thus to increase the service life of concrete structures, is by incorporation of supplementary cementitious materials, such as fly ash (FA), silica fume (SF) or slag (Aïtcin 1998). Despite the undeniable overall improvement of durability (particularly in terms of increase in resistance to chloride-ion penetration) of concrete containing SCMs, fly ash and slag typically exhibit relatively low reactivity (Neville 1995).

It follows that at locations where low seasonal temperatures can occur, construction date may need to be considered in order to ensure that the concrete achieves a desired level of performance by the time of first exposure to deicing salts. Application of salts on immature (in terms of durability) concrete can lead to increased penetration of chloride ions and surface scaling. In fact, the evident difference in scaling resistance of ternary concrete containing FA and SF observed on the bridge deck on SR 23 over SR 20 constructed in two phases (Phase 1 in fall of 2004 and Phase 2 in spring of 2005) suggests that the placement time plays a significant role in the performance of the ternary OPC/FA/SF concrete (see Fig. 5.28). Furthermore, high coulomb values of Phase 1 bridge deck concrete and low scaling resistance (Fig. 5.28(a)) compared with low RCP results and very good scaling resistance exhibited by the Phase 2 concrete (Fig. 5.28(b)) suggest that RCP test results may potentially serve as a good indicator of scaling resistance of concrete. Indeed, based on the results of laboratory studies discussed in sections 4.3 and 4.4, there appears to be a strong linear relationship between RCP and scaling within each group of specimens subjected to a certain initial curing condition (Fig. 5.29). Also, for the most common field curing conditions (3 or 7 days of curing under wet burlap), the acceptance scaling limit of 0.80 kg/m^2 (as per recommendation by the

Ontario Ministry of Transportation (MTO 1989)) implies the maximum coulomb value of about 1500.

Fig. 5.28 – General appearance of SR 23 bridge deck: (a) scaled areas on BDC-1; (b) no signs of scaling on BDC-2

Fig. 5.29 – RCP vs. cumulative mass loss due to salt scaling after 50 cycles (for laboratory OPC/FA/SF concrete)

Therefore, the ability to predict RCP value of concrete at a given age appears very desirable, as it may potentially serve as a good indicator of its overall durability-related performance. Recently, Bentz (2007) introduced a “Virtual Rapid Chloride Permeability Test” allowing for prediction of RCP test values based on the composition of concrete mixture and its individual constituents. Although in general this virtual method can produce RCP values that are in a reasonable agreement with the measured ones, its main drawback is that it relies on the assumed degree of hydration (DOH), rather than on actual time and temperature history. Since it is difficult to accurately estimate DOH based on time and field temperature history (even though these are often readily available), the applicability of this virtual test may be limited in typical field applications.

The primary aim of this part of the research was to investigate whether low ambient temperature, which was experienced by Phase 1 bridge deck concrete under

investigation (BCC-1), could have had an unexpectedly adverse impact on the development of resistance to chloride-ion penetration of this ternary concrete. To accomplish that, the maturity-based procedure which allows for estimation of resistance to chloride-ion penetration of concrete (using AASHTO T 277 (2006f) rapid chloride permeability test) while taking into account the temperature history of concrete was developed.

The second objective of this study was to assess the effect of construction date (implying certain temperature history of concrete) on the magnitude of RCP value at the predefined time of first salt exposure. This information has been used to quantify the reduction in time to corrosion initiation (as a function of the construction date) and to provide recommendations regarding the calendar date restrictions as to the use of ternary concrete in bridge decks in order to minimize the risk of surface scaling.

The final goal of this study was to demonstrate another potential application of the developed maturity method for quality control purposes. Specifically, it will be demonstrated that an accelerated curing could be applied to concrete to obtain an estimation of its late age (typically 56-day) resistance to chloride-ion penetration in a reasonably short amount of time. This should result in considerable reduction in time needed for verification of compliance if a given concrete complies with the specification acceptance criteria.

5.7.2 Development of Maturity Method for Estimation of RCP Values

Since up to date no method for estimation of time- and temperature-dependent RCP values exists, a special procedure was developed in this study. The proposed procedure is broadly based on the ASTM C 1074 (ASTM 2006d) method for “Estimating Concrete Strength by Maturity Method” which was modified for the purposes of this study.

As a first step, the apparent activation energy was determined on the laboratory mixture from which a number of 102×203-mm cylinders were cast. Due to limited space in the environmental chambers used for long-term curing, all specimens were kept at 23°C for first 24 hours. Next, they were demolded and cut using a water-cooled diamond blade saw into 52 mm-thick disks. The disks were then split into three

groups, each of which was stored in lime-saturated water at different temperature (11.5°C, 21.5°C and 38.0°C).

At the pre-designated age the specimens were subjected to RCP test using ASTM C 1202 (ASTM 2006g) test procedure. The RCP tests were carried out at the age of 2, 3, 5, 7, 14, 21, 28, 90 and 180 days. With the exception of the second day (the first testing age) when three replicate specimens were used, the RCP test was performed on one disk per curing temperature. The second day was the earliest possible testing age due to 22-hours long conditioning required by ASTM C 1202 method. The test results are shown in Fig. 5.30.

The RCP test results obtained for each of three groups of specimens cured at different temperature were employed to determine the rate constant k . To do so, the following equation, being a modified version of the equation used for prediction of compressive strength using maturity method (ASTM 2006f), was incorporated:

$$RCP_t = RCP_0 - RCP_0 \frac{k(t - t_{ini})}{1 + k(t - t_{ini})} \quad \text{Eq. 5.3}$$

where RCP_t is the ASTM C 1202 measured resistance to chloride-ion penetration (coulombs) at age t , RCP_0 is initial RCP value obtained at an age t_{ini} (coulombs), k is rate constant, t is testing age (hours) and t_{ini} is the age of first test (here $t_{ini} = 48$ hours).

Using the non-linear regression analysis, the rate constant k was obtained for each of three curing temperatures from Eq. 5.4. Subsequently, to calculate the apparent activation energy, the following Arrhenius equation was employed:

$$k = Ae^{-\frac{E_a}{RT}} \quad \text{Eq. 5.4}$$

Accordingly, the natural logarithms of rate constant k plotted against the reciprocals of corresponding absolute curing temperature (shown in Fig. 5.31) give a linear relationship. The resulting negative slope of the line (8611 K) represents the value of apparent activation energy E_a divided by gas constant R (8.314 J/(mol·K)). Thus, the obtained apparent activation energy E_a is 71.6 kJ/mol. This value is almost twice as high as the apparent activation energy with respect to compressive strength (39.7 kJ/mol) obtained for a mortar of the same binder composition as the concrete studied in present

research (see section 4.5.3.1). This discrepancy may be explained by Joule effect associated with ASTM C 1202 test (Julio-Betancourt and Hooton 2004). The increase in temperature of concrete (especially at early ages) experienced during the test increases the apparent ion conductivity, and thus results leads to higher RCP values. As a result, k values derived from Eq. 5.3 are “artificially” lowered, which in turn increases the apparent activation energy E_a , as calculates using Eq. 5.4.

Fig. 5.30 – RCP_t results for laboratory determination of activation energy

Fig. 5.31 – Plot of natural logarithm of k values vs. inverse of absolute curing temperature

The next step involved development of a mathematical formula linking the predicted values of RCP with both the time and the curing temperature. This was done by first combining the time and the curing temperature into a term called “equivalent age”, and then linking RCP values with equivalent age. Regarding the first part, the formula based on Arrhenius equation (Carino and Lew 2001) was used:

$$t_e = \sum e^{-Q \left(\frac{1}{T_a} - \frac{1}{T_s} \right)} \Delta t \quad \text{Eq. 5.5}$$

where t_e is equivalent age at specified temperature T_s (days), Q is activation energy divided by the gas constant (K) (here 8611 K), T_a is average concrete temperature during

time interval Δt (K), T_s is specified temperature (K) (here 293.2 K) and Δt is time interval (h).

When deciding what type of mathematical function should be used to best represent the relationship between the coulomb values and the equivalent age, it was not only important that the resulting curve gives a high coefficient of determination (R^2) but preference was also given to a function containing the lowest possible number of constants. Several equations have been examined, and the following equation was found to give an excellent fit to the data (see the resulting master curve in Fig. 5.32) while having only two constants:

$$RCP_{pred} = \frac{I}{a + bt_e} \quad \text{Eq. 5.6}$$

where RCP_{pred} is predicted RCP value (coulombs), a and b are constants ($a = 7.077 \times 10^{-5}$; $b = 1.654 \times 10^{-5}$).

Fig. 5.32 – RCP_t vs. equivalent age

5.7.3 Laboratory Validation of Developed Maturity Method

The validity of the developed maturity method was verified on additional mixture with the same composition as that used for the development of the method (Table 5.2). The concrete was prepared in the laboratory and two series of cylindrical specimens were cast,

one of which was continuously cured under wet burlap at about 4°C, whereas the other one was continuously cured under wet burlap at about 23°C.

The comparison of the measured RCP values with those predicted using the maturity method at the ages of 7, 14, 28, 56 and 90 days is shown in Fig. 5.33. Considering relatively poor precision of the RCP test method itself (the difference between two individual results of 42% allowed by ASTM C 1202), it seems that the developed maturity method accurately predicts the RCP values regardless of the age and curing temperature.

Fig. 5.33 – Predicted vs. measured RCP for laboratory concrete (numbers in parentheses denote the testing age)

Fig. 5.34 – Predicted vs. measured RCP for field concrete (numbers in parentheses denote testing age)

5.7.4 Prediction of RCP Values for BDC-1 and BDC-2 Concretes

During both construction phases a number of 102×203-mm cylinders were cast for RCP test. All BDC-1 specimens were field exposed (including initial curing under wet burlap for first 7 days) until the 35th day, at which time they were split into three groups. The first group of specimens remained exposed to in-situ environment, the second group was moved to the laboratory (23°C and 50% RH) and the last group was moved to the moist room (23°C and 100% RH). The BDC-2 specimens were divided into two groups from

the very beginning. The first group underwent a standard continuous moist curing (at 23°C) and the other group was subjected to the field exposure after 7 days of curing under wet burlap.

The previously described procedure was used to predict RCP values for construction Phase 1 and 2 bridge deck concretes (BDC-1 and BDC-2, respectively) by plotting the predicted RCP values were plotted against the measured ones (Fig. 5.34). It can be seen that, while the agreement between the predicted and measured coulomb values for BDC-2 concrete (either continuously moist cured under laboratory conditions or field exposed) was satisfactory, the RCP values for BDC-1 were in all cases underestimated.

Fig. 5.34 also shows the prediction of 56-day and 90-day RCP for BDC-1 specimens moved at 35th day from outdoor exposure to either a moist room or a laboratory. It can be seen that, compared to field exposed concrete, the prediction of 56-day RCP was in both cases much improved. However, when tested at 90 days, the prediction of moist cured specimens remained fairly accurate, whereas that of specimens moved to the laboratory environment was unsatisfactory.

A thorough analysis was carried out in order to identify the possible reasons for relatively poor accuracy of prediction for field exposed BDC-1 concrete. The first part of the analysis involved determination of the effect of duration of wet burlap curing (3, 7, 14 days or continuous until testing) on RCP test results for the same ternary concrete. In an attempt to simulate the in-situ temperature fluctuations, the specimens were cured alternately (every 24 hours) at 23°C and 4°C. The results shown in Fig. 5.35 indicate that the effect of curing was not significant at 7 and 14 days. On the contrary, the 28-day and 56-day RCP test results of continuously cured specimens were lower by 1000 coulombs than those for 3-day burlap curing and by about 500 coulombs than those for 7-days burlap curing. These differences can be explained by lower rate of hydration due to limited water supply past the moist curing period.

Fig. 5.35 – Effect of moist curing duration on RCP

Fig. 5.36 – Effect of w/cm on RCP

While the above results confirm the importance of moisture availability on measured RCP values (and thus on the accuracy of predicted values), the extent to which 7-day burlap curing (as opposed to continuous moist curing) could affect the RCP test results appears limited. Therefore, in the second part of analysis, the effect of w/cm on RCP test results was examined. Accordingly, three mixtures with w/cm of 0.37, 0.40 (target for BDC concrete) and 0.43, respectively, were made and cured continuously under wet burlap while alternating the temperature between 23°C and 4°C every 24 hours. As seen in Fig. 5.36, w/cm had quite pronounced effects on RCP test results at 7 and 14 days but less significant at 28 and 56 days, which is the opposite of what was observed in the case of effect of curing duration (Fig. 5.35). Specifically, the increase in w/cm from 0.40 to 0.43 resulted in an increase in coulomb values of about 2000 at 7 days and approximately 1500 at 14 days.

From the above analysis, it appears that both moist curing for a limited period as well as slightly higher w/cm (e.g. 0.43 instead of 0.40) could have resulted in less accurate prediction of RCP values for BDC-1 concrete. It is believed that the accuracy of the developed method could be further improved by incorporating a term which would account for relative humidity to which the concrete was exposed during the

pertinent monitoring period. Such modification to standard ASTM C 1074 maturity method was recently successfully implemented with respect to compressive strength prediction (Liao et al. 2008). Nonetheless, this analysis not only helped explain poor predictive capability of the developed maturity method for BDC-1, but it also gave a useful insight into sensitivity of the developed method resulting from sensitivity of RCP test itself.

5.7.5 Quantification of Reduction in Time to Corrosion Initiation and Assessment of Risk of Scaling as a Function of Construction Date for Pilot Bridge Deck on SR 23

The developed maturity method was further utilized to determine by how much the time to corrosion initiation was reduced due to fall bridge deck construction (BDC-1) compared to spring construction (BDC-2). Several assumptions were made in the analysis:

- Chloride concentration threshold for corrosion initiation is 1.20 kg/m^3 .
- Surface concentration of chlorides is constant and equals $C_s = 12.88 \text{ kg/m}^3$. Selection of this value was driven by two reasons. Similar concentration was found at the surface of the core obtained from BDC-1 149 days after construction (Table 5.7). Additionally, this value falls in the range of $10.52\text{--}13.75 \text{ kg/m}^3$ determined at different ages (3 to 9 years) for a different bridge near Fort Wayne, IN (Olek et al. 2003).
- Apparent diffusion coefficient D_a is linearly correlated with RCP values through the following formula derived on the basis of the independent laboratory study (see Fig. 5.7(b)):

$$D_a = 9.70 \cdot 10^{-16} \cdot RCP + 7.16 \cdot 10^{-13} \quad \text{Eq. 5.7}$$

- To account for the differences in the actual (measured) and predicted RCP values resulting from non-continuous moist curing (Fig. 5.35) and potentially higher w/cm of BDC-1 concrete (Fig. 5.36), the following relationship (see Fig. 5.37) was used to correct the predicted RCP values for BDC-1 concrete:

$$RCP = 4.059 \cdot RCP_{pred}^{0.861} \quad \text{Eq. 5.8}$$

where RCP is actual (corrected) RCP value and RCP_{pred} is RCP value predicted on the basis of equivalent age from Eq. 5.6.

- Ultimate apparent diffusion coefficient D_{ult} of $1.23 \cdot 10^{-12}$ m²/s corresponding to 505 coulombs obtained for the core retrieved from Phase 1 bridge deck at 303 days was assumed. The age coefficient “m” was not included in the analysis due to its questionable validity (Luping 2007).
- The first salt exposure was assumed on Nov. 24, which is the actual date for BDC-1 concrete.
- The analysis carried out for BDC-2 was based on the temperature history from 2004, which implies the assumption of identical temperature history in 2004 and 2005.

As a first step of analysis, the time to corrosion initiation T_i for BDC-2 was determined. The Crank’s solution for the Fick’s second law was used:

$$C(x, t) = C_s - (C_s - C_0) \cdot \operatorname{erf}\left(\frac{x}{\sqrt{4D_a t}}\right) \quad \text{Eq. 5.9}$$

where $C(x, t)$ is time- and depth-dependent chloride concentration (here $C(x, t)$ is a chloride concentration threshold of 1.20 kg/m² at the reinforcement level), C_s is surface concentration (12.88 kg/m²), C_0 is background concentration (0.25 kg/m²), x is concrete cover thickness (0.065 m) and D_a is apparent diffusion coefficient equal to D_{ult} ($1.23 \cdot 10^{-12}$ m²/s).

By substituting the above values for corresponding symbols in Eq. 5.9, the time t , which represents time to corrosion initiation T_i , was found to be equal 17.2 years. Subsequently, the time to corrosion initiation T_i was determined for BDC-1 case. This was done by introducing a modification to Eq. 5.9, as follows:

$$C(x, t) = C_s - (C_s - C_0) \cdot \operatorname{erf}\left(\frac{x}{\sqrt{4(Z_1 + Z_2)}}\right) \quad \text{Eq. 5.10}$$

where Z_1 has a form:

$$Z_1 = \int_{t_0}^{t_{ult}} D_t dt \quad t_0 \leq t \leq t_{ult} \quad \text{Eq. 5.11}$$

whereas Z_2 is the original term:

$$Z_2 = D_a \cdot t \quad t > t_{ult} \quad \text{Eq. 5.12}$$

where t_0 is time of first exposure to salt (here 0.058 years = 21 days), t_{ult} is time at which concrete achieves its ultimate diffusion coefficient D_{ult} (here 0.80 years = 303 days), t is time (years), D_t is a time-dependent apparent diffusion coefficient (m^2/s) and D_a is equal to D_{ult} ($1.23 \cdot 10^{-12} \text{ m}^2/\text{s}$).

This approach leads to variation of diffusion coefficient with time between the age of first exposure to deicing salts and the age when concrete was assumed to have achieved its ultimate diffusion coefficient (303 days for BDC-1 case). By combining Eq. 5.6, Eq. 5.7 and Eq. 5.8, the final formula used for calculation of the time-dependent diffusion coefficient for BDC-1 takes the following form:

$$D_t = c \cdot \left[e \left(\frac{1}{a + b \cdot t_e} \right)^f \right] + d \quad \text{Eq. 5.13}$$

where t_e is equivalent age (days), a through f are constants ($a = 7.077 \times 10^{-5}$; $b = 1.654 \times 10^{-5}$; $c = 9.70 \cdot 10^{-16}$; $d = 7.16 \cdot 10^{-13}$; $e = 4.059$ and $f = 0.861$).

The calculated time to corrosion initiation for BDC-1 was found to be equal $T_i = 15.9$ years. Thus, the late age construction led to a theoretical reduction in service life by 1.3 years. In spite of the fact that this reduction in service life of the bridge deck can be considered relatively insignificant, it appears desirable to establish how the construction date may affect the RCP value at the time of first salt exposure and the associated reduction in time to corrosion initiation. As stated previously, the latter is believed to provide a good indicator of the overall quality of microstructure and the actual performance of concrete, i.e. scaling resistance. A period from April 14 to November 14 was considered as a potential construction season and the temperature history from 2004 was used for the purpose of this analysis. The RCP values were calculated on the basis of equivalent age t_e using Eq. 5.6 and Eq. 5.8. The reduction in time to corrosion initiation ΔT_i was computed by subtracting a resulting time to corrosion initiation (computed using Eq. 5.10 through Eq. 5.13) from 17.2 (T_i for BDC-2 case). It should be noted that different construction age (being the main input in this analysis)

implied certain temperature history which in turn affected three variables, namely, t_e , t_{ult} and D_r . The results of these simulations are presented Fig. 5.38.

It can be seen that in the worst case scenario of the construction taking place on November 14, the reduction in service life is 1.45 years, which is again considered rather insignificant. However, at the first time of salt exposure the RCP of that concrete would be nearly 9000 coulombs which is certainly unacceptable from the scaling resistance standpoint. For this particular case of concrete and temperature history, it appears that in order for the concrete to drop the coulomb values to 1500 until the salt exposure, the bridge deck would have to be constructed by the end of August (Fig. 5.38). This limitation is likely too rigorous. However, one can take an advantage of such information and in order to achieve the desired RCP value by the time of first salt exposure, extended curing period and/or heating blankets may be applied.

Fig. 5.37 – Relationship between actual and measured RCP for BDC-1 concrete

Fig. 5.38 – Reduction in time to corrosion initiation (ΔT_i) and RCP at first salt exposure as a function of construction date

While this analysis was carried out for the specific case of bridge deck of interest, similar analysis can be performed for any other location by employing certain temperature history and an expected time of first exposure to deicing salt (equivalent to

time of first snowfall). These data can be gathered from the local meteorological stations and average values can be efficiently incorporated for the purpose of this kind of analysis. It should be noticed that the time to corrosion initiation may be much more important for other concrete mixtures which are more permeable and more sensitive to curing temperature than the ternary concrete studied herein.

5.7.6 Estimation of RCP Values for Concrete Subjected to Accelerated Curing

One of widely recognized downsides of the usage of RCP test for quality control purposes is that often 56 days is required to verify the compliance of a given concrete with an applicable specification. The age of 56 days is typically specified to account for relatively low reactivity of some SCMs, such as fly ash or slag. However, extending the quality control process to 56 days is frequently not convenient and thus shortening that period seems to be very desirable.

To partially address this issue, Virginia Transportation Research Council (VTRC) has been widely using an accelerated curing procedure consisting of 7 day moist curing at 23°C followed by 21 moist curing at 38°C (Ozyildirim 1998). The RCP test results obtained from the above procedure found to correlate well with RCP test results obtained after 6 to 12 months of standard curing at 23°C. While such reduction in curing period from 56 to 28 days would certainly improve current provisions, it is believed that that the required curing period could be shortened even further. Using the developed maturity method, it is possible to obtain the 56-day RCP value in a matter of about two weeks (or even less depending on the curing temperature).

To illustrate the discussed methodology, the 28-day and 56-day coulomb values of the ternary OPC/FA/SF mixture studied in this chapter cured at the temperature of 38°C rather than 21.5°C have been predicted. As seen in Table 5.9, increasing the curing temperature from 21.5°C to 38.0°C led to reduction in curing period from 28 to about 7 days and from 56 to about 13 days, while achieving the same equivalent age of 32.5 days and 65.0 days, respectively. It should be noted that this substantial reduction in curing period resulted in very accurate predictions at both ages (see Table 5.9).

Table 5.9 – RCP values for curing temperature of 21.5°C and 38.0°C based on equivalent age

Curing temperature (°C)	Real age (days)	Equivalent age (days)	RCP (coulombs)
21.5	28.0	32.5	1299
38.0	6.8		1326
21.5	56.0	65.0	853
38.0	12.7		951

To further verify the accuracy of the developed accelerate curing procedure, the 28-day and 56-day coulomb values were predicted for two other concrete mixtures based on results obtained for specimens cured at 38°C. It should be noted that the composition of these two mixtures was very different from that of the ternary concrete studied previously. One of the mixtures contained 16% fly ash and was used for the construction of another bridge deck (denoted in Table 3 as “Bridge #2 Mix”), whereas the other mixture was a plain cement INDOT class C (INDOT 2008) concrete and was used for the approach slab of the same bridge deck (denoted in Table 3 as “Approach Slab Mix”). The comparison presented in Table 5.10 indicates that, although the mixture composition of these two mixtures is dissimilar and different than that of the aforementioned ternary (BDC) concrete (which would intuitively suggest different activation energy), all predicted values were very accurate. This may imply that the developed method is applicable to a wide variety of concrete mixtures. However, further research needs to be conducted to verify it.

Table 5.10 – Comparison between the measured and predicted values of RCP results for Bridge #2 Mix and Approach Slab Mix

Mix	Curing Temperature (°C)	Testing Age (days)	RCP (coulombs)
Bridge #2 Mix	21.5	28.0	2079
	38.0	6.8	2172
	21.5	56.0	1544
	38.0	12.7	1370
Approach Slab Mix	21.5	28.0	4242
	38.0	6.8	4597
	21.5	56.0	3701
	38.0	12.7	3939

The described above procedure was presented only as an example and, in principal, any curing temperature may be used to accelerate hydration. For instance, if 50°C was used instead of 38°C, the estimates of 28-day and 56-day RCP would be obtained at only 3.1 and 5.2 days, respectively. However, curing at higher temperature may not be desirable since it may lead to microstructural changes (in terms of chemical composition of hydration products) and potential microcracking.

5.7.7 Conclusions

A procedure allowing for estimation of resistance to chloride-ion penetration of concrete (using ASTM C 1202 rapid chloride permeability test) was developed using the concept of maturity which takes into account the temperature history of concrete. The following conclusions have been drawn:

- The accuracy of prediction of RCP values for continuously moist cured laboratory ternary concrete was very good.
- Due to high sensitivity of RCP test method, the predictive capability of the maturity method depends greatly on the moist curing duration and the actual w/cm (compared with that of concrete used for development of maturity function).
- The accuracy of prediction of RCP values for field bridge deck concrete was satisfactory for the deck constructed in spring (BDC-2), whereas for the deck constructed in fall (BDC-1) the RCP values were consistently underestimated.
- Unexpectedly high coulomb values of BDC-1 concrete were likely the result of both, low ambient temperature and slightly higher w/cm, as compared to designed value.
- The developed method can be used to project resistance of field concrete to chloride ion penetration at the time of first exposure to deicing salts. This information could be used to estimate the risk of early age salt scaling and the reduction in time to corrosion initiation resulting from early application of deicing salts.
- The developed maturity method can be efficiently used for quality control purposes to estimate 28-day or 56-day RCP values at, respectively, about 7 and 13

days, by subjecting concrete to accelerated curing at an elevated temperature (i.e. 38°C).

CHAPTER 6 – SUMMARY AND CONCLUSIONS

6.1 Summary of Findings

The conclusions drawn from individual studies conducted within the scope of this research have been listed at the end of each respective sections of Chapters 3 through 5. The most important findings from this work are summarized below.

Selection of Optimum Content of Fly Ash and Silica Fume in the Ternary Mixture

Table 6.1 summarizes the relative performance of the mixtures studied with respect to several properties believed to be critical from the standpoint of bridge deck concrete applications. Using these data, the following conclusions could be drawn:

Both mixtures containing 20% fly ash outperformed the mixtures containing 30% fly ash (Table 6.1). Of the two mixtures with 20% fly ash, 20FA/7SF mixture possessed higher resistance to salt scaling and slightly better transport-related properties, but it was less resistant to shrinkage cracking than 20FA/5SF mixture. Therefore, since shrinkage cracking resistance is viewed as one of the most important properties of bridge deck concrete, 20FA/5SF mixture is believed to represent the optimum mixture. The third best performing ternary mixture is 30FA/5SF whose performance was, in general, only slightly better than that of 30FA/7SF mixture.

- Considering the transport properties, the standard INDOT class C mixture exhibited significantly worse performance than any of the ternary mixtures studied (Table 6.1). Although the compressive strength developed by that mixture was comparable to the strength of 30FA/5SF mixture, the resistance to shrinkage cracking displayed by class C concrete was significantly lower than that of the ternary mixtures studied. The only real advantage of utilizing INDOT class

C concrete is that it offers higher freeze-thaw resistance than OPC/FA/SF mixtures.

- The cost analysis showed that the recommended 20FA/5SF mixture is only slightly more expensive than INDOT class C mixture (126 \$/m³ vs. 123 \$/m³). However, the ternary mixture offers a range of much improved durability-related properties, which should lead to significantly increased service life, and thus to much lower maintenance and rehabilitation costs.

Table 6.1 – Comparison of performance of mixtures studied (Note: rank 1 = best)

Parameter	20FA/5SF	20FA/7SF	30FA/5SF	30FA/7SF	INDOT C mix
Compressive Strength	1	2	3	4	3
Resistance to Chloride-Ion Penetration (RCP)	1	1	2	2	4
Rate of Absorption	1	1	2	2	3
Scaling Resistance	2	1	3	4	n/a
Freeze-Thaw Resistance	3	3	4	4	1
Resistance to Shrinkage Cracking	1	2	3	4	4
Unit Cost* (\$/m ³)	126	136	128	139	123

Sensitivity to Curing Conditions

- The following order of quality of initial curing regime (from best to worst) was identified: 7 days wet burlap, 3 days wet burlap, 7 days curing compound, air drying (no curing). The exceptions to this order were scaling resistance, which was generally the highest for concrete cured using a curing compound and

* Unit costs of the mixture are the approximate costs including all materials and delivery, based on information obtained in 2005 from concrete supplier for bridge deck on SR 23 over SR 20 near South Bend, IN.

resistance to shrinkage cracking, which appeared to be higher for air drying than curing compound treatment.

- The mixtures with 30% FA were generally more sensitive to lack of curing than 20% FA mixtures with respect to compressive strength and resistance to chloride-ion penetration, but lower sensitivity with respect to shrinkage.

Applicability of Maturity Method for Prediction of Strength of Ternary Concrete

- The accuracy of maturity method used for estimation of compressive strength decreased with the decreasing curing quality in the following order: 7 days wet burlap, 3 days wet burlap, 7 days curing compound, air drying.
- For substandard curing, the accuracy of maturity method decreased with an increase in fly ash content (from 20% to 30%) and decrease in silica fume content (from 7% to 5%).
- Maturity method can be reliably used for in-situ compressive strength prediction of the ternary concrete up to 28 days, provided that concrete is moist cured for a minimum of 3 days.
- If the air content of field concrete is significantly different (by more than $\pm 0.5\%$) than that of concrete for which the maturity function was developed, a suitable correction factor should be implemented in the prediction equation.

Use of Maturity Method for Prediction of Resistance to Chloride-Ion Penetration

- The developed maturity method for estimation of resistance to chloride-ion penetration (using rapid chloride permeability test) appeared to be generally accurate. However, it was found to depend to a great extent on the duration of moist curing and on the relative value of the actual w/cm, as compared to w/cm of the concrete mixture used for the development of maturity function.
- The developed method can be efficiently used for quality control purposes, as it allows for estimation of 28-day and 56-day RCP values of concrete using accelerated curing (e.g., at 38°C) for either 7 or 13 days, respectively.

- For a given (known or projected) temperature history, the developed maturity method can be used to predict chloride-ion penetrability of field concrete at the assumed age of exposure to deicing salts. This information can be utilized to assess the risk of early age salt scaling and the reduction in time to corrosion initiation associated with early application of deicing salts.

Field Performance

- The overall performance of ternary concrete containing 20% FA and 5% SF in the pilot HPC bridge deck constructed in northern Indiana was considered to be satisfactory.
- The properties developed by the field concrete from both construction phases were comparable to those of concrete which was made and cured under laboratory conditions. The exception was a relatively low resistance to chloride-ion penetration of concrete from Phase 1 bridge deck (constructed in late fall) observed for prolonged period of time.
- Bridge deck constructed during Phase 1 underwent moderate surface scaling, which likely resulted from low ambient temperatures the concrete was exposed to shortly after placement and early application of deicing salts which took place within a month after construction.
- Bridge deck constructed during Phase 2 (in spring) experienced some transverse cracking, which was likely due to additional restraint at the construction joint between the new and existing (Phase 1) bridge deck and higher thermal stresses resulting from elevated ambient temperature.

6.2 Conclusions and Recommendations

The main purpose of this research was to evaluate potential use of ternary binder systems incorporating ordinary portland cement, fly ash and silica fume for bridge deck concrete. In general, it was concluded that OPC/FA/SF concrete is capable of developing a unique combination of properties, such as very low chloride-ion diffusivity and water absorptivity, good resistance to salt scaling and shrinkage cracking, satisfactory strength

and satisfactory resistance to freezing and thawing. However, the following issues are crucial to successful application of OPC/FA/SF concrete in bridge decks:

- Mixture composition: fly ash content should be 20%, silica fume content should not exceed 5% (by total mass of binder) and paste content should not exceed 24%.
- Curing conditions: wet burlap curing for a minimum of 3 days (at temperature of 23°C) is required. If the temperature is lower, wet burlap curing period needs to be extended.
- Calendar restrictions: In order to minimize scaling problem, calendar restrictions should be specified for the use of ternary concrete in bridge decks constructed in the late fall season. Alternatively, contractor should be required to extend the moist curing period to 14 days or to use heating blankets in order to maintain higher temperature essential for hydration of slower reacting supplementary cementitious materials.
- Construction practices: concrete should not be placed in bridge decks using a pump unless contractor can prove during the preconstruction trial pumping that placement of concrete using pump can be achieved without altering not only the air content but also the air-void system parameters. To minimize the propensity of ternary concrete to cracking, it is recommended that the time-span of two-phased projects be limited to minimum. Also, the restraint at the construction joint between the individual phases should be minimized. Finally, placement of concrete should take place in moderate temperatures to minimize the thermal shrinkage stresses.

6.3 Expected Benefits, Deliverables, Implementation and Cost Savings

This section identifies the project deliverables and provides recommendations regarding the implementation of the research findings. The details are presented in the subsections below:

Expected Deliverables and Benefits

The main deliverables of this project are in the form of mixture composition recommendation and construction practices guidelines shown below:

1. Mixture composition: fly ash content should be 20%, silica fume content should not exceed 5% (by total mass of binder) and paste content should not exceed 24%.
2. Curing conditions: wet burlap curing for a minimum of 3 days (at temperature of 23°C) is required. If the temperature is lower, wet burlap curing period needs to be extended.
3. Calendar restrictions: In order to minimize scaling problem, calendar restrictions should be specified for the use of ternary concrete in bridge decks constructed in the late fall season. Alternatively, contractor should be required to extend the moist curing period to 14 days or to use heating blankets in order to maintain higher temperature essential for hydration of slower reacting supplementary cementitious materials.
4. Construction practices: concrete should not be placed in bridge decks using a pump unless contractor can prove during the preconstruction trial pumping that placement of concrete using pump can be achieved without altering not only the air content but also the air-void system parameters. To minimize the propensity of ternary concrete to cracking, it is recommended that the time-span of two-phased projects be limited to minimum. Also, the restraint at the construction joint between the individual phases should be minimized. Finally, placement of concrete should take place in moderate temperatures to minimize the thermal shrinkage stresses.

While the unit cost of proposed ternary mixture was shown to be comparable to that of standard (INDOT class C mixture) bridge deck mixture, the enhanced durability of ternary concrete is expected to yield significantly increased service life of bridge decks. The long-term cost savings will be primarily associated with bridge deck less frequent maintenance, repair, rehabilitation and ultimate replacement.

Implementation

The findings from the study shall be implemented by adapting the proposed ternary mixture composition by the Indiana Department of Transportation in future construction of bridge decks in Indiana. This adaptation should be in the form of the specification that should be developed and approved for inclusion in the INDOT's book of Standard Specifications. The guidelines developed during this investigation with respect to curing methods, construction practices and construction calendar considerations will help to maximize the potential of ternary concrete for superior performance.

In addition, the research conducted as part of this study successfully verified (with certain limitations) applicability of maturity method for estimating compressive strength and resistance to chloride-ion penetration of the ternary concrete. Incorporating these methods in routine INDOT's quality assurance protocol during bridge deck construction will allow for monitoring the estimated the in-situ compressive strength and resistance to chloride-ion penetration of ternary concrete as a function of temperature history that the concrete has been exposed to after construction. At the same time, these methods offer a potential for reducing the number of specimens cast during bridge deck construction that would be typically required for monitoring the development of these properties by laboratory testing. The data obtained from the maturity method with respect to compressive strength may be utilized as a tool for determining the time for opening to traffic and formwork removal.

The developed methodology of utilizing the maturity method for estimating resistance of concrete to chloride-ion penetration (using AASHTO T 277 "rapid chloride permeability" (RCP) test) may be used as a tool to assess the risk of surface scaling due to deicing salts for late-season bridge deck construction. If the in-situ RCP values at the time of first salt application are estimated to be excessive (> 1500 coulombs), decisions may be made to extended moist curing period and/or apply heating blankets to minimize the risk of scaling.

Finally, the developed maturity method may be used for quality control purposes to obtain the 28-day and 56-day RCP values, respectively, at about 7 and 13 days (for the ternary concrete studied with 20% fly ash and 5% silica fume), by subjecting the concrete

to accelerated curing at an elevated temperature (i.e., 38°C). This approach allows for increased efficiency in verifying the compliance of ternary concrete of with applicable specifications.

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APPENDIX

Table A1 –Experimental details for mixtures from literature review

Reference	Mixture Type	No. of Ternary/ Total Mixtures Studied	Control/ Binary Ref. Mixture	Cement Type	Fly Ash Class	Silica Fume Form	Binder Composition					W/CM	Air Entrainment	Tests Performed	Testing Age (days)	Curing Condition
							Cement Content (kg/m ³)	Fly Ash Content		Silica Fume Content						
								(kg/m ³)	(%) by mass of binder	(kg/m ³)	(%) by mass of binder					
Popovics (1993)	Mortar	18/60	Y/Y (FA, SF)	I, III	C, F	Slurry	525	218	29	8	1	0.57	N	▪ Flow table ▪ Compr. strength ▪ Ultrasonic pulse vel.	▪ 1,7,28 ▪ 1,7,28	Standard
							525	210	28	15	2					
							525	188	25	38	5					
Zhang et al. (1999)	Concrete	3/5	Y/Y (FA)	I	F	Dry, Uncompacted	179	267	55	39	8	0.33	Y	▪ Compr. strength ▪ Splitting-tensile str. ▪ Modulus of elast. ▪ Bond b/w shotcrete and base concrete ▪ F-T	▪ 7,28,91 ▪ 28,180 ▪ 91 ▪ 91 ▪ 365	▪ Moist curing ▪ 7 days under wet burlap followed by curing in air outdoors
							144	312	65	24	5					
							176	262	55	38	8					
Yaşar et al. (2004)	Concrete	1/4	Y/N	I	C	na	350	100	20	50.0	10	0.55	N	▪ Compr. strength ▪ Flexural strength	▪ 3,7,14, 28,90	65% RH
Sarkar et al. (2004)	Concrete	1/2	N/Y (SF)	V	F	na	315	90	20	45	10	0.30	N	▪ Compr. strength ▪ SEM, TEM, EDXA, XRDA	▪ 0.5,1, 7,14, 28,91	Standard

Carette and Malhotra (1993)	Concrete	18/ 30	Y/Y (FA)	I	F	Condensed	265 263 262 209 206 208 171 172 172 148 149 149 131 130 130	113 113 113 90 88 90 74 74 74 63 63 63 56 55 56	27 26 25 27 26 25 28 26 25 27 25 27 26 28 25	39 56 73 30 45 60 24 37 49 21 32 43 18 28 37	9 13 16 9 13 17 9 13 17 9 13 17 9 13 17	0.36 0.35 0.33 0.45 0.43 0.42 0.55 0.52 0.50 0.64 0.61 0.58 0.73 0.69 0.67	Y	<ul style="list-style-type: none"> Compr. strength Flexural strength 	<ul style="list-style-type: none"> 1,3,7, 28,56, 91 7,14 	Standard
Chen et al. (2003)	Paste	1/ 11	Y/Y (FA, SF)	na	na	na	na	na	0-30	na	0-30	0.28	N	<ul style="list-style-type: none"> Compr. strength 	<ul style="list-style-type: none"> 7,28 	Standard
Khan and Lynsdale (2000)	Paste	21/ 32	Y/Y (FA, SF)	I	F	Slurry	na	na	15 20 25 30 35 40 45	na	5 10 15	0.30	N	<ul style="list-style-type: none"> Compr. strength Porosity 	<ul style="list-style-type: none"> 7,28, 90 	Standard
Khan (2003a)	Mortar	9/ 16	Y/Y (FA, SF)	I	F	Slurry	na	na	20 30 40	na	5 10 15	0.27	N	<ul style="list-style-type: none"> Compr. strength Porosity Oxygen permeability 	<ul style="list-style-type: none"> 28,90, 180 	Standard

Khan (2003b)	Concrete	9/24	Y/Y (FA, SF)	I	F	Slurry	na	na	20 30 40	na	5 10 15	0.27 0.40 0.50	N	<ul style="list-style-type: none"> ▪ Porosity ▪ Oxygen permeability ▪ Sorptivity ▪ MIP ▪ Compr. strength 	<ul style="list-style-type: none"> ▪ 28,90, 180 	<ul style="list-style-type: none"> ▪ Standard (moist curing) ▪ Air curing (65% RH) ▪ Curing at 5°C (65% RH)
Khan and Lynsdale (2002)	Concrete	9/24	Y/Y (FA, SF)	I	F	Slurry	na	na	20 30 40	na	5 10 15	0.27 0.40 0.50	N	<ul style="list-style-type: none"> ▪ Splitting tensile strength ▪ Compr. Strength ▪ Oxygen permeability ▪ Carbonation 	<ul style="list-style-type: none"> ▪ 28,90, 180 	Standard
Thomas et al. (1999)	Concrete	na/na	Y/Y (FA, SF)	na	F, CI, C	Blended w\ cem.	na	na	10 15 20 25 30 40 50	na	3 5 7 10	na	Y na	<ul style="list-style-type: none"> ▪ Compr. Strength ▪ Chloride diffusion ▪ ASR ▪ Sulfate resistance 	<ul style="list-style-type: none"> ▪ 1,3,7, 14,28, 90,365 (str.) ▪ na 	Standard
Suksawang et al. (2005b)	Concrete	1/7	Y/Y (FA, SF)	I	F	na	517	138	20	34	5	0.27	Y	<ul style="list-style-type: none"> ▪ Compr. Strength ▪ Drying shrinkage ▪ Creep 	<ul style="list-style-type: none"> ▪ 1,3,7, 14,28, 56,90 (str.) ▪ <90 (shrinkage) ▪ < 365 (creep) 	Standard

Nikam and Tambvekar (2003)	Concrete	4/ 22	Y/Y (FA, SF)	I	F	na	178 207 178 207	128 96 118 89	40 30 37 28	14 17 24 24	4.5 5.2 7.5 7.5	0.45	na	▪ RCP ▪ ISAT	▪ 28	Standard
Olek et al. (2003)	Concrete	2/ 10	Y/Y (FA, SF)	I	C	Dry, Densified	269 211	98 156	25 40	23 23	6 6	0.40	Y	▪ Compr. Strength ▪ CTH	▪ 1,3,7, 14,28, 56,180 (str.) ▪ 28,180	Standard
Jones et al. (1997)	Concrete	5/ 22	Y/Y (FA)	na	na	na	180 140 250 220 265	20 140 30 220 265	10 45 10 45 45	20 30 30 45 65	10 10 10 10 10	0.84 0.60 0.60 0.38 0.31	na	▪ Compr. Strength ▪ Electroch emical chloride ingress ▪ Carbonati on ▪ ISAT	▪ 28	Standard
Shehata and Thomas (2002)	Concrete, Paste	6/ 20	Y/Y (FA, SF)	na	C, CI	na	357 336 315 273	42 63 184 126	10 15 20 30	21 21 21 21	5 5 5 5	0.45	na	▪ ASR expansion ▪ Pore solution chemistry ▪ TGA	na	Standard

Blomberg (2003)	Concrete, Mortar	1/11	Y/Y (FA, SF)	I	C	na	282	53	15	21	6	0.42	Y	<ul style="list-style-type: none"> Compr. Strength Elastic modulus F-T resistance Salt scaling resistance RCP Ponding test Cracking resistance (ring test) Plastic shrinkage crack test Autoclave expansion Drying shrinkage 	<ul style="list-style-type: none"> 3,7,14,28,56,90 (str.) 3,7,14,28,56 (elastic mod.) 120 (scal.) 106 (F-T) 28,56,90 (RCP) 90 (ponding) 	Standard
Nagataki et al. (1998)	Mortar, Concrete	11/18 (mort.) 10/15 (con.)	Y/Y (FA)	I	F	na	400	80 70 120 105 160 140	16 14 22 19 27 23	20 30 30 45 40 60	4 6 5.5 8 7 10	0.30	Y	<ul style="list-style-type: none"> PSD Flowability Compr. strength MIP 	<ul style="list-style-type: none"> 3,7,14,28,91 	<ul style="list-style-type: none"> Standard
Marchand et al. (1992)	Concrete	2/13	Y/Y (FA, SF)	I	F	with	280	60	17	21	6	0.30	Y/ N	<ul style="list-style-type: none"> Air-void system analysis Scaling 	<ul style="list-style-type: none"> 28 	<ul style="list-style-type: none"> Standard Curing compound

Nassif et al (2005)	Concrete	17/ 30	Y/Y (FA, SF)	I	F	Dry, Densified	405 358 334 310 360 338 371 347 427 402 402 441 453 500 513 499 414	48 95 95 95 42 63 71 86 50 75 75 85 85 96 64 143 118	10 20 20 20 10 15 15 18 10 10 15 15 15 10 20 20	24 24 48 72 21 21 33 43 25 25 25 40 28 48 32 71 59	5 5 10 15 5 5 7 9 5 5 5 7 5 7 10 10	0.39 0.39 0.39 0.39 0.44 0.44 0.39 0.39 0.37 0.37 0.35 0.33 0.33 0.29 0.29 0.29 0.35	Y/ N	<ul style="list-style-type: none"> Compr. strength Elastic modulus 	<ul style="list-style-type: none"> 1,3,7, 14,28, 56,90 	<ul style="list-style-type: none"> Standard Air drying Curing compound Wet burlap
Palecki and Setzer (2006)	Concrete	3/ 11	Y/Y (SF)	I	na	na	375	75	20	26	7	0.30	N	<ul style="list-style-type: none"> F-T (capillary suction and F-T) MIP 	<ul style="list-style-type: none"> 7,28, 180, 1095 	<ul style="list-style-type: none"> Standard Lab storage Field storage
Nehdi and Sumner (2002)	Mortar	13/ 26	Y/Y (FA, SF)	I	F	na	na	na	30 60	na	10 20	0.40	N	<ul style="list-style-type: none"> Flow table Drying shrinkage Setting time Compr. strength Expansion due to exposure to sulphates 	<ul style="list-style-type: none"> < 112 (drying shrinkage) 1,7,28, 56 (compr str.) <150 (sulphate resistance) 	<ul style="list-style-type: none"> Standard

Xi et al. (2001)	Concrete	31/ 35	Y/Y (FA, SF)	I/II	C, F	Dry, Densified	267	53	16	11	3	0.37	Y	<ul style="list-style-type: none"> Compr. strength RCP Resistance to cracking Drying shrinkage 	<ul style="list-style-type: none"> 3,7,28, 56 (str.) 28,56 (RCP) 1,7,12 (restrained ring test) 	<ul style="list-style-type: none"> Standard
							267	53	16	11	3	0.41				
							267	53	16	11	3	0.44				
							267	67	20	11	3	0.37				
							267	67	20	11	3	0.41				
							267	67	20	11	3	0.45				
							288	58	16	11	3	0.37				
							288	58	16	11	3	0.41				
							288	58	16	11	3	0.45				
							288	72	20	11	3	0.37				
							288	72	20	11	3	0.41				
							288	72	20	11	3	0.45				
							307	61	16	12	3	0.37				
							307	61	16	12	3	0.41				
							307	61	16	12	3	0.45				
							307	76	20	12	3	0.37				
							307	76	20	12	3	0.41				
							307	76	20	12	3	0.45				
							288	66	18	11	3	0.41				
							267	77	21	15	4	0.41				
							279	53	15	15	4	0.42				
							279	47	14	15	4	0.42				
							291	66	18	12	3	0.41				
							291	58	16	12	3	0.41				
							276	69	19	11	3	0.37				
Weng et al. (1997)	Paste	1/6	Y,Y (FA, SF)	I	C	na	-	-	30	-	10	0.30 0.40	N	<ul style="list-style-type: none"> TGA 	<ul style="list-style-type: none"> 0.5,1, 2,3,7, 28,90, 180 	Standard
Magee and Olek (2000)	Concrete	-	-	-	-	-	-	-	-	-	-	-	-	-	-	

Bouzoubaâ et al. (2004)	Concrete, mortar	28/48	Y,Y (FA, SF)	I	F, C	na	267 255 236 221 195 184 301 283 257 241 212 195	70 71 107 107 139 141	20 20 30 30 40 40	14 28 14 28	4 8 4 8 4 8	0.40 0.34	Y	<ul style="list-style-type: none"> ▪ Bleeding ▪ Setting time ▪ Autogenous temperature rise ▪ Plastic shrinkage ▪ Compr. strength ▪ RCP 	<ul style="list-style-type: none"> ▪ 1,7,28,91 (str.) ▪ 28,120/150 (RCP) 	<ul style="list-style-type: none"> ▪ Standard ▪ Moist curing for 3 days followed by air drying (50% RH)
Temiz and Karakeci (2002)	Paste	1/3	Y,Y (FA)	I	C	na	-	-	24	-	6	0.40	N	<ul style="list-style-type: none"> ▪ XRD ▪ SEM 	<ul style="list-style-type: none"> ▪ 7,28,90 	Sealed
Mehta and Gjorv (1982)	Concrete, Paste	1/4	Y,Y (FA, SF)	I	F	na	210	30	11	30	11	0.75	N	<ul style="list-style-type: none"> ▪ Compr. strength ▪ MIP ▪ Relative lime content 	<ul style="list-style-type: none"> ▪ 3,7,28,90 (str.) ▪ 7,28,90 (MIP, lime content) 	Standard
Lilkov and Stoitchkov (1996)	Mortar, Paste	na	Y,Y (FA, SF)	PC 35 A2 0 (w 15 % G GB S)	F	na	-	-	22.5 15 7.5 5	-	7.5 15 2.5 5	0.33	N	<ul style="list-style-type: none"> ▪ Pozzolanic activity ▪ TGA ▪ Compr. strength ▪ Flexural strength 	<ul style="list-style-type: none"> ▪ 7,28,90,360 	Standard

Ho et al. (2003)	Concrete	4/ 20	Y,Y (FA, SF)	I	F	na	na	na	30	na	10	0.81 0.60 0.49 0.34	N	<ul style="list-style-type: none"> ▪ Compr. strength ▪ Sorptivity 	<ul style="list-style-type: none"> ▪ 1,28 (str.) ▪ 28 (sorptivity) 	<ul style="list-style-type: none"> ▪ Standard (27°C) ▪ Steam curing (up to 55°C) (strength) ▪ 2-d at 27°C followed by curing at 27°C and 75% RH ▪ 6-d at 27°C followed by curing at 27°C and 75% RH ▪ 1-d of steam cycles+2-d wrapped in plastic envelope ▪ 1-d of steam cycles + 6-d wrapped in plastic envelope (sorpt.)
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Olek et al. (2002)	Concrete	15/55	Y,Y (FA, SF)	I	C	Dry, Densified	293	78	20	20	5	0.40	N, Y	<ul style="list-style-type: none"> Compr. strength Elastic modulus RCP Chloride conductivity Scaling F-T Maturity Drying shrinkage (free and restrained) RMP Chloride ponding 	<ul style="list-style-type: none"> 7,28, 56 (str.) 28,56 (elastic mod., RCP, chloride conduct.) 28 (F-T, scaling) < 7 (maturity) 90 (chloride ponding) 28,56, 180 (RMC, RCP) 	Standard
							322	39	10	29	7.5	0.45				
							244	117	30	29	7.5	0.45				
							341	39	10	10	2.5	0.45				
							263	117	30	10	2.5	0.45				
							322	39	10	29	7.5	0.35				
							244	117	30	29	7.5	0.35				
							341	39	10	10	2.5	0.35				
							263	117	30	10	2.5	0.35				
							293	78	20	20	5	0.50				
							215	156	40	20	5	0.40				
							293	78	20	20	5	0.30				
							273	78	20	39	10	0.40				
							269	98	25	23	6	0.40				
							211	156	40	23	6	0.40				
Streeter (1996)	Concrete	4/5	Y/N	na	F	na	297	80	20	24	6	0.40	Y	<ul style="list-style-type: none"> RCP Plastic shrinkage cracking Compr. strength Scaling Ponding test 	<ul style="list-style-type: none"> 1,3,4,7, 14,28 (compr. str) 90 (ponding test) 	na
							289	71	19	24	6	0.40				
							279	71	19	24	6	0.40				
							343	46	11	36	8	0.36				

Bajorski and Streeter (2000)	Concrete	24/ 24	N/N	na	F	na	263 228 249 214 228 262 215 247 238 273 205 236 238 203 273 311 325 249 260 303 316 241 252 282	53 88 53 88 65 75 65 75 49 56 82 94 70 105 32 82 85 143 149 82 85 143 149 115	15 25 15 25 20 20 20 20 15 15 25 25 20 30 10 20 20 35 35 20 20 35 27.5	35 35 49 49 33 37 46 52 39 45 39 45 42 42 16 17 16 17 25 26 25 26 21	10 10 14 14 10 10 14 14 12 12 12 12 12 12 4 4 4 4 6 6 6 6 5	na	na	<ul style="list-style-type: none"> ▪ RCP ▪ Plastic shrinkage cracking ▪ Compr. strength ▪ Scaling 	<ul style="list-style-type: none"> ▪ 3,7,14, 28 (compr. str) 	na
Suksawang et al. (2005a)	Concrete	11/ 11	N/N	na	F	na	na 330 394 503	na 88 46 59	15 10 15 20 25 18 20 20 20 22 10 10	na 22 23 30	7 5 5 5 5 9 10 15 5 5 5	na 0.39 0.38 0.29	Y	<ul style="list-style-type: none"> ▪ Compr. str. ▪ Elastic modulus ▪ Drying shrinkage ▪ Creep from compr. load ▪ RCP ▪ Scaling ▪ F-T 	<ul style="list-style-type: none"> ▪ 1,3,7,1 4,28,56 ,90 (compr. str) ▪ 28 (elastic mod.) ▪ 14 (F-T) ▪ 28,56, 90 (RCP) 	<ul style="list-style-type: none"> ▪ Moist curing ▪ Air drying ▪ Burlap curing ▪ Curing compound

Panchalan and Ramakrishnan (2007)	Concrete	3/10	Y/Y (FA, SF)	na	F	Dry, Densified	301 309 290	70 70 93	17 17 23	32 23 23	8 6 6	0.39 0.39 0.38	Y	<ul style="list-style-type: none"> ▪ RCP ▪ Compr. Str. ▪ Flexural Str. 	<ul style="list-style-type: none"> ▪ 7,90 (RCP) ▪ 14,28 (compr. str., flexural str.) 	<ul style="list-style-type: none"> ▪ Standard Accelerated (at 38°C for 7 days)
Lilkov and Petrov (1997)	Paste	1/4	Y/Y (FA, SF)	na	F	na	-	-	5	-	5	0.33	N	<ul style="list-style-type: none"> ▪ DTA ▪ IR spectroscopy ▪ XRD 	<ul style="list-style-type: none"> ▪ 1,2,3,7, 28,90, 180, 360 (TGA) 	na
Isaia (1997)	Paste	2/12	Y/Y (FA, SF)	na	F	Undensified	- -	- -	15 30	- -	10 20	0.40	N	<ul style="list-style-type: none"> ▪ XRD ▪ Fratini test (pozzolanic activity) ▪ TGA ▪ Compr str. ▪ Particle size 	<ul style="list-style-type: none"> ▪ 28,91 	na
Wong et al. (2007)	Paste	3/6	Y/Y (FA, SF)	I	na	Densified	- - -	- - -	13 27 42	- - -	11 12 12	0.1- 0.35	N	<ul style="list-style-type: none"> ▪ Wet packing density ▪ Marsch cone flow ▪ Minislump spread ▪ PSD (Laser diffraction) 	-	-
Lilkov et al. (1997)	Paste	1/4	Y/Y (FA, SF)	na	F	na	-	-	5	-	5	0.33	N	<ul style="list-style-type: none"> ▪ DTA ▪ IR spectroscopy ▪ XRD ▪ PSD (Laser diffraction) ▪ SEM 	<ul style="list-style-type: none"> ▪ 1,3,4,6, 8,12, 24 h (TGA, IR) 	At 20°C in plastic containers

Ozyildirim and Halstead (1994)	Concrete	16/ 18	Y/N	II, III	F	na	232	114	32	11	3	0.44	Y	<ul style="list-style-type: none"> Compr. str. RCP 	<ul style="list-style-type: none"> 1,7,28 (compr . str) 14,28 (RCP) 	<ul style="list-style-type: none"> Moist cur. for 1d or 14d at one of three temperature s: 6°, 23° and 38°C (series 1) 1d moist cur. at 23°C or 38°C + 27d air drying 14d moist cur. at 23°C + 14d air drying 1d moist cur. at 38°C + 13d at 23°C +14d air drying 3d moist curing at 38°C+11d at 23°C +14d air drying
							232	93	26	32	9	0.44				
							194	141	40	18	5	0.44				
							193	122	35	36	10	0.44				
							211	123	35	18	5	0.40				
							213	124	35	18	5	0.45				
							231	106	30	18	5	0.40				
							231	106	30	18	5	0.45				

Ronne (1989)	Concrete	1/2	N/Y (FA)	Blended (with 25% FA)	F	Densified, Slurry	na	na	23	na	9	na	N	<ul style="list-style-type: none"> ▪ Compr. str. 	<ul style="list-style-type: none"> ▪ 7,28 	<ul style="list-style-type: none"> ▪ Standard ▪ Drying at 20°C and 50% RH at 1 day ▪ Drying at 20°C and 50% RH at 3 days ▪ Drying at 20°C and 50% RH from one side (at 24h) ▪ Standard until 28 days, then in water at 70°C ▪ Drying at 20°C and 50% RH until 28 days, then drying at 70°C
Nocuń-Wczelik (2007)	Paste	6/1 3	Y/Y (FA)	I	F	na	na	na	10 20 30	na	10	0.50	N	<ul style="list-style-type: none"> ▪ Isothermal calorimetry ▪ TGA ▪ SEM 	<ul style="list-style-type: none"> ▪ 1 and 3 (TGA) ▪ 7 (SEM) 	na

Baoyu et al. (1989)	Concrete	13/17	N/Y (FA, SF)	425R, 525R (early-age strength)	F	Condensed	192 191 191 169 145 166 152 140 183 183 166 157	48 48 48 48 47 41 40 60 61 61 61 61	19 19 18 21 23 19 23 28 23 23 25 25	13 17 25 17 16 14 14 14 17 26 16 23	5 7 10 7 8 7 7 7 10 7 10	0.51 0.50 0.43 0.51 0.60 0.52 0.53 0.52 0.42 0.39 0.46 0.44	N	<ul style="list-style-type: none"> Compr. str. Tensile str. Adiabatic calorimetry Abrasion resistance Permeability Ultimate elongation Modulus of elasticity 	3,7,28, 90 (compr. and tensile str.)	na
Lawler et al. (2005)	Concrete	4/10	Y,Y (FA)	I	C F	na	274 203 312 262	98 156 59 98	25 40 15 5	20 31 20 31	5 8 5 8	0.37 0.37 0.37 0.45	Y	<ul style="list-style-type: none"> Compr. str. Cracking tendency (ring test) Heat of hydration Free shrinkage Modulus of elasticity Air-void system parameters F-T resistance RCP Salt scaling resistance Ponding test 	3,7,28, 56 (compr. str.) 1,3,7,14, 28,56, 90 (free shrinkage) 7,28 (modulus of elasticity)	Standard

Note: “na” means that the specific information is not available in the original publication