AN ABSTRACT OF THE DISSERTATION OF

Tengfei Fu for the degree of Doctor of Philosophy in Civil Engineering presented on June 3rd, 2013

Title: Shrinkage Study of High Performance Concrete for Bridge Decks.
Abstract approved:

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Dr. Jason H. Ideker

In the field of civil infrastructures, bridge decks have been widely constructed using high performance concrete (HPC). Concrete bridge decks demand qualities such as low permeability, high abrasion resistance, superior durability, and long design life. Over decades of fields and laboratory experience, many HPC for bridge decks were found susceptible to shrinkage and cracking issue, which is also regarded as a major cause for structure deficiency and deterioration.

A comprehensive study was presented in this dissertation on shrinkage and shrinkage induced cracking in HPC, in three aspects: 1) to mitigate the shrinkage and cracking issues in HPC, internal curing by fine lightweight aggregate (FLWA) and incorporation of shrinkage reducing admixture (SRA) have proven effective during the last 15 years. To determine the optimum FLWA content, chemical shrinkage of the cementitious materials needs to be determined. A simple and improved procedure was recommend to determine the long-term chemical shrinkage for HPC systems containing supplementary cementitious materials (SCMs) and/or SRA; 2) due to the fact that drying shrinkage significantly affects concrete bridge deck cracking performance, it is important to develop drying shrinkage prediction models. However, each existing model is limited by the data source used to develop the model, which is not likely representative all modern HPC. To solve this issue, a procedure was proposed based on the current ACI 209 model

to predict long-term drying shrinkage for modern HPC concrete by using short-term experimental measurements; 3) a state-of-the-art literature review on shrinkage and cracking issue on bridge deck HPC showed that the causes behind cracking in HPC were well known and documented. However, appropriate shrinkage limits and standard laboratory/field tests which allow proper criteria to ensure cracking-free of highly cracking resistant HPC is not clearly established either in the technical literature or in specifications. A “cracking potential indicator” (CPI) concept was proposed to assess cracking risk of candidate concrete mixture deigns. A simple and robust test procedure to determine CPI, involving only free shrinkage and mechanical properties of HPC, was also identified.

Shrinkage Study of High Performance Concrete for Bridge Decks

by

Tengfei Fu

A DISSERTATION

submitted to

Oregon State University

in partial fulfillment of
the requirements for the
degree of

Doctor of Philosophy

Presented June 3rd, 2013

Commencement June 2013

Doctor of Philosophy dissertation of Tengfei Fu presented on June 3rd, 2013.

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Dean of the Graduate School

I understand that my thesis will become part of the permanent collection of Oregon State University libraries. My signature below authorizes release of my thesis to any reader upon request.

Tengfei Fu, Author

ACKNOWLEDGEMENTS

First of all, I would like to express my sincere gratitude to my advisor, mentor, and friend, Dr. Jason Ideker, for his generous support over the years I was pursuing a doctoral degree. I appreciated all the opportunities you have given me. You made my graduate experience so busy and rewarding. I will never forget travelling all over the world to study concrete. You truly set an example for me to follow. As your first doctoral students, I sincerely hope I did not let you down. I also hope this mentorship and friendship could last lifelong.

I also would like to thank Dr. Burkan Isgor, Dr. Christopher Higgins, Dr. Brady Gibbons, and Dr. Lech Muszyński for their service as my committee members. All your comments and questions are well appreciated.

I would like to thank all my colleagues and friends. Tyler Deboodt, without your consistent help through all these projects, none of these journal articles would come to existence. It was such a pleasure working with you. Matthew Adams, you are such a pleasant and intelligent colleague to talk to. I enjoy our conversions over life, politics, and of course, research. I wish you the best in your academic endeavors and look forward to working with you in the future. Nicholas Tymvios, I could never forget the tough time we went through together looking for jobs and writing dissertation. I would also thank other colleagues Dr. Thomas Schumacher, Kelsea Schumacher, Chang Li, Jose Bañuelos, David Rodriguez, Jimmy Chen, Lapyote Prasittisopin. And special thanks to Manfred Dittrich, James Batti, and all other CCE staff, for a pleasant graduate school experience. I also cherish the friends I made during all the conference, Dr. Anthony Bentivegna, Karla Kruse, Fred Aguayo, Yi Huang and Ted Moffatt. In addition, I’d also thank all my other friends during my stay in Corvallis, Juan Zheng, Meng Li, Yisen Guo, Dr. Wei Liu, Wei Wang, Fan Zhang, Jian Huang, Xiaoou Han, Hai Yang, and Jinzhou Cao.

Last, but far from least, I would dedicate this dissertation to my parents, for their unconditional love and support. Also, I couldn’t say thank you enough times to my wife, Mochi, for the love we shared, and for always believing in me even when I lost faith in myself.

CONTRIBUTION OF AUTHORS

Tyler Deboodt was involved in all aspects of work including lab equipment setup, data collection/processing, help with reviewing all three manuscripts. Dr. Jason Ideker advised on data analysis/interpretation, and reviewing/revising all three manuscripts.

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1 GENERAL INTRODUCTION

1.1 BACKGROUD AND INTRODUCTION

Cracking, especially at early age, in high performance concrete (HPC) may result in a significant decrease in concrete durability and service life of the structure containing it. Concrete bridge decks demand qualities from HPC such as low permeability, high abrasion resistance, superior durability, long design life. To meet these requirements, bridge deck concrete is usually produced with low water to cementitious material ratio (w/cm) less than 0.40, high overall cement contents, inclusion of supplementary cementitious materials (SCMs) such as silica fume, fly ash and slag, and smaller maximum aggregate size (due to reinforcement constraints). All these features in the mixture design make the HPC bridge decks inherently susceptible to shrinkage and increased cracking risk.

At early age, HPC is prone to autogenous shrinkage, plastic shrinkage, drying shrinkage and sometimes thermal changes, due to the immature skeleton structure in the cement paste to resist the stress generated by the volume change. Internal curing, as the names suggests, refers to the technique that cures the concrete from the inside out, by incorporating reservoir water through curing agents, such as pre-wetted fine lightweight aggregate (FLWA) and/or superabsorbent polymers (SAP). It has been proven effective to mitigate early age cracking and has gradually moved from laboratory experiments to field applications.

At later age, there are many factors which can lead to cracking in HPC bridge decks, including drying shrinkage, creep (sometimes can be beneficial), environmental fluctuations, loading and restraint conditions. In fact, more than 100,000 bridges decks in this country have suffered from transverse cracking, which is a pattern indicating the presence of drying shrinkage. Over the past 40 years, most of the above-mentioned causes of cracking in concrete bridge decks have been well identified and documented through laboratory research and field experience. Furthermore, proper mixture modifications and construction practices have been developed to minimize the risk of cracking. Nevertheless, concrete still exhibits cracking during its service life and as a result this continues to be a significant research thrust by many agencies.

1.2 SCOPE AND ORGNIZATION OF THIS DISSERTATION

This dissertation consists of a focused literature review and three manuscripts, derived from research work as part of two projects sponsored by the Oregon Department of Transportation (ODOT): SPR711 Internal Curing of Concrete Bridge Decks and SPR728 Development of Shrinkage Limits and Testing Protocols for ODOT High Performance Concrete. The later project is closely related to the former one. And the cracking limits research was also partially funded by the Portland Cement Association (PCA) through Education Foundation Fellowship.

The objectives of the research projects addressed in this dissertation are listed as follow:

- Identify chemical shrinkage of cement paste in HPC;
- Evaluate the shrinkage reducing efficiency of fine light weight aggregate (FLWA) and/or shrinkage reducing admixture (SRA) incorporated HPC mixtures;
- Identify an effective and accurate method to predict long-term drying shrinkage based on short-term experiments;
- Evaluate the effect of external wet curing duration on FLWA and/or SRA incorporated HPC mixtures, in the hope of reducing current 14 day external curing period;
- Analyze and identify a drying shrinkage threshold limit/criteria for HPC bridge deck to ensure high cracking-resistance concrete;
- Develop a simple testing procedure for the above limit/criteria which can be easily applied by contractors or materials suppliers;
- Investigate alternative cement options such as calcium aluminate cement, shrinkage compensating cement or calcium sulfoaluminate cement.

This dissertation is organized as follows:

Chapter 1: General Introduction.

Chapter 2: Literature Review. The focuses of this literature review are drying shrinkage and cracking studies in the last decade.

Chapter 3: Manuscript 1. The title of this manuscript is “A Simple Procedure on Determining Long-Term Chemical Shrinkage for Cementitious Systems Using Improved Standard Chemical Shrinkage Test”. Research has shown that the benefits of incorporating pre-wetted fine lightweight aggregate and shrinkage reducing admixture in high performance concrete. To determine the optimum FLWA content, information about the propensity for shrinkage in the cement paste, specifically the chemical shrinkage value, is needed. However, there is a lack of information on how to determine the long-term chemical shrinkage value for HPC with supplementary cementitious materials (SCMs) and/or shrinkage reducing admixture (SRA). In this research, a simple procedure was identified to predict the long-term chemical shrinkage value for high performance cementitious systems containing SCMs and/or SRAs. Several improvement to the ASTM C1608 (dilatometry procedure) were proposed and investigated. Among which, “mess check” and “lowered sample higher” were adopted in the lasted ASTM C1608. An experimental prediction model was adopted and verified to estimate long-term chemical shrinkage values for portland cement systems containing SCMs and/or SRA. A recommended procedure was proposed to determine the long-term chemical shrinkage values for HPC systems containing SCMs and/or SRA, and a modification to a commonly used LWFA proportioning equation was suggested. This manuscript was published in the *ASCE Journal of Materials in Civil Engineering* on Janurary 26, 2012.

Chapter 4: Manuscript 2. The title of this manuscript is “Prediction of Drying Shrinkage for Internally Cured High Performance Concrete”. Since concrete durability is closely related to effects of shrinkage, it is important to develop proper prediction models. The ACI 209 model is recommended by American Concrete Institute and widely used in the

U.S. for normal strength concretes using conventional aggregates. It was recommended that a short-term testing should be performed to calibrate the model to improve predictions for local materials. However, the calibration procedure is not clearly stated in the document. In this research, a procedure based on the ACI 209 shrinkage model to predict long-term shrinkage strain using short-term experimental measurements was proposed. In addition, evaluation of the accuracy of six existing shrinkage models is reported compared to the authors' experimental data. These models are the ACI 209 model, the CEB90 model, the AASHTO model, the B3 model, the GL2000 model, and the ALSN model. The shrinkage values determined by each model were compared against the experimental results from ten high-performance concrete mixtures with incorporation of FLWAs and/or SRAs. This manuscript was published in *ACI Symposium Publication 290 The Economics, Performance and Sustainability of Internally Cured Concrete*, and presented on October 23rd, 2012, Toronto, Canada.

Chapter 5: Manuscript 3. The title of this manuscript is "Assessing Drying Shrinkage Related Cracking Potential of High Performance Concrete for Bridge Decks". Cracking at an early-age of high performance reinforced concrete structures, in particular bridge decks, results in additional maintenance costs, burden on serviceability and reduced long-term performance and durability. The causes behind cracking in high performance concrete are well known and documented in the existing literature. However, appropriate shrinkage limits and standard laboratory/field tests that allow proper criteria to ensure crack-free or highly cracking-resistant high performance concrete are not clearly established either in the technical literature or in specifications. The purpose of this research was to provide shrinkage threshold limits for specifications and to provide a robust test procedure that allows easy determination on compliance with specified threshold limits. It has been shown that the "ring" tests (ASTM C1581 and AASHTO T334) are the most comprehensive accelerated laboratory tests to accurately identify cracking potential. In addition, acceptable correlation between the ring test and the field test has been observed and documented. However, a more simple and robust test procedure is in demand from materials suppliers and Departments of Transportation. A data analysis of current experimental results showed that the free shrinkage to shrinkage capacity (theoretical strain related to tensile strength and modulus of elasticity) ratio, namely “cracking potential indicator”, is a promising assessment of cracking resistant performance. In this way, only free shrinkage test (ASTM C157) and basic mechanical properties are required to assess cracking risk of certain concrete mixture design. This manuscript will be submitted to *Journal of Cement and Concrete Composites*.

Chapter 6: General Conclusions.

Appendix A: A summary of five drying shrinkage prediction models evaluated in Manuscript 2.

2 LITERATURE REVIEW

2.1 INTRODUCTION

In 2010, the United States (US) Federal Highway Administration (FHWA) reported that 27% of the country's bridges in the National Highway System were considered "structurally deficient" or "functionally obsolete", which refers to bridges having major deterioration, cracks, or other deficiencies in their structural components including decks, girders, or foundations (*U.S. Department of Transportation 2010*). According to a survey conducted by Krauss and Rogalla (*Krauss and Rogalla 1996*) in 1996, 62% of respondents in the US departments of transportation (DOTs) believed transverse cracking was a significant problem. More than 100,000 bridges decks had suffered from transverse cracking, which is a pattern indicating the presence of drying shrinkage. However, there are many factors which can lead to cracking in concrete bridge deck, such as concrete dimensional stabilities (shrinkage and creep), environment fluctuations and restraint conditions. Fig.1 shows a summary of major causes of bridge deck cracking. All causes shown in Fig. 1 that can lead to cracking are manifested as a volume change (including differential) in concrete. Cracking is determined by the competition between strength gain of the materials and the development of tensile stress, which is mainly due to restraint provided by the structural elements. Over the past 40 years, most of the causes of cracking in concrete bridge decks have been well identified and documented through laboratory research and field experience. Furthermore, proper mixture modifications and construction practices have been developed to minimize the risk of cracking. Nevertheless, concrete still exhibits cracking during its service life.

Cracking at early ages (especially within the first year after placement) results in additional costs and a significant maintenance burden. These added costs could be reduced through improved testing techniques, improved materials specifications, and improved construction requirements related to reducing cracking risks in such structures. The most significant challenge, from a concrete materials perspective, to overcoming cracking risk is to reduce the shrinkage, and ultimately the stresses generated as a result of such shrinkage. However, there is no singular testing method or a subsequent shrinkage threshold limit commonly agreed upon to reduce such risk. The goal of this literature review is to summarize previous efforts on linking drying shrinkage of HPC and performance specifications (i.e. limits).

A detailed literature review on drying shrinkage testing methods, factors affecting drying shrinkage, and mitigation strategies can be found in literature (*Fu 2011*). In addition, a summary of drying shrinkage prediction models was given in Appendix B.

2.2 RECENT RESEARCH ON BRIDGE DECK SRHINAKGE/CRACKING

2.2.1 **American Standards of Testing Materials (ASTM)**

The restrained ring test has been used by a number of researchers since the 1940's. It is a practical tool to evaluate cracking risk of concrete and mortar. It was not until a few decades ago, that quantitative analysis of this test has come into existence by implementing strain gauges to qualify the stress state of the cementitious specimens (*ACI Committee 231 2010*). Weiss (*Weiss 1999*) made contribution in stress distribution analysis in the ring using a nonlinear fracture mechanics model. Later, the ground work of ASTM C1581 was laid down by See et al. (*See et al. 2004*). They investigated a wide range of concrete and mortar mixtures using a specific ring test (Figure 2.1), which was later adopted by ASTM as a standard testing produce. Based on the results, they suggested a cracking potential classification (as shown in Table 2.1) on the basis of either time-to-cracking or stress rate development in the concrete ring specimen. This classification was also adopted by ASTM C1581.

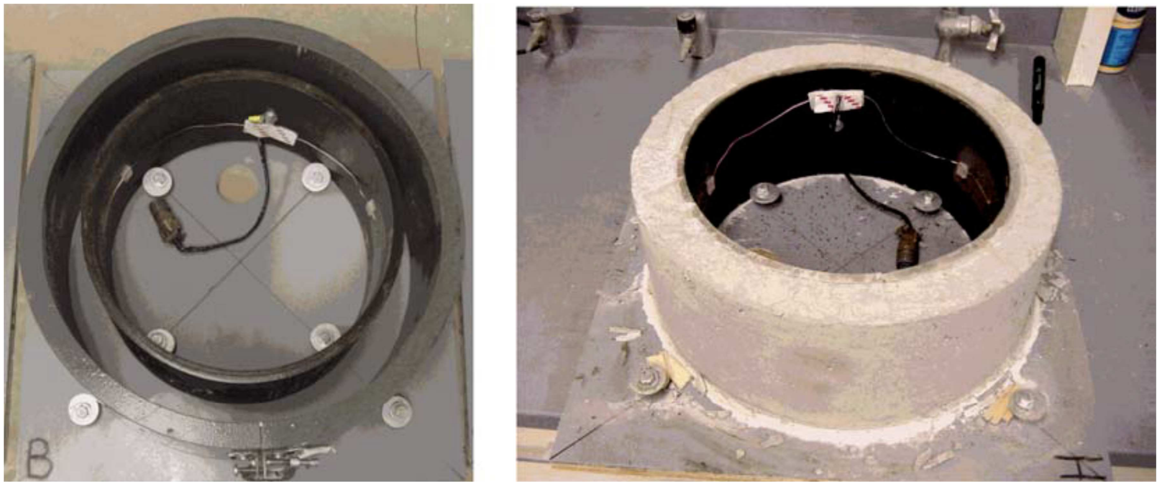

Figure 2.1 ASTM test specimen mold (left) and test specimen (right) (*See et al. 2004*)

Table 2.1 Cracking potential classification (Based on stress rate at time-to-cracking). (*ASTM C1581 2004; See et al. 2004*)

Time-to-Cracking, t_{cr}, Days	Stress Rate at Cracking, S, MPa/Day	Potential for Cracking
$0 < t_{cr} \leq 7$	$S \geq 0.34$	High
$7 < t_{cr} \leq 14$	$0.17 < S < 0.34$	Moderate-High
$14 < t_{cr} \leq 28$	$0.10 < S < 0.17$	Moderate-Low
$t_{cr} > 28$	$S < 0.10$	Low

Time-to-cracking is the difference between the age at cracking and the age drying was initiated. It can be used to assess the relative cracking performance of specimens that cracked during the test. If not cracked, the stress rate at the age that test was terminated can be compared between tested materials.

2.2.2 Texas Department of Transportation (TxDOT)

From 2002 to 2006, a two-phase project titled "*Evaluation of Alternative Materials to Control Drying-Shrinkage Cracking in Concrete Bridge Decks*" was conducted by TxDOT and the University of Texas, Austin. The major goal of this project was to identify an effective materials-based method of controlling drying shrinkage.

In the Phase-I of this research, a detailed summary on factors affecting cracking in concrete bridge deck was given in terms of shortcomings in materials, design practices and construction techniques. Common methods of controlling shrinkage cracking were also identified in the literature review section, including conventional and innovative methods. The innovative methods included fiber-reinforced concrete, SRAs, shrinkage compensating concrete and high-volume fly ash (HVFA), which were all evaluated in laboratory experimentation. To better identify the cracking propensity, a combination of laboratory tests were recommended:

- Free shrinkage prism test (ASTM C157/AASHTO T-160);
- Restrained ring test (ASTM C1581/AASHTO PP34), and;
- Early-age strength properties:
  - Compressive strength test (ASTM C39/AASHTO T-22);
  - Tensile strength test, and (ASTM C496/AASHTO T-198);
  - Modulus of elasticity test (ASTM C469).

Each of these tests by itself was not capable of providing sufficient information to evaluate the propensity for drying shrinkage induced cracking. It was recommended that an ideal crack-free or highly crack-resistant mixture should be one showing no cracking in the ring test, having a relatively low free shrinkage strain and early-age modulus of elasticity, meanwhile high early-age tensile strength. However, the complicated interaction among all these properties made it very difficult to prescribe a specific free shrinkage limit as a permissible threshold for materials selection. Therefore, a number of comprehensive considerations were recommended for the decision making process in literature (*Folliard et al. 2003*).

In the phase-II study, a satisfactory correlation was found between the ring test and large-scale bridge decks (LSBD) cast and monitored at an outdoor exposure site in Austin, Texas. However, to determine the relative susceptibility to drying shrinkage cracking, the ASTM C 157 prism test would be inadequate on its own, and many other recommended tests results should be considered. Based on a comprehensive test result, concrete mixtures containing SRAs, polypropylene fibers, shrinkage compensating cement or

HVFA were recommended to minimize early-age shrinkage stress and cracking risk (Brown et al. 2007).

2.2.3 **Virginia Department of Transportation (VaDOT)**

A study in 2004 by VaDOT recommended drying shrinkage limits of 0.04% length change at 28 days and 0.05% length change at 90 days for concrete containing SCMs following ASTM C157 test. For OPC concrete, the limits are set to 0.03% at 28 days, and 0.04% at 90 days. This was done by comparing unrestrained drying shrinkage in the ASTM C157 prisms to restrained cracking tendency in ASTM C1581 testing. However, mixtures with the lowest free shrinkage did not subsequently exhibit the lowest strains in restrained ring testing. Due to the lack of variance in the data, this draws into question the validity of the shrinkage limits purported by the study for mixtures of lower w/c and ternary blends (*Mokarem et al. 2005*).

From 2007 to 2010, a project titled "*Bridge Deck Concrete Volume Change*" was conducted by the Virginia Transportation Research Council (*Ramniceanu et al. 2010*). The goal of this research was to develop a field quality control method for shrinkage and its associated limits. Shrinkage was evaluated at the early-age (24 hours) and long-term (180 days) for VaDOT concrete bridge deck mixtures, including ternary blended mixtures (fly ash and microsilica), latex modified mixtures and expansive mixtures. A modified ASTM C157 prism test was used to test early-age shrinkage, and normal ASTM C157 procedures were used to measure the long-term shrinkage. Ring tests, v-notch tests and scaled bridge deck overlays were used to evaluate shrinkage cracking potentials. Based on the test results, the ASTM C157 test method was recommended to VaDOT as an implementation to control shrinkage of field overlays and general bridge deck mixtures. The limits of each current mixture are shown in Table 2.2. These limits were based on ASTM C157 free shrinkage test when compared to the scaled bridge overlay specimens. Since none of the scaled bridge overlay specimens cracked, the limits were taken from the free shrinkage values.

Table 2.2 ASTM C157 Shrinkage Control Limits*, inspired by (*Ramniceanu et al. 2010*)

Age	Overlay Mixtures (μm/m)				A4 Mixtures (μm/m)		
	LMC	RSL	LMK	TRN	A4-FA	A4-S	A4-K
3 Days	300(310)	150(125)	150(125)	400(380)	-	-	-
7 Days	400(395)	250(215)	300(280)	700(670)	250(206)	350(350)	300(273)
28 Days	600(580)	350(295)	400(350)	800(750)	500(370)	500(537)	400(385)

* Values between barracks are experimental measurements.

For example, the measured free shrinkage for standard HPC mixture (TRN) at 28 day was 750 micron, while the limit was set for 800 micron. However, the restrained ring tests were not in the good agreement with scaled bridge overlay specimens. Nonetheless, the researchers stated that the scaled bridge deck specimen best mimicked field conditions, thus the free shrinkage limits should be linked to the performance of scaled bridge deck (*Ramniceanu et al. 2010*).

2.2.4 **Kansas Department of Transportation (KDOT)**

In 2005, KDOT's report "*Evaluating Shrinkage and Cracking Behavior of Concrete Using Restrained Ring and Free Shrinkage Tests*", provided a detailed review of previous research efforts on concrete bridge deck cracking was provided. In addition, it also provided a comprehensive review of the ring test including the background of the ring test, different types of ring test and effect of ring geometry. Free shrinkage and restrained ring tests were used in this study to evaluate concrete bridge deck mixture designs used within the state. The major conclusions in this study were as follows: (*Tritsch et al. 2005*)

- Using coarser ground (Type II) cements could reduce shrinkage;
- Shrinkage increased with increased paste content;
- Use of a shrinkage reducing admixture (SRA) significantly reduced shrinkage;
- Longer curing times were beneficial to reduce shrinkage, and;
- Free shrinkage was found to be a weak predictor of actual restrained shrinkage.

The researchers attempted to correlate free shrinkage with restrained shrinkage rate, but found that the free shrinkage was a weak predictor of actual restrained shrinkage rate. Of 39 restrained ring tests, only one mixture with a high paste content cracked. Such low cracking sensitivity was due to the thickness of the steel ring, which was too thin to provide enough restraint to promote cracking in concrete rings. This resulted in a lack of comprehensive view of mixtures with low to high cracking risk (*Tritsch et al. 2005*).

Nevertheless, this study provided guidance to reduce shrinkage and laid down the groundwork for the later two-phase pooled fund study "*Construction of Crack-Free Concrete Bridge Decks*". This study focused on: (*Lindquist et al. 2008; McLeod et al. 2009*)

- Development of an aggregate optimization and concrete mixture design program;
- Free-shrinkage tests to evaluate potential low cracking HPC (LC-HPC) mixtures;
- Evaluation of the chloride penetration into concrete using long-term salt-ponding tests;
- Specification for LC-HPC construction and standard practices in Kansas, and;
- Construction and preliminary evaluation of LC-HPC bridge decks in Kansas.

The LC-HPC mixture, also usually referred as "KU Mix", has been proved effective in reducing cracking in bridge decks by field applications (*Darwin et al. 2010*). Some of the features of this low cracking mixture are listed as follows:

- Optimized aggregate gradation
- Recommended moderate strength (25-30 MPa)
- Low cementitious materials content (≤ 320 kg/m^3)
- Moderate w/cm (0.43-0.45)
- 25mm maximum aggregate size
- Air content of 8±1.5 %
- Low designated slump (40-90 mm)
- Controlled construction temperature (13-21 ºC)

2.2.5 **New Jersey Department of Transportation (NJDOT)**

New Jersey DOT (NJDOT) performed a research project from 2005 to 2007, to investigate the cracking potential of the HPC mixes for bridge decks in New Jersey State (*Nassif et al. 2007*). Comprehensive laboratory tests were conducted, including compressive strength, splitting tensile strength, modulus of elasticity, free shrinkage, and restrained shrinkage. For restrained shrinkage test, AASHTO PP34-99 was utilized with modifications to better capture the cracking performance by monitoring the relative displacement within the ring specimen (as shown in Figure 2.2). In addition to the strain gauges attached to the inner surface of the steel ring, six vibrating wire strain gauges (VWSG) were installed to monitor the relative movement in concrete ring sections. In this way, the actually strain in the concrete could be measured and quantified, which allows a more accurate comparison between mixtures.

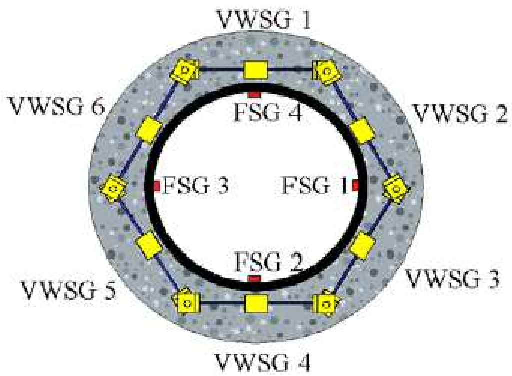

Figure 2.2 Modified AASHTO restrained ring tests setup (FSG: foil strain gauge; VWSG: vibrating wire strain gauge) (*Nassif et al. 2007*).

They found that high coarse aggregate to fine aggregate ratio (over 1.5) with high coarse aggregate content (over 1110 kg/m^3) could help significantly reduce cracking potentials. By correlating free shrinkage to restrained shrinkage performance, a free shrinkage limit of 450 microns at 56 days was recommended to ensure a high cracking resistant performance for HPC bridge deck.

2.2.6 **West Virginia Division of Highways (WVDOH)**

Recent research by Ray and co-workers discovered a correlation between material properties (28-day compressive strength and 90-day free shrinkage strain) and time of cracking obtained from AASHTO ring test (*Ray et al. 2012*). In this research, 18 different HPC mixture designs with different SCMs and different w/c were investigated. The ASTM C157 test was used to measure free shrinkage strain. AASTHO ring tests were used to obtain cracking potential (time to cracking in the ring). According to the test results, a correlation was established between "cracking index" and time to cracking in ring. The cracking index was given as $100f^{0.1}\varepsilon^{-1.0}E^{-1.2}$ based on observation, a combined factor of common material properties such as free shrinkage strain (ε), compressive strength (f), and modulus of elasticity (E). A data set from representative highway bridge projects was used to determine the cracking threshold, as shown in Figure 2.3. The conclusion was that to be conservative any concrete mixture design with a time to cracking of 30 days or later in the AASHTO ring test would be acceptable in cracking resistance on the field.

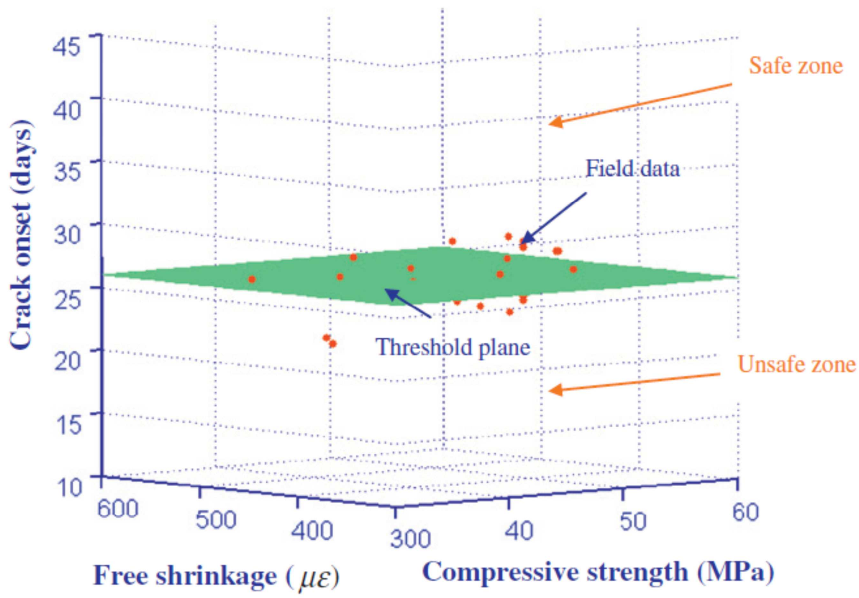

Figure 2.3 Threshold plane of cracking onset based on field data. (*Ray et al. 2012*)

This research was the first attempt to combined free shrinkage with common materials properties, which provides a more comprehensive understanding of cracking issues in concrete. Although this method still needs to be further confirmed or upgraded, a new prospective was provided in how to determine the laboratory testing threshold limits to minimize cracking risk in the field. Another noticeable contribution of this work was that a simple and feasible modification to the ACI 209 shrinkage model was proposed to more accurately predict shrinkage using local materials. Details can be found in literature (*Ray et al. 2012*).

2.2.7 **Washington Department of Transportation (WSDOT)**

In 2010, WSDOT conducted research with Washington State University, titled "*Mitigation Strategies for Early-Age Shrinkage Cracking in Bridge Decks*". The goal of this research was to identify effective early-age cracking mitigation strategies for concrete bridge decks in Washington State. The research report included a comprehensive literature review and suggested the focus of this study was to identify mitigation methods based on material properties, such as different sources and sizes of aggregates, paste content, cementitious materials including SCMs and SRAs. Free shrinkage and restrained ring tests were performed on 22 mixtures designs including two current WSDOT concrete mixes. Based on the laboratory evaluations, the major conclusions are listed: (*Qiao et al. 2010*)

- SRAs significantly reduced the free shrinkage and restrained shrinkage cracking tendency of all mixtures;
- Less paste volume due to larger aggregate size reduced free shrinkage and delayed cracking in the ring specimens, and;
- Lower free shrinkage strain, with acceptable flexural strength, generally indicated relatively good restrained shrinkage cracking resistance.

In this study, two different sizes of rings were used for restrained ring testing. This provided different degrees of restraint and could accommodate different sizes of coarse aggregates. Hardened concrete properties, such as compressive strength, splitting tensile strength, flexural strength and modulus of elasticity were tested at 7 days and 28 days. "KU Mix" was also applied in one of the investigated mixtures. The shrinkage was reduced from 400 μm/m to 150 μm/m at 28 day. The significant differences between the control mixture and the "KU Mix" include: reduced cement content from 440 kg/m^3 to 325 kg/m^3, increased maximum aggregate size from 19 mm to 25 mm, and optimization of aggregate gradation.

The authors also attempted to link free shrinkage strain to cracking, and determined the concrete cracking resistance was the combination of its tensile strength and its free shrinkage properties. However, no shrinkage limit was proposed. Further field evaluation was needed to verify the link between free shrinkage with restrained cracking and ultimately with field performance (*Qiao et al. 2010*).

2.2.8 **Other Works**

Al-Manaseer and coworkers (*Al-Manaseer et al. 2011*) conducted a long-term shrinkage and creep study on high strength concrete (HSC). This work was also supported by California Department of Transportation (Caltrans). Eighty-one mixtures with different SCMs and superplasticizers were investigated. Free drying shrinkage measurements in cement and concrete samples lasted up to 3000 days. They documented the effect of SCMs (i.e. fly ash, silica fume, slag, and metakaolin), superplasticizers, and especially SRAs on compressive and long-term free drying shrinkage. No cracking evaluation was performed. They found that by incorporating SRA the shrinkage was significantly reduced. They also found that increasing the SRA dosage above 1.5% had no significant effect on free drying shrinkage. A shrinkage prediction model which takes SRA into consideration was proposed, termed ALSN 2004 model (*Al-Manaseer and Ristanovic 2004*). This model was also evaluated in Chapter 3, and details of this model can be found in Appendix B.

In 2002 to 2003, Michigan DOT (MDOT) conducted an investigation of causes and methods to minimize early-age deck cracking on Michigan Bridge decks (*Aktan et al. 2003*). A nationwide survey was also conducted as part of the research. The results showed 30 of 31 responding states (as shown in Figure 2.4) reported early-age bridge decking cracking issues except Hawaii. Twenty-five states indicated the cracking happened during the first several months after placement, and 11 responded as during the first year. The literature review pointed out that main factors influencing bridge deck cracking performance were restrained thermal and shrinkage coupled with construction practices. From the field inspection data and laboratory testing, a thermal load of approximately 11 ºC was identified to initiate deck cracking. The research team suggested that the hydration temperature rise should be limited in the standard specifications. They also suggested a continuation of this research to develop a specific mixture design for the minimization of thermal load.

Figure 2.4 Map of responding states (*Aktan et al. 2003*)

For the last decade, Ohio Department of Transportation (ODOT) investigated the bridge decking cracking issues through an in-state field survey (*Crowl and Sutak 2002*), laboratory testing (*Delatte et al. 2007*), and full scale bridge deck study(*Delatte and Crowl 2012*). The survey covered a total of 116 HPC bridge decks constructed between 1994 and 2001. All 64 bridge decks which showed minimal or no cracking used coarse aggregate with higher absorption capacity (>1%). Meanwhile, 75% of the remaining 52 bridge decks with severe cracking used coarse aggregate with lower absorption capacity (<1%). To rule out other possible factors, a bridge deck was casted as field trial in 2002, in two phases. The only difference between two phases was coarse aggregate sources. As shown in Figure 2.5, Phase 2 which used lower absorption (<1%) coarse aggregate cracked, while Phase 1 which used higher absorption >1%) coarse aggregate did not show any cracking. This strongly suggested that the cracking resistance was related to the aggregate scours. In the later laboratory evaluation, they found the internal curing by FLWA significant was able to reduce shrinkage in HPC, and more significant reduction could be achieved by using larger coarse aggregate. It was also supported by a full scale bridge deck field trail.

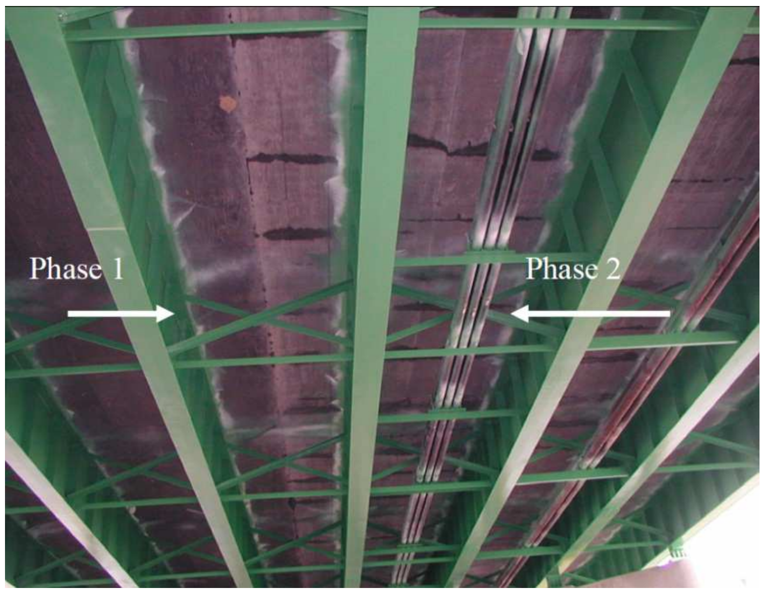

Figure 2.5 Phased construction of HPC bridge deck in Ohio (*Delatte et al. 2007*)

2.3 SUMMARY

Cracking in the bridge decks causes shortened service life of the structure, and increased burdens to state DOTs through maintenance, retrofit and inspection. In this literature review, recent studies on shrinkage and cracking issues on bridge decks were summarized. The current understanding of high-cracking-resistance concrete is that the concrete should have low free shrinkage and early-age young's modulus, meanwhile high tensile (or flexural) strength. From the testing perspective, several well-established tests exist for assessing shrinkage and/or cracking risk of concrete mixtures (e.g. standard/modified ring tests, and scaled bridge deck). It is well-agreed upon that the restrained test (ring test) can provide the best prediction of concrete cracking. Along with materials properties tests (such as compressive strength, tensile strength and modulus of elasticity), it is possible to set shrinkage limits. It is anticipated that a laboratory testing procedure using the ring test and other mechanical properties tests is promising to determine cracking potential of HPC mixture for bridge decks.

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3 MANUSCIPT 1

A Simple Procedure on Determining Long-Term Chemical Shrinkage for Cementitious Systems Using Improved Standard Chemical Shrinkage Test

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Journal of Materials in Civil Engineering

American Society of Civil Engineers

Published on January 26, 2012

DOI: 10.1061/(ASCE)MT.1943-5533.0000486

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3.1 ABSTRACT

In the past ten years, renewed research interest has shown the benefits of internal curing by incorporating pre-wetted lightweight fine aggregate (LWFA) in high performance concrete (HPC). To determine the optimum LWFA content, information about the propensity for shrinkage in the cement paste, specifically the chemical shrinkage value, is needed. However, there is a lack of information on how to determine the long-term chemical shrinkage value for HPC with supplementary cementitious materials (SCMs) and/or shrinkage reducing admixture (SRA). The purpose of this research was to identify a simple procedure to determine long-term chemical shrinkage values for given cementitious systems with SCMs and/or SRA. Several improvement to the ASTM C1608 (dilatometry procedure) were investigated. An experimental prediction model was adopted and verified to estimate long-term chemical shrinkage values for portland cement systems containing SCMs and/or SRA. A recommended procedure is proposed to determine the long-term chemical shrinkage values for HPC systems containing SCMs and/or SRA, and a modification to a commonly used LWFA proportioning equation is suggested.

3.2 INTRODUCTION

In the past ten years, renewed interest in the form of research has shown the benefits of internal curing (IC) by incorporating pre-wetted lightweight fine aggregate (LWFA) in high performance concrete (HPC) *(Bentz and Snyder 1999; Lura 2003; Bentz 2007; Cusson and Hoogeveen 2008; Wei and Hansen 2008; Henkensiefken et al. 2009; Sahmaran et al. 2009*). As a results, this technology has steadily progressed from laboratory studies to field applications*(Henkensiefken et al. 2009*), including highway pavement design as well as residential construction *(Villarreal and Crocker 2007; Delatte et al. 2008; Friggle and Reeves 2008; Villarreal 2008*). To determine the optimum LWFA content, Bentz and co-workers developed an equation (also known as Bentz Equation) for determining the replacement of the standard fine aggregate with dry LWFA *(Bentz and Snyder 1999; Bentz et al. 2005*). This is shown as follows:

$$M_{LWFA} = \frac{C_f \times CS \times \alpha_{\max}}{S \times \Phi_{LWFA}} \quad with \quad \alpha_{\max} = \frac{w/cm}{0.36} \leq 1 \tag{3.1}$$

Where C_f = cement factor (content) for concrete mixture (kg/m^3); CS = chemical shrinkage of the cement (g of water/g of cement); α_{max} = maximum expected degree of hydration of cement; S = degree of saturation for LWFA; and Φ_{LWFA}= absorption of lightweight fine aggregate (kg water/kg dry LWFA).

To use this predictive equation, several parameters need to be determined including: C_f, CS, α_{max}, S and Φ_{LWFA}. Then the mass of the lightweight aggregate per unit volume of concrete (M_{LWFA} in kg/m^3) can be calculated. The maximum degree of saturation can be established by first determining the absorption capacity of the LWFA. If the moisture content of the aggregate is at or above the absorption capacity, the maximum degree of saturation was taken as 1. While there is no agreed upon method to determine this value, Weiss and co-workers have provided recommendations by using the standard cone test outlined in ASTM C128 as a reasonable alternative for finding the absorption capacity of the LWFA (*Schlitter et al. 2010*). For typical HPC with water to cement ration lower than 0.36, the complete hydration usually cannot be achieved and the maximum expected degree of hydration can be estimated by (w/cm)/0.36 (*Bentz et al. 2005*)

Chemical shrinkage is usually referred to the volume reduction due to chemical reactions between cement and water forming hydration products of higher density than original reactant. Chemical shrinkage value is important to optimize LWFA content in internal curing. Higher chemical shrinkage value might indicate that more moisture is needed from LWFA. Therefore, a proper chemical shrinkage value is crucial when estimating proper LWFA content. In the previous research, chemical shrinkage value has been determined by the phase composition of cement *(Paulini 1992; Justnes et al. 1998; Mounanga et al. 2004; Bentz et al. 2005*). However, there is a lack of information on how to determine long-term chemical shrinkage value when applying Bentz Equation for

HPC with supplementary cementitious materials (SCMs) and/or shrinkage reducing admixture (SRA).

It should be noted that a better estimate of the CS value does not necessarily indicate an optimum replacement level of LWFA to create a better long-term performing concrete. Other factors such as water accessibility, drying shrinkage, durability, and mechanical properties should be considered when determining an appropriate replacement level for field applications. While previous research has indicated that the use of a pre-wetted LWFA was effective at reducing autogenous shrinkage *(Bentz 2007; Sahmaran et al. 2009; Slatnick et al. 2011*) and drying shrinkage in mortars up to 28 days (*Schlitter et al. 2010*), drying shrinkage on concrete with LWA/SRA is not well-established. Many of these other parameters have are either currently under investigation or merit further research.

The purpose of this study was to identify a simple procedure to determine long-term chemical shrinkage value for given concrete systems with SCMs and/or SRA. Improvements to the ASTM C1608 *Standard Test Method for Chemical Shrinkage of Hydraulic Cement Paste* were investigated. There improvements included 1) extended testing duration (up to 14 days); 2) smaller sample size (height); 3) quality check for each sample by measure the mass change before and after the test; 4) an automated data acquisition system, and; 5) an experimental prediction model, first proposed by Xiao et al. (*Xiao et al. 2009*), was adopted and verified to estimate the long-term chemical shrinkage value for portland cement systems containing SCMs or SRA. All the above mentioned improvements enable the Bentz Equation to accommodate concrete mixtures with SCMs and SRAs when determining the appropriate pre-wetted LWFA replacement level.

3.3 MATERIALS AND MIXING PROCEDURES

The high performance mixture used for chemical shrinkage contained 30% class F fly ash and 4% silica fume replacement by weight of cement. The ASTM C150 Type I/II portland cement and ASTM C618 class F fly ash were used in this study. The chemical composition of the cement and fly ash is shown in Table 3.1. A silica fume containing nearly pure silica dioxide in noncrystalline form with approximately 1% crystalline silica was used. One SRA was also applied in this study. Paste mixture ratio for chemical shrinkage test is listed in Table 3.2.

Table 3.1 Cement and fly ash oxide analysis (wt %)

	CaO	SiO_2	Al_2O_3	Fe_2O_3	MgO	Na_2O	K_2O	TiO_2	MnO_2	P_2O_5	SrO	BaO	SO_3	Total Alkalies as Na_2O	Loss on Ignition	Insoluble Residue
OPC	64.2	20.5	4.72	3.23	0.8	0.3	0.29	0.23	0.08	0.07	0.17	0.07	2.7	0.49	2.63	0.21
Fly Ash	10.2	55.2	15.8	6.27	3.64	3.64	2.08	0.94	0.12	0.23	0.32	0.62	0.7	-	0.23	-

Table 3.2 Paste Mixture Ratio for Chemical Shrinkage Test (w/cm=0.37, 23°C isothermal)

Mixture	Cement (%)	Fly Ash (%)	Silica Fume (%)	SRA (%)
P-1	100	0	0	0
P-2	96	0	4	0
P-3	70	30	0	0
P-4	100	0	0	2
P-5	66	30	4	0

An expanded shale LWFA was used as the IC agent with an absorption capacity of 17% and specific gravity of 1.87. Additionally, the LWFA was able to release as much as 94% of its absorbed water at a relative humidity of 84% through desorption isotherms. Oven dry LWFA was pre-soaked until the absorption capacity had been achieved. The LWFA was then brought to a surface dry condition with a known mass. It was then placed in a series of temperature and humidity controlled chambers, each with a decreasing value of RH. The sample was placed in each respective chamber until hygroscopic equilibrium had been achieved. Upon reaching equilibrium at the lowest RH chamber (84%), the aggregate was then brought to an oven dry condition. The water loss was determined as the amount of water given off by the aggregate from its absorption capacity to 84% RH. A local siliceous river sand and local river gravel were used as the normal weight aggregates.

Concrete mixture proportions are outlined in Table 3.3. The same w/cm as the CS testing was used in concrete testing. The aggregate proportions in Table 3.3 are given in the oven dry state. Prior to mixing The LWFA was pre-soaked for a minimum of 5 days and brought to near surface dry conditions prior to mixing. For mixture C-3, an underestimate of the CS value was selected. The CS value for this mixture was 0.056 g/g. This resulted in a 20% replacement of the normal weight sand with LWFA, and provided 22.4 kg/m^3 of absorbed water for internal curing. In mixture C-4, the measured value performed in this research of CS = 0.07 g/g which resulted in a 25% replacement of normal weight sand with LWFA. The LWFA had absorbed 27.8 kg/m^3 of water prior to mixing. It should be noted that the dosage of air entraining admixture and high range water reducing admixture were adjusted to produce similar workability and air content for all three mixtures.

Table 3.3 Concrete Mixture Proportioning for Drying Shrinkage Test

Mixture	Cement (kg/m^3)	Fly ash (kg/m^3)	Silica fume (kg/m^3)	Water (kg/m^3)	Coarse aggregate (kg/m^3)	Sand (kg/m^3)	LWFA (kg/m^3)	SRA (kg/m^3)	AEA (mL/m^3)	HRWR (L/m^3)
C-1	249	112	15	139	1074	659	-	-	103	1.5
C-2	249	112	15	131	1074	659	-	6	174	1.4
C-3	249	112	15	139	1074	451	132	-	103	1.5
C-4	249	112	15	139	1074	400	164	-	103	1.5

The concrete was mixed according to ASTM C192, and unrestrained drying shrinkage prisms were cast according to a modified ASTM C157. The concrete was removed from their respective molds at an age of 24 hour (± 30 mins) and subjected to a drying environment of 23 ^{o}C and 50% relative humidity.

3.4 CHEMICAL SHRINKAGE TEST

3.4.1 **Automated ASTM C1608**

ASTM C1608 was used as the standard testing procedure to investigate chemical shrinkage with improvements. To prepare the paste sample, the standard procedure outlined in ASTM C305 was used. Cement paste (with or without SCMs) was mixed with de-aerated de-ionized water at room temperature, and was carefully placed to the desired height in a vial (25mL) with dimension of 50mm in height and 25mm in diameter. After placing the paste in the vial, de-aerated de-ionized water was used to fill the vial to the top carefully without disturbing the paste. A one-hole rubber stopper with an inverted glass pipette (1 mL) passing through it, was placed slowly on top of the vial to make sure no air bubbles were trapped. Additional water was filled from the top of the pipette to bring the water level close to the highest reading in the pipette. Then a few drops of color indicating dye (red in color) were added by a syringe in order to prevent evaporation and also to provide the reference for the automated data acquisition system. Then the mass of each pipette-vial setup was measured before they were placed in the specifically designed rack, which allow all vials submerged in the water bath for temperature control at 23 °C. At the end of the 14 day testing period, the mass was measured again. A successful test without leakage in the system (complete pipette-vial setup) achieved mass conservation with no more than 0.02 g change in mass.

To monitor chemical shrinkage for a longer period of time (e.g. up to 14 days and more) than recommended by ASTM C1608, which is currently “at least 24 hours” (*ASTM 2007*), an automated test regime was developed. A functioning setup is shown in Figure 3.1. In front of the pipettes, there is a webcam with a resolution higher than 1.3 megapixels for image acquisition. Images are taken and analyzed using a computer software program developed in the Laboratory of Materials of Construction (LMC) at EPFL in Lausanne, Switzerland, to determine the total water uptake by the hydrating cement paste (*Bishnoi 2009*). Similar experimental setups have been used by several other researchers *(Costoya 2008; Ideker 2008; Jaouadi 2008*). A statistical study was performed showing that the automated measurements were consistently of less variance than the measurements taken by hand using the same images (*Fu 2011*). All results presented in this paper are thus from automated measurements.

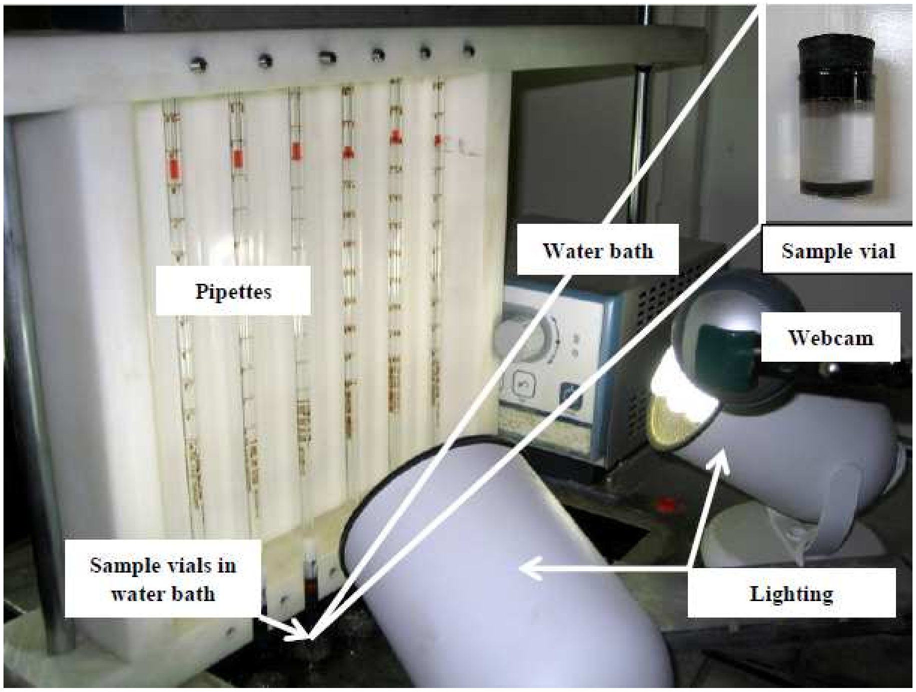

Figure 3.1 Automated chemical shrinkage setup

3.4.2 **Depercolation Study**

As per ASTM C1608, the thickness should be 5mm to 10mm, which is approximately 5g to 10g paste in the vial used in this testing. Boivin and co-workers have shown that for low w/cm sample with thickness less than 10mm, the influence of sample height was insignificant *(Boivin et al. 1998*). However, recent work by Sant and co-workers has shown a thickness (size) dependent deviation on chemical shrinkage measurements. *(Sant et al. 2006; Sant et al. 2011*) It should be noted that the influence was investigated based on test period of 24 to 48 hours in both studies.

In this research, four different thicknesses were selected, 3 mm, 5 mm, 7mm and 10 mm, to be tested up to 14 days, to investigate the long-term effect of sample size dependence.

Noted that the unit of chemical shrinkage is g/g, which indicates water consumed (g) by unit weight (g) of cement. The results show that the chemical shrinkage increased with a decrease of sample thickness (See Figure 3.2). One possible explanation of the sample size dependence would be that the SCMs and low w/cm densify paste so that the water cannot penetrate through the thicker section. Sant and co-workers describe this phenomenon as "with increasing hydration in the thick section, a combination of effects due to a global porosity decrease and a decrease in connectivity of the capillary pores due to hydration impedes the movement of water" (*Sant et al. 2011*). Therefore, a sample thickness of 3 mm (approximately 3 g of paste) was selected to better quantify the water demand for pre-wetted lightweight fine aggregate. This sample thickness was also justified by another recent work using neutron tomography indicating that water can travel up to 3 mm 20 hours after casting from lightweight aggregate particle to the paste with a w/cm of 0.25 (Trtik *et al.* 2011).

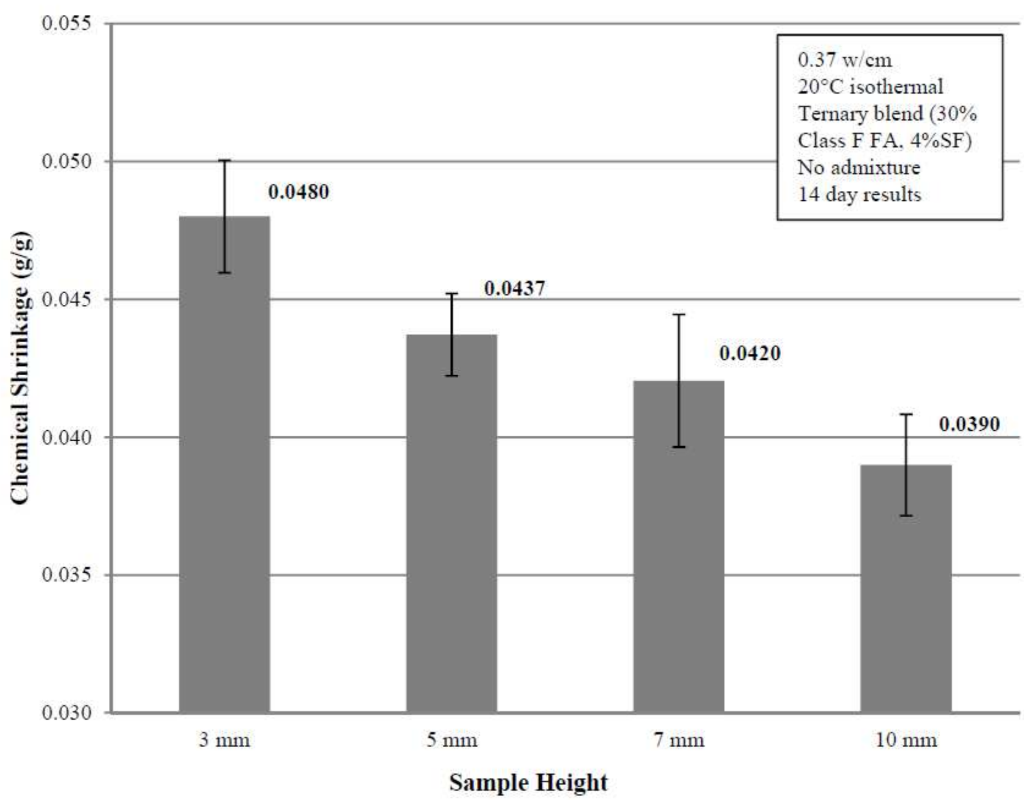

Figure 3.2 CS depercolation study

3.4.3 **Long-term Chemical Shrinkage Prediction Model**

Previous research has shown that the chemical shrinkage of cement paste was decided by its chemical composition. The most important four minerals in cement clinker were C_3S, C_2S, C_3A and C_4AF. With different composition proportion, the reaction rate and ultimate chemical shrinkage value varies. A semiempirical model was proposed to predict the evolution of chemical shrinkage of cement and Eq. (3.2) and Eq. (3.3) shows the calculation of the ultimate chemical shrinkage value based on cement composition *(Bentz et al. 2005)*:

Assuming sufficient sulfate to convert all of the aluminate phases to ettringite:

$$CS_{Ultimate} = 0.0704\,[C_3S] + 0.0724\,[C_2S] + 0.117[C_4AF] + 0.171[C_3A] \qquad (3.2)$$

Assuming total conversion of the aluminate phases to monosulfate:

$$CS_{Ultimate} = 0.0704\,[C_3S] + 0.0724\,[C_2S] + 0.086[C_4AF] + 0.115[C_3A] \qquad (3.2)$$

Values in the brackets are the mass percentage of the composition of specific mineral, and the coefficient in front has a unit of g/g. Therefore, given the composition information of cement used, the ultimate chemical shrinkage value for Bentz Equation can be easily calculated. However, there are several complications behind this method. First of all, the coefficients corresponding to each mineral vary between different researchers *(Paulini 1992; Justnes et al. 1998; Mounanga et al. 2004*). In addition, with the presence of SCMs, the equation cannot be applied, due to different composition and hydration properties of these admixtures. Furthermore, the maximum expected hydration degree is complicated by the incorporation of SCMs. Although Lura and his co-workers explained one approach to include the effect of silica fume (*Lura et al. 2003*), for other SCMs or combinations of SCMs, it becomes more complex to estimate the degree of hydration due to different chemical composition and reactivity.

Based on the above mentioned reasons, a simple empirical prediction model for long-term chemical shrinkage value proposed by Xiao and co-workers was applied in this study. A hyperbolic-like function converging to an asymptote could be the best to describe and predict the tested chemical shrinkage shape (*Xiao et al. 2009*):

$$cs(t) = \frac{CS \times t^a}{t^a + b} \tag{3.4}$$

Where *CS(t)* = chemical shrinkage value at age of t (day); CS_∞ = long-term chemical shrinkage value; *a,b* = hydration constants related to cementitious materials properties.

To determine a particular shape of a CS curve, three parameters are needed. CS_∞ represents the long-term chemical shrinkage value to which the curve converges. And *a* is a hydration constant which depends on cement composition, curing temperature, w/cm, fineness of the cement and addition of SCMs or SRA. Another hydration constant *b* represents the time needed to reach half of the long-term CS value. A nonlinear curve fit was performed to find each set of parameters for one particular mixture.

An experiment results with prediction equation fitting are shown in Table 3.4 the pastes studied in this experiment. It shows that the incorporation of silica fume and fly ash (P-2, P-3 and P-5) increased CS. This is because the typical chemical shrinkage values for SCMs are usually much higher than portland cement. According to Bentz, the typical chemical shrinkage value for silica fume is 0.20 to 0.22 g/g, and 0.10 to 0.16 g/g for Type F fly ash (*Bentz 2007*). Meanwhile, other researchers showed the reaction degree of SCMs is quite low in blended cement and it is almost impossible to reach 100% reaction rate *(Yajun and Cahyadi 2004; Termkhajornkit et al. 2005; Xiao et al. 2009*). Mixture P-3 (with 30% fly ash) shows a particularly high CS might be due to the higher replacement ratio. Mixture P-5 (ternary blend) shows only insignificant increase in CS. It can probably be explained by lower reaction degree of SCMs within the mixture. The results also show that the long-term CS values will not be affected by SRA (P-4).

Table 3.4 Curve fit and predicted long-term chemical shrinkage values

	Parameters			Correlation Coefficient R^2	Predicted 14 days CS (g/g)		
	Predicted Long-term CS (g/g)	a	b		Measured	Predicted	Diff%
P-1*	0.0670	0.771	1.58	0.993	0.0552	0.0555	0.5%
P-2	0.0693	0.681	1.47	0.990	0.0550	0.0557	1.3%
P-3	0.0796	0.658	2.10	0.992	0.0588	0.0581	1.2%
P-4	0.0671	0.856	1.25	0.995	0.0599	0.0594	0.8%
P-5	0.0676	0.691	1.60	0.993	0.0547	0.0536	1.9%

*Ultimate CS calculated by phase composition gives 0.0688 g/g for P-1.

The fitted curves show good consistency with the tested curves (as shown in Figure 3.3), with a correlation coefficient over 0.99. At the age of 14 days, the difference between the predicted value and the measured value is less than 2%. The degree of hydration at the age of 14 days is also calculated based on predicted long-term CS value.

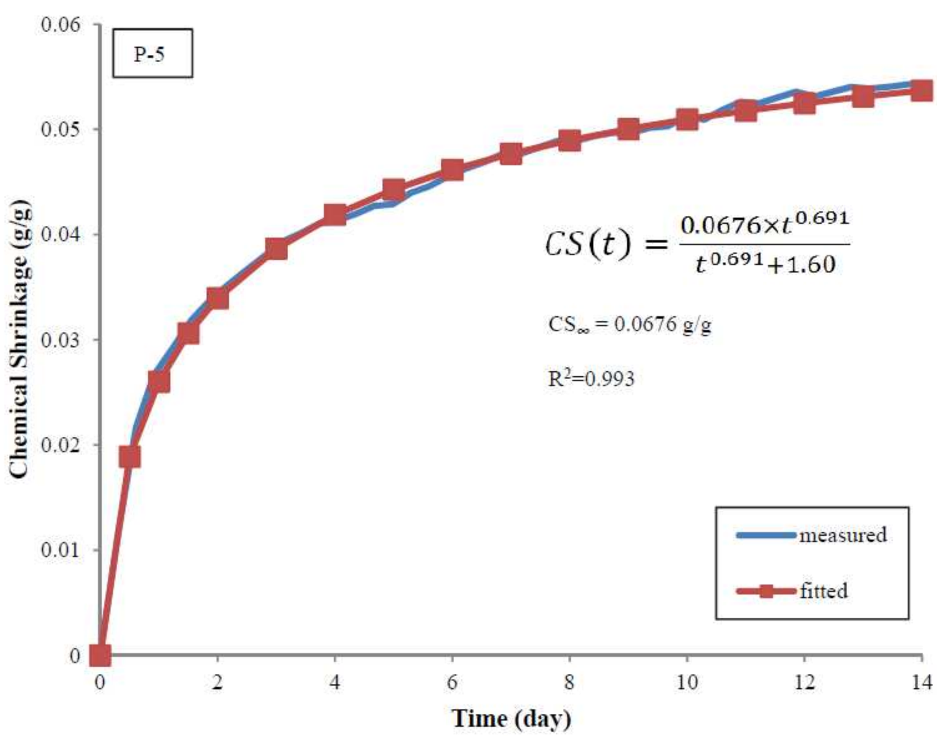

Figure 3.3 Prediction of P-5 mix of 0.37 w/cm at 23ºC isothermal

Another study on the stability of predicted long-term chemical shrinkage based on Xiao and co-workers' data showed that that after the age of 14 days, the predicted long-term chemical shrinkage values are stable. The predicted long-term chemical shrinkage values using data up to different ages longer than 14 days were within 3% difference, which is acceptable and well within the coefficient of variation of other testing parameters. Moreover, the long-term chemical shrinkage values predicted using data of 17 day or older age showed no particular trend (*Fu 2011*).

3.4.4 Recommended Modification on Procedure for Determining the Long-Term Chemical Shrinkage

As discussed previously, the presence of SCMs and SRAs do not allow calculation of ultimate chemical shrinkage value of the cement paste from composition analysis. The selection of a CS value for use in the Bentz Equation therefore needs further refinement including such parameters. Based on the aforementioned discussion, long-term chemical shrinkage value based on a prediction model is recommended. A procedure to obtain predicted CS_∞ is summarized as follow:

Step 1: Perform a modified ASTM C1608 test on the sample cement paste (reducing the sample thickness ≤ 3 mm), SCMs and/or SRA can be included;
Step 2: Record the chemical shrinkage development curve up to 14 day;
Step 3: Use Eq. (3.4) to fit the curve, find CS_∞.

The proposed procedure simplified the chemical shrinkage determination by chemical composition analysis, regardless of the SCMs and other admixtures applied in the mixture. This enables the application of Eq. (3.4) to more complicated mixtures with SCMs or other chemical admixtures like SRA.

Nevertheless, the optimum LWFA content is not solely decided by the chemical shrinkage value. Factors such as absorption/desorption properties of LWA, change in strength and durability parameters are also of great consideration in terms of optimizing LWFA content. Research has shown that a 20% partial replacement of natural sand by saturated LWFA of a 0.35 w/cm concrete would insignificantly decrease the strength and increase permeability *(Durán-Herrera et al. 2007*). While other research has showed that due to the internal curing process, the concrete show higher strength and less permeability. *(Cusson 2008; Henkensiefken et al. 2009*) To verify the proposed procedure, further experimentation is needed to comprehensively investigate all influential factors to optimize LWFA content.

3.5 MODIFICATION OF THE BENTZ EQUATION

In the original Bentz Equation, Eq. (3.1), CS is the amount of water sorbed by cement hydrated. However, a long-term chemical shrinkage value is desirable, yet hardly available. Therefore, a method based on the phase composition of the cement was introduced and described in the previous section. The chemical shrinkage value calculated is referred as “ultimate chemical shrinkage” in this paper, since it is a theoretical value representing the volume reduction when 100% hydration is reached. In this perspective, the CS in the original Bentz Equation is a material property, which is independent to testing time, w/cm, and paste micro structure. And when this value is multiplied by the degree of hydration (α), the amount of water needed for complete internal curing in certain mixture is obtained.

A closer look at the measured chemical shrinkage value reveals that $CS(t)=CS_U\times\alpha(t)$, which indicates that the measured CS value are always related to the hydration degree at the time of measuring, if the ultimate CS is taken as a material property. In this paper, “long-term chemical shrinkage” is referred to the predicted CS value when time tends to infinity, which denoted as CS_∞. Therefore, based on all above-mentioned discussion, the Bentz Equation can be rewritten to accommodate the proposed procedure for determining CS_∞:

$$M_{LWFA} = \frac{C_f \times CS_\infty}{S \times \Phi_{LWFA}} \tag{3.5}$$

It should be noted that C_f in Eq. (3.5) is the total cementitious material content (kg/m^3) for concrete mixture including SCMs such as fly ash, silica fume and slag. This approach enables the incorporation of SCMs and/or SRA in the mixture, which is not the case for the original Bentz Equation. In addition, with the presence of SCMs, the ultimate degree of hydration still needs to be determined to use the Bentz Equation. However it is avoided by the Eq. (3.5). To further improve the efficiency of the Bentz Equation, Castro and his co-workers proposed another modification to accommodate the absorption and desorption properties of LWFA (*Castro et al. 2011*).

3.6 DRYING SHRINKAGE TEST RESULTS

Previous research has shown that the use of pre-wetted LWFA was effective at reducing autogenous shrinkage (*Lura 2003; Schlitter et al. 2010; Slatnick et al. 2011*), the long-term effects of drying shrinkage have been minimally studied. Two different mixtures incorporating LWFA were researched. One mixture was underproportioned for internal curing using the CS of 0.056 g/g, tested at 20°C isothermal with 3 mm sample thickness. The other mixture was proportioned with the CS listed in Table 3.2. In addition, a safety factor of 5% was applied to the measured CS value of 0.0676 g/g to account for variability in the CS test as well as the absorption of the LWFA (*Bentz et al. 2005*). In addition to the LWFA mixtures, a mixture incorporating a commercial shrinkage reducing admixture was cast. The specimens were removed from the molds at 24 hours and then placed in the drying environment that was controlled at 23^{o}C and 50% RH (*ASTM 2008*).

Drying shrinkage results are found in Figure 3.4. The initial reading was taken when the concrete was placed in the drying environment. Through 56 days of testing, the mixture incorporating SRA was the most effective at reducing drying shrinkage. A 42% reduction in drying shrinkage was found when comparing the SRA mixture (C-2) to the control mixture (C-1). It can be seen that incorporating pre-wetted LWFA was less effective at reducing the drying shrinkage when compared to the mixture incorporating SRA. For mixture C-3, the drying shrinkage was reduced by 11% at 56 days when compared to the control mixture. Mixture C-4 reduced the drying shrinkage 15% at 56 days when compared to the control mixture. Underestimating the value for CS increased the amount when compared to a more accurate estimate of the CS value. The difference might be attributed to a lesser amount of internally absorbed water being available for hydration and refilling of capillary voids. While the SRA was the most effective at 56 days, longer-term drying shrinkage of the HPC mixtures needs to be determined.

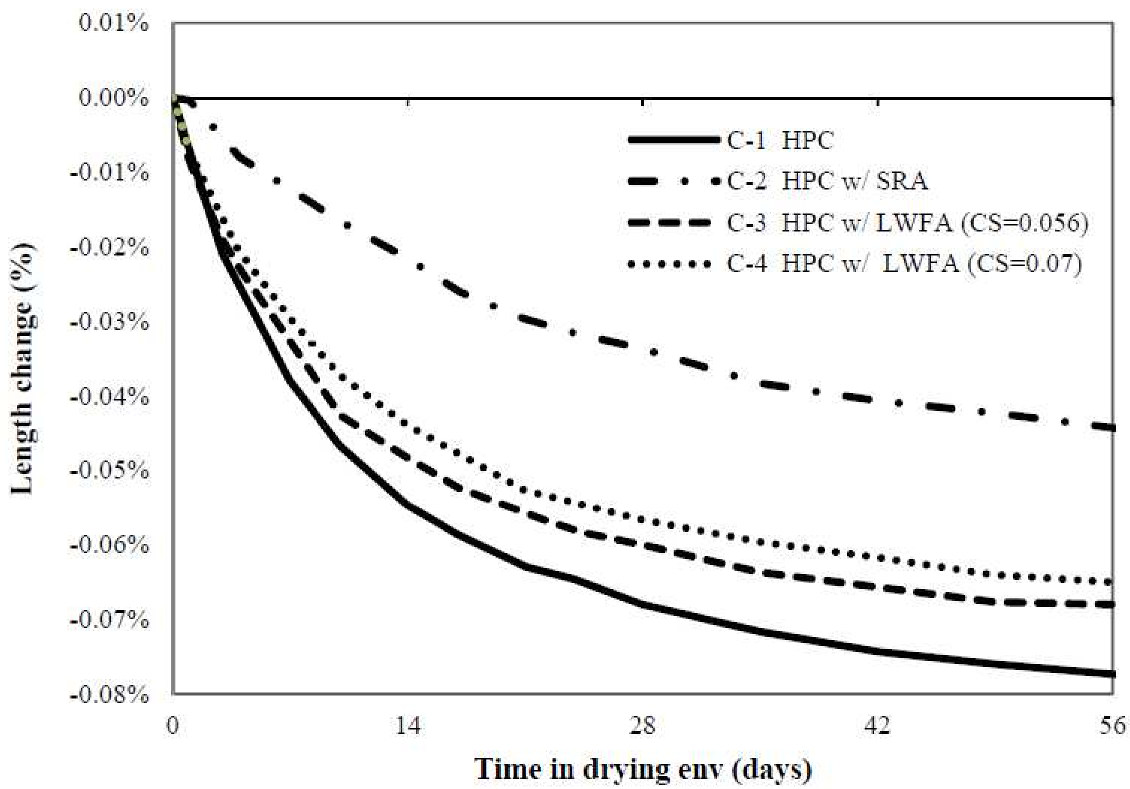

Figure 3.4 Drying shrinkage results, 1 day cure

Further studies on long-term performance of LWFA on reducing drying shrinkage need to be performed. In order to accurately determine how effective pre-wetted LWFA is at reducing drying shrinkage, autogenous shrinkage testing on similar concrete mixtures needs to be performed. The difference between drying shrinkage results of the control HPC mixture and the mixtures incorporating pre-wetted LWFA presented in this paper could be attributed to the pre-wetted LWFA reducing the amount of autogenous shrinkage at early ages, but not providing any additional benefit for drying shrinkage. To provide protection against drying shrinkage, additional LWFA may need to be added to the mixture.

3.7 CONCLUSIONS AND RECOMMENDATIONS

This research identified a simple procedure to predict the long-term chemical shrinkage value for high performance cementitious systems containing SCMs and/or SRAs. A recommended procedure to determine the ultimate CS value was proposed by using a predictive equation combined with experimental testing. Suggested improvements to the ASTM C 1608 standard test method can be used effectively and efficiently to estimate long-term shrinkage for portland cement systems with SCMs and/or SRAs. The recommended improvements include:

- An image acquisition and analysis system based on ASTM C1608 standard chemical shrinkage test was applied in the research, which provided an improved chemical shrinkage development curve with less variance in the data.
- The mass change was checked for each sample before and after the test. In order to ensure the validity of the test, samples (complete pipette-vial setup) with a mass change more than 0.02 g was considered to be non-compliant.
- The test was conducted for 14 days to obtain sufficient data for a long-term chemical shrinkage prediction model.
- A sample thickness of 3 mm is recommended for all long-term chemical shrinkage investigations. This was confirmed through a depercolation study where samples of different thickness (3 mm, 5 mm, 7 mm and 10 mm) were tested; the results showed that the chemical shrinkage decreased with an increase in sample thickness. This confirmed depercolation in sample thicknesses greater than 3 mm.
- It is recommended to use a prediction model (previously developed) to estimate long-term chemical shrinkage when time tends to infinity (e.g. representative of full hydration and the greatest internal water demand).
- Based on this model, a modification to the original Bentz Equation was suggested by replacing ($CS \cdot \alpha_{max}$) with predicted long-term chemical shrinkage (CS_{∞}).

Therefore the complication of identifying ultimate chemical shrinkage and maximum degree of hydration in cementitious systems with SCMs and/or SRAs can be avoided.

Using the modifications outlined above, concrete prisms were cast according to ASTM C 192 and then tested in a modified ASTM C157 test (reduced external cure). It was shown that:

- Incorporation of LWFA (25% replacement for normal weight sand), reduced the drying shrinkage by 15% at 56 days.
- Incorporation of SRA was the most effective in reducing drying shrinkage with a reduction of 42% at 56 days.

3.8 ACKNOWLEDGEMENTS

The authors would like to thank the Oregon Department of Transportation for financial support for this research effort.

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4 MANUSCIPT 2

Prediction of Drying Shrinkage for Internally Cured High Performance Concrete

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SP 290 The Economics, Performance and Sustainability of Internally Cured Concrete

American Concrete Institute

ACI Symposium Publication 290, 2012

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4.1 SYNOPSIS

In this research, ten different high performance concrete (HPC) mixtures internally cured by pre-wetted lightweight fine aggregate (LWFA) and/or shrinkage reducing admixture (SRA) were cast and their drying shrinkage strain was monitored using the ASTM C157 test. The data collected was used to evaluate six shrinkage prediction models, namely, ACI 209 model, CEB90 model, AASHTO model, B3 model, GL2000 model and ALSN model. The study finds that the GL2000 model shows the best overall performance in predicting shrinkage strain for internally cured HPC. However, more accurate long-term shrinkage prediction can be achieved based on the current ACI 209 model with experimental measurements. This proposed procedure is capable to predict long-term drying shrinkage for concrete using local materials mixture by using short-term experimental measurements.

4.2 INTRODUCTION

Drying shrinkage is regarded as a major cause contributing to the complex cracking issue in concrete. In 2006, the Federal Highway Administration reported that 12% (72,500 out of 599,976) of the country's bridges in the National Highway System were considered structurally deficient, which refers to bridges having major deterioration, cracks, or other deficiencies in their structural components including decks, girders, or foundations (*Federal Highway Admistration 2006*). According to a survey conducted by Krauss and Rogalla in 1996, 62% of respondents in the state departments of transportation (DOTs) believed transverse cracking was a significant problem, and more than 100,000 bridges decks had suffered from transverse cracking, which is a pattern indicating the presence of drying shrinkage (*Krauss and Rogalla 1996*). However, there are many factors which can lead to cracking in concrete bridge decks, such as dimensional stabilities (shrinkage and creep), environment fluctuations and restraint conditions. Drying shrinkage refers to the volume decrease over time due to moisture loss to the surrounding environment. It is affected by many factors, such as cement properties, quality of aggregate, size and grading of aggregate, water to cement ratio (w/cm) as well as water content, relative humidity, chemical admixtures, duration of curing and the size of the concrete specimen (*Huo and Wong 2000*). A comprehensive summary of factors affecting shrinkage of hardened concrete can be found in literature (*ACI Committee 209 2005*).

Research has shown that a greater potential of cracking due to the combination of drying shrinkage, autogenous shrinkage and plastic shrinkage exists in modern high performance concrete (HPC), which is usually comprised of supplementary cementitious materials (SCMs) and has a low w/cm below 0.40 (*Holt 2001; Bentz and Jensen 2004*). To reduce the effect of shrinkage, internal curing using pre-wetted lightweight fine aggregates (LWFA) has been found effective both in the laboratory research and field applications in the last decade (*Lura 2003; Villarreal and Crocker 2007; Delatte et al. 2008; Friggle and Reeves 2008; Lopez et al. 2008; Villarreal 2008; Paul and Lopez 2011; Slatnick et al. 2011*). Similarly, shrinkage reducing admixtures (SRAs) have also been found to be successful in reducing cracking potential due to reductions in autogenous and drying shrinkage (*Shah et al. 1992; Tazawa and Miyazawa 1995; Folliard and Berke 1997; Bentz et al. 2001; Bentz and Jensen 2004; Bentz 2005; Rongbing and Jian 2005; Bentz 2006; Slatnick et al. 2011*). More information and a list of literatures can be found in ACI SP-256 (*Mohr and Bentz 2008*), a compilation specifically focused on internal curing of HPC.

Currently there are various models to predict the long-term drying shrinkage of concrete. However, there has not been any previous research done on predicting the efficacy of these models on internally cured HPC. Six existing drying shrinkage prediction models were evaluated in this paper to evaluate their ability to predict the drying shrinkage of internally cured HPC. The models analyzed in this paper were: ACI 209 model (*ACI Committee 209 2008*), CEB90 model (*Comité euro-international du béton 1993*), AASHTO model (*AASHTO 2007*), B3 model (*Bažant and Baweja 2000*), GL2000 model (*Gardner and Lockman 2001*), and ALSN model (*Al-Manaseer and Ristanovic 2004*).

The ACI 209 model is predominately used in the United States, and has been incorporated into many of the building codes. This model was developed empirically and is based on drying shrinkage data obtained prior to 1968. The equations can be used to predict the drying shrinkage of normal weight, sand lightweight and all lightweight concretes.. A detailed description of the method can be found in ACI Committee 209 report (*ACI Committee 209 2008*).

European code specifies the prediction of drying shrinkage using the method developed in 1990 by the Comité Euro-International du Béton (CEB) (*Comité euro-international du béton 1993*). The CEB90 model was derived using mathematical functions rather than strictly empirical data, and has been optimized from information from a data bank of normal weight plain structural concrete performance. It is not clearly stated weather the model can be applied to internally cured HPC, however, the A detailed description and guidance for this model can be found in the CEB 1990 code, section 2.1.6.4.4 (*Comité euro-international du béton 1993*).

The AASHTO model of determining shrinkage, specified in AASHTO LRFD Bridge Design Specifications (*AASHTO 2007*) Article 5.4.2.3.3, was developed by Huo et al. (*Huo et al. Decermber, 2001*), All-Omaishi (*Al-Omaishi 2001*) and Tadros et al. (*Tadros et al. 2003*) based on the ACI 209 model. This model is derived from the study of prestress losses in high strength concrete.

The B3 model developed by Bažant and Baweja is a better theoretically justified model than the rest models. It is based on “a systematic theoretical formulation of the basic physical phenomena involved”, and it was “calibrated by a computerized data bank comprising practically all the relevant test data obtained in various laboratories throughout the world” (*Bažant and Baweja 2000*). Bažant and Baweja state the coefficient of variations for the B3 model are much lower than the CEB 90 model and the ACI 209 model.

The GL 2000 was developed by Gardner and Lockman (*Gardner and Lockman 2001*). Several minor coefficients have been modified in the latest version (*Gardner and Tsuruta Septermber, 2004*). This model is effective at predicting shrinkage in normal strength concrete with a 28-day compressive strength less than 82 MPa and a w/cm ratio ranging from 0.4 to 0.6. Gardner and Lockman stated that the GL2000 method can be used to accurately predict the shrinkage regardless of which admixtures, mineral by-products, curing regime or casting temperature are employed (*Gardner and Lockman 2001*). This is realized by tracking concrete strength development with time, and measuring modulus of elasticity. Then, the concrete stiffness is taken into account thus the model can be applied to internally cured concretes. However, one assumption in the model is that the shrinkage decreases with the increase of strength and modulus elasticity. This is not true for the incorporation of SRAs, which significantly reduces the shrinkage and can adversely affect the strength slightly.

The ALSN model was proposed by Al-Manaseer and Ristanovic (*Al-Manaseer and Ristanovic 2004*). The model works exactly the same as the GL2000 model, expect that a coefficient was added to take the influence of SRAs into account. Thus these two models are evaluated at the same time for concrete mixtures without SRAs, and mentioned as GL2000/ALSN model in the following figures and tables. The ALSN model is the only model which is designed to predict the shrinkage of concrete containing a SRA dosage between 0 and 2.5% based on mass of total cementitious materials (*Al-Manaseer and Ristanovic 2004*). Due to above-mentioned reason, only data of concrete mixtures with SRAs (Mix 1-SRA and Mix 3C-SRA) are used to evaluate this model.

With all different models at hand, it is critical to choose a proper model to predict shrinkage for concrete using local materials. A major concern for each model is that whether the data source used to develop the model is representative of all concretes, such as concrete mixtures with SRAs. With the presence of SRAs in concrete, the shrinkage is reduced significantly, thus most existing models are unable to predict shrinkage in these types of concretes. Another example would be portland pozzolan cement concrete, which has been widely used in some countries (*Videla et al. 2004*). ACI 209 committee states in the 209.2R-08 report that the average ultimate shrinkage value along with correction factors should be used only in the absence of specific shrinkage data for local aggregates and conditions. The report also recommends that to perform sensitivity analysis in selecting a proper model and to carry out short-term testing to calibrate the models to improve prediction. However, long-term shrinkage data is usually not readily available, especially with novel materials or admixtures. On the other hand, there are no set rules on how to use short-term testing to calibrate the model or to predict long-term performance. Additionally, very little work has been done to deal with this issue.

To solve this dilemma, Videla et al. (*Videla et al. 2004*) proposed a methodology to update prediction models when different materials were used compared to those used to develop the current prediction models. They included a correction factor applied to an ultimate shrinkage value, and a correction time function applied to shrinkage development. An experimental program was designed and carried out to derive a modified CEB90 model which enabled an accurate shrinkage prediction for concrete made with locally available materials. This research provided a feasible example on how to modify and utilize an established shrinkage prediction model to fit the local materials. However, they concluded that to achieve estimation with 30% or less coefficient of variation, the minimum testing time required for 75×75×280 mm and 100×100×500 mm sample size are 100 and 170 days, respectively.

A simple alternative procedure based on the ACI 209 model is proposed. It allows the prediction of long-term shrinkage strain using short-term experimental measurements. The reliability of proposed procedure was also discussed. In addition, free shrinkage data collected from 10 different HPC mixtures was compared to calculated shrinkage strains using all six prediction models. The appropriateness of using each model for concrete containing lightweight fine aggregate (LWFA) and/or SRAs was examined.

4.3 RESEARCH SIGINIFICANCE

Since concrete durability is closely related to effects of shrinkage, it is important to develop proper prediction models. The ACI 209 model is recommended by American Concrete Institute and widely used in the U.S. for normal strength concretes using conventional aggregates. It recommends to perform short-term testing on concrete to calibrate the model to improve predictions for local materials (*ACI Committee 209 2008*). However, the calibration procedure is not clearly stated in the document. The significance of this research is to propose a procedure based on the ACI 209 shrinkage model to predict long-term shrinkage strain using short-term experimental measurements. In addition, evaluation of the accuracy of six existing shrinkage models is reported compared to the authors' experimental data. These models are the ACI 209 model, the CEB90 model, the AASHTO model, the B3 model, the GL2000 model, and the ALSN model. The shrinkage values determined by each model are compared against the experimental results from 10 high-performance concrete mixtures with incorporation of LWFAs and/or SRAs.

4.4 EXPERIMENTAL

The experimental program was designed to investigate the effect of LWFA and/or SRAs on reducing drying shrinkage in HPC. Two different types of LWFAs (one expanded shale and one expanded clay) and one SRA were incorporated into the standard local DOT HPC mixture, which contains 30% class F fly ash and 4% silica fume replacement by weight of cement. Drying shrinkage was monitor using the ASTM C157 test up to 180 days; compressive strength tests (ASTM C39) and modulus of elasticity tests (ASTM C469) was also performed at 28 days.

4.4.1 **Materials and Mixture Proportions**

The cement and SCMs used in this research were an ASTM C150 type I/II cement, an ASTM C618 class F fly ash, and an ASTM C1240 silica fume containing nearly pure silica dioxide in a noncrystalline form with approximately 1% crystalline silicon.

Local siliceous river gravel and natural siliceous river sand were used in all concrete mixtures. The maximum size of the river gravel was 19 mm (3/4 in). The LWFAs used met ASTM C330 specifications. The absorption test (ASTM C128 cone test) was performed and the results showed the absorption capacity of the two LWFAs were 17.5% and 34.1% for expanded shale and clay, respectively. A desorption test (modified ASTM C1498) showed that more than 97% of the LWFA absorbed moisture was released by the point when an external relative humidity (RH) of 84% was obtained. This was true for both LWFA, regardless of their composition. More detailed information about the LWFAs can be found in another reference by one of the authors (*Deboodt 2011*).

Table 4.1 shows a summary of the 10 mixture proportions investigated in this research. All mixing was conducted according to ASTM C192. The 28-day compressive strength was targeted at 34.5 MPa. A w/cm of 0.37 was used in all mixtures. The mixtures contained 375 kg/m^3 of cementitious materials, including cement, fly ash and silica fume. The SRA was added at 2% of the total cementitious materials by mass, as an equal mass replacement of mixing water. A high-range water reducing admixture (HRWR) was adjusted and used to ensure uniform workability with similar slumps of 100 to 150 mm among all mixtures. An air entraining admixture was also used to ensure proper free/thaw resistance specified by the local DOT, and had a target air content of 5% to 7%. The standard LWFA replacement level was determined using a previously published equation (*Bentz et al. 2005; Fu et al. 2011*), which was 164 kg/m^3 for the shale LWFA and 77 kg/m^3 for the clay LWFA, which would provide the same amount of internally available water in the reservoir of two types of LWFA based on their different absorption capacities, provided above. In addition, two additional replacement levels, approximately

60% and 80% of the standard level, were investigated to determine the effectiveness of the LWFA. A full replacement of normal weight fine aggregate by the shale LWFA was also studied for a maximum effectiveness. The effect of the combination of LWFA and SRA was also studied with the LWFA shale mixture with 2% SRA. All LWFA was pre-wetted at least 24 hours prior to mixing to ensure their absorption capacity was reached.

Table 4.1 Curve fit and predicted long-term chemical shrinkage values

Mixture	Note	Cement (kg/m^3)	Fly Ash (kg/m^3)	Silica Fume (kg/m^3)	Water (kg/m^3)	Coarse Aggregate (kg/m^3)	Fine Aggregate (kg/m^3)	LWFA (kg/m^3)	SRA (kg/m^3)	AE (mL/m^3)	HRWR (mL/m^3)
1	Control	248	112	14.8	139	1074	659	0	0	61	905
1-SRA	w/ SRA	248	112	14.8	131	1074	659	0	7.53	451	1030
2A	Low Clay	248	112	14.8	139	1074	556	45	0	61	900
2B	Medium Clay	248	112	14.8	139	1074	518	62	0	56	880
2C	Standard Clay	248	112	14.8	139	1074	482	77	0	41	900
3A	Low Shale	248	112	14.8	139	1074	512	93	0	56	800
3B	Medium Shale	248	112	14.8	139	1074	452	131	0	54	900
3C	Standard Shale	248	112	14.8	139	1074	400	164	0	52	860
3D	Full Shale	248	112	14.8	139	1074	0	466	0	104	920
3C-SRA	Standard Shale w/ SRA	248	112	14.8	131	1074	400	164	7.53	133	945

4.4.2 **ASTM C157 Prism Test**

Free shrinkage was monitored using the ASTM C157 prism test, which utilizes a 75×75×280 mm (3×3×11.25 in) concrete prism. The curing duration was modified from 28 days as specified in the standard (*2008*) to better represent actual field exposure conditions. For all mixtures, three prisms for each mixture and specified curing duration were cast and cured for 1, 3, 7, 10, or 14 days, except for mixtures 3D and 3C-SRA for which curing time of 1, 7 and 14 days were applied, totaling 138 drying shrinkage prisms.

The specimens were cast and sealed in molds using wet burlap and plastic sheeting to protect against moisture loss until demoldling at 24 hours. Then the prisms were transferred to a fog room for curing. Upon reaching their specified curing duration, the prisms were moved to an environmental chamber which maintains a drying environment of 23±2 °C and 50±4% relative humidity (RH), and initial length of each prism was recorded. The length change and mass loss were monitored up to 180 days from the initiation of drying. A brief summary of testing results is shown in Table 4.2.

Table 4.2 Summary of free shrinkage testing results (up to 180 days) and ultimate shrinkage strain predicted. Environmental chamber condition: 23±2 °C 50% RH

Mixture	Curing period (day)	Measured shrinkage strain (μm/m) at time (number of days) from initiation of drying							Predicted ultimate shrinkage (μm/m)					
		7 day	14 day	28 day	56 day	90 day	120 day	180 day	ACI209	CEB90	AASHTO	B3	GL2000 /ALSN	ALSN (SRA only)
	1	-344	-484	-637	-764	-811	-832	-868	-912					
1	7	-307	-444	-564	-660	-724	-750	-779	-725	-578	-673	-431	-753	-
	14	-300	-427	-540	-660	-725	-745	-776	-668					
	1	-210	-327	-467	-577	-631	-658	-707						
1-SRA	7	-160	-244	-400	-530	-578	-622	-654	-	-	-	-	-	-437
	14	-160	-260	-384	-494	-507	-613	-660						
	1	-420	-574	-717	-800	-847	-871	-894	-912					
2A	7	-307	-457	-610	-744	-773	-798	-837	-725	-666	-956	-456	-936	-
	14	-297	-454	-627	-730	-772	-792	-834	-668					
	1	-317	-477	-610	-690	-730	-751	-807	-912					
2B	7	-240	-400	-557	-647	-700	-731	-770	-725	-634	-827	-445	-854	-
	14	-207	-350	-514	-620	-680	-705	-751	-668					
	1	-317	-454	-560	-637	-696	-706	-757	-912					
2C	7	-254	-394	-530	-630	-692	-729	-743	-725	-569	-653	-429	-740	-
	14	-237	-370	-510	-620	-674	-716	-729	-668					
	1	-277	-394	-527	-600	-639	-660	-750	-912					
3A	7	-271	-417	-550	-654	-693	-726	-744	-725	-547	-609	-425	-711	-
	14	-244	-377	-520	-617	-676	-703	-760	-668					
	1	-327	-484	-600	-681	-723	-741	-784	-912					
3B	7	-264	-400	-580	-660	-713	-738	-799	-725	-613	-762	-440	-812	-
	14	-260	-394	-544	-654	-707	-750	-790	-668					
	1	-304	-440	-567	-651	-690	-713	-757	-912					
3C	7	-247	-397	-534	-647	-714	-745	-798	-725	-579	-675	-431	-775	-
	14	-251	-384	-544	-644	-702	-756	-791	-668					
	1	-250	-420	-517	-618	-667	-686	-697	-912					
3D	7	-207	-317	-454	-580	-654	-678	-703	-725	-511	-548	-419	-668	-
	14	-147	-230	-370	-514	-594	-630	-663	-668					
3C-	1	-167	-267	-387	-507	-527	-563	-620	-	-	-	-	-	-426

SRA	7	-130	-210	-357	-477	-522	-553	-590
	14	-107	-197	-320	-447	-499	-527	-577

The predicted ultimate shrinkage strains are also given in Table 4.2 for each model. For simplicity, results of 3 and 10 day curing was not displayed in this table, however all data were included in the evaluation. Generally, longer curing time resulted in lower shrinkage at the early age, but did not significantly affect shrinkage at the later age (180 day). In addition, higher replacement levels of pre-wetted LWFA exhibited more benefit in terms of reducing shrinkage. The incorporation of SRAs was more effective to reduc the shrinkage compared to the LWFA. And the combination of LWFA and SRA further reduced shrinkage. A more detailed discussion on the testing results is beyond the scope of this paper, and can be found in another publication by one of the authors (*Deboodt 2011*).

4.4.3 **Mechanical Properties**

In addition to the free drying shrinkage test, the compressive strength and modulus of elasticity were also measured at 28 days. Concrete cylinders measuring Φ75×150 mm (Φ3×6 in) were cast, cured and tested according to ASTM C39 and C469. The summary of the test results is listed in Table 4.3. The measured compressive strengths were used in all shrinkage prediction models.

Table 4.3 Mechanical properties of concrete cylinders at 28 day

Mixture	1	1-SRA	2A	2B	2C	3A	3B	3C	3D	3C-SRA
Compressive Strength (Mpa)	36.8	33.245	23.84	28.65	38.1	41.35	31.67	36.64	46.77	35.01
Modulus of Elasticity (Gpa)	37.3	31.5	28.2	28.7	28.8	32.7	24.5	27.2	-	-

4.5 EVALUTION OF PREDICTION MODELS

For each prediction model, certain criteria apply as well as different input factors. For the ACI 209 model, the CEB90 model, the B3 model and the GL2000 model, a thorough summary of criterion and input factors with a numeric example can be found in an ACI Committee 209 report (*ACI Committee 209 2008*). More details about AASHTO model is outlined in literature (*AASHTO 2007*) and as for the ALSN model (*Al-Manaseer and Ristanovic 2004*).

To evaluate the accuracy of the prediction models, five methods can be used, including the residual method, the B3 coefficient of variation method, the CEB coefficient of variation method, the CEB mean square error method, and the CEB mean deviation method. Detailed descriptions of these methods have been summarized by Al-Manaseer and Lam (*Al-Manaseer and Lam 2005*) and ACI Committee 209(*ACI Committee 209 2008*) . The CEB mean square error (F_{CEB}%) method was arbitrarily selected in this study:

$$f_j = \frac{(Cal\ X_{ij} - Obs\ X_{ij})}{Obs\ X_{ij}} \times 100 \tag{4.1}$$

$$F_i = \sqrt{\frac{1}{n-1} \sum_{j=1}^{n} f_j^2} \tag{4.2}$$

$$F_{CEB} = \sqrt{\frac{1}{N} \sum_{i=1}^{N} F_i^2} \tag{4.3}$$

Where *Cal* X_{ij} = predicted shrinkage strain at time j of experiment i; *Obs* X_{ij} = experimental shrinkage strain at time j of experiment i; f_j = percent difference between calculated and observed data point j; F_i = mean square of residuals, %; F_{CEB} = mean square error, %; n = total number of values j of experiment i considered at a fixed time; and N = total number of data sets considered.

Figure 4.1 to 4.6 show the comparisons between the experimental data and the calculated data from each prediction model. The solid diagonal line in each figure represents perfect correlation between the measured value and calculated value for each model respectively. Two other reference lines (±40% of measured value) are added in all the figures to show the relative accuracy of each prediction models. All models, except the GL2000, underestimate the shrinkage strain, especially at the later age.

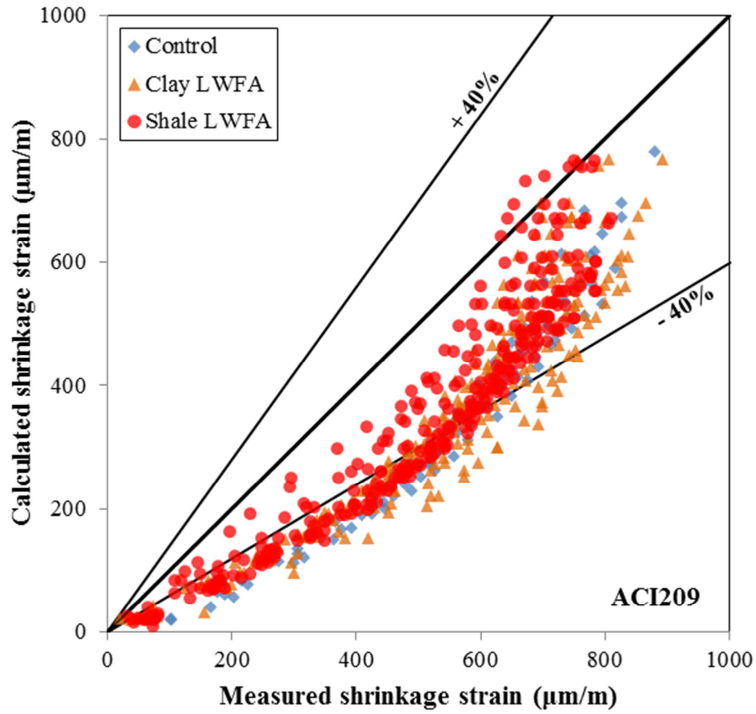

Figure 4.1 Model evaluation, ACI 209

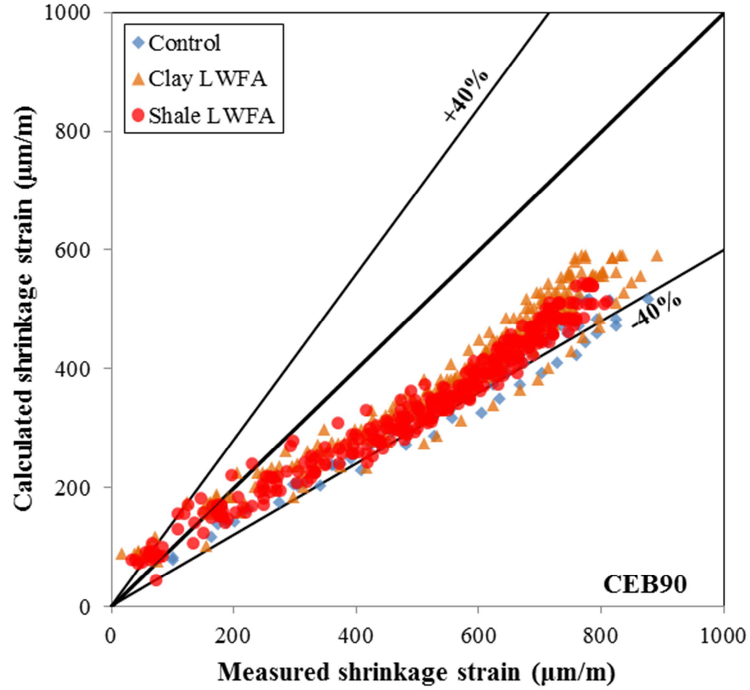

Figure 4.2 Model evaluation, CEB90

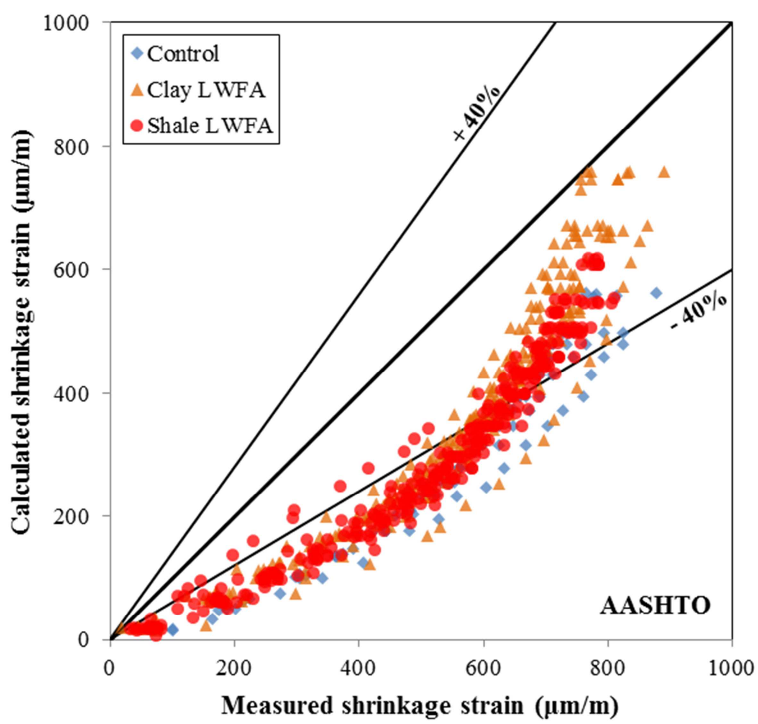

Figure 4.3 Model evaluation, AASHTO

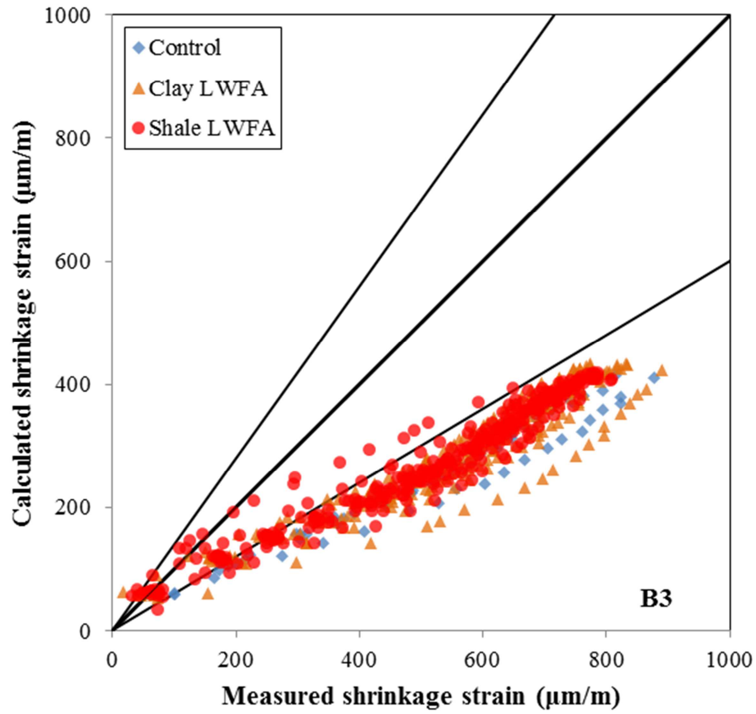

Figure 4.4 Model evaluation, B3

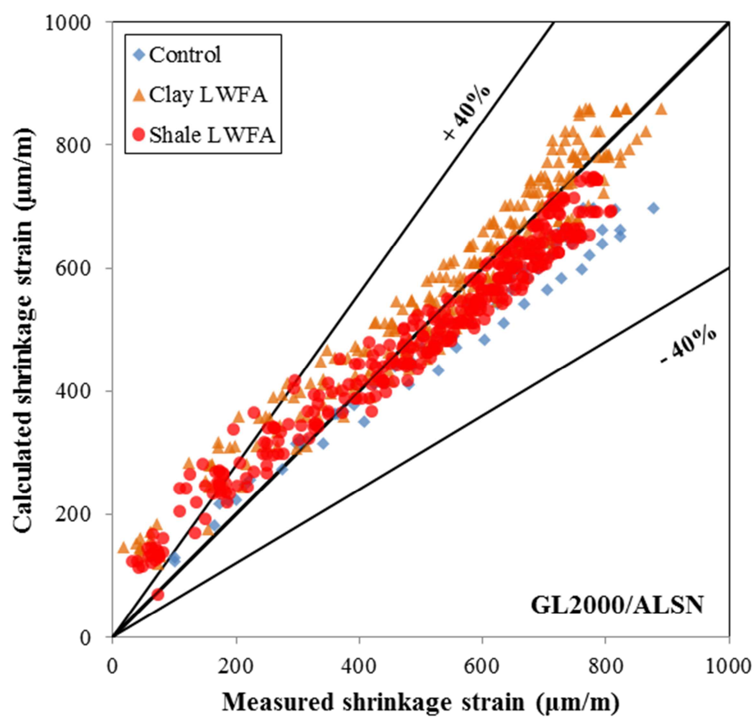

Figure 4.5 Model evaluation, GL2000/ALSN

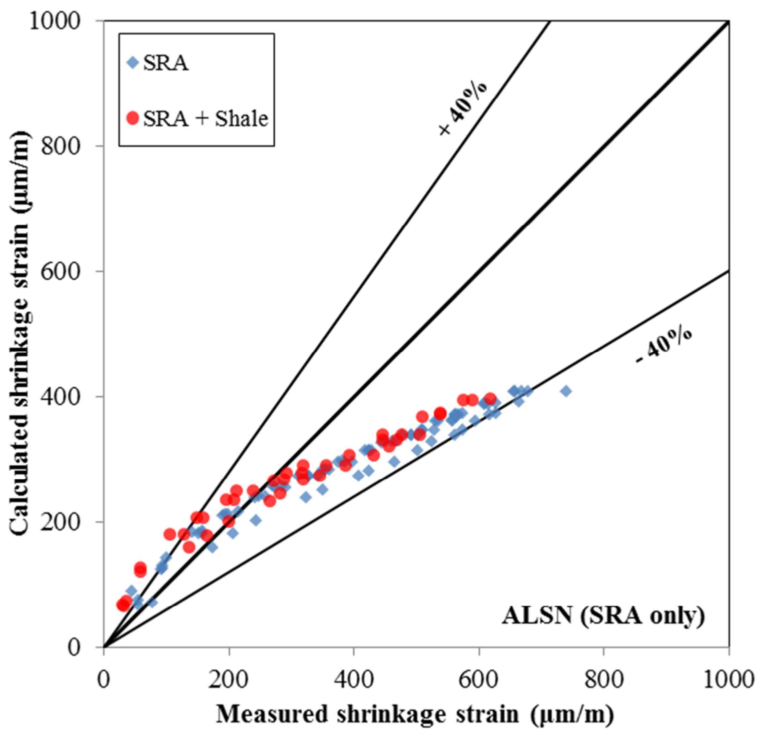

Figure 4.6 Model evaluation, ALSN

In addition, a summary of calculated mean square error is given in Table 4.4. For concrete without SRAs, the results show that the GL2000 model (F_{CEB} = 20%) performed best in this research, followed by the CEB90 model (F_{CEB} = 38%) and the ACI 209 model (F_{CEB} = 42%), while the B3 model (F_{CEB} = 50%) and AASHTO model (F_{CEB} = 49%) show the largest variation. All models show similar performance among HPC control mixtures and HPC with LWFA mixtures, except the GL2000 model which gives a better prediction for HPC control mixtures (F_{CEB} = 13%) than HPC with LWFA (F_{CEB} = 20%).

Table 4.4 Summary of mean square error (F_{CEB}%) of different models

		ACI 209	CEB90	AASHTO	B3	GL2000/ALSN	ALSN
F_{CEB}% for each group	CONTORL	42	38	55	53	13	
	CLAY	40	40	45	51	20	
	SHALE	38	35	50	47	21	
	SRA						31
	SRA+SHALE						27

F_{CEB}% overall	42	38	49	50	20	29

There are noticeable slope changes in Figure 4.1and Figure 4.3, in a similar pattern. This occurred in the ACI 209 model and AASHTO model. These trends occurred because these three models work better to calculate long-term shrinkage strains, and share a similar time function "f(t)= t/(t+a)". Meanwhile the B3 model (Figure 4.4) uses a different time function "f(t) = tanh (t/a)", and the CEB90 model (Figure 4.2), the GL2000 model (Figure 4.5) and the ALSN model (Figure 4.6) use "f(t)= sqrt((t/t/+a))". To some extent, the B3 and the GL2000 model describe the time dependence better since the slopes in those figures remain quite constant. The errors of these three models mostly come from the discrepancy between estimated ultimate shrinkage strain and measured shrinkage strain.

4.6 PROPOSED PROCEDURE BASED ON ACI 209 MODEL

Closer attention was given to the ACI 209 model in this paper. Although directly applying the model did not yield the most favorable accuracy according to the experimental data in this research, the authors believed that this model has a potential to capture any given hyperbolic-like drying shrinkage curve, regardless of the properties of constituents and admixtures. The current ACI 209 model is given as:

$$\varepsilon_{sh}(t, t_c) = \frac{(t-t_c)^{\alpha}}{f+(t-t_c)^{\alpha}} \cdot \varepsilon_{shu} \tag{4.4}$$

$$\varepsilon_{shu} = 780\gamma_{sh} \times 10^{-6} mm/mm\ (in/in) \tag{4.5}$$

Where $\varepsilon_{sh}(t,\ t_c)$ = shrinkage strain at concrete age t since the start of drying at age t_c, mm/mm (in/in); ε_{shu} = ultimate shrinkage strain, mm/mm (in/in); α, f = constants defining the shape of time-dependent curve; γ_{sh} = the cumulative product of the applicable correction factors including initial moist curing duration, ambient relative humidity, size of the drying specimen in terms of the volume-surface ratio, and fresh concrete properties (i.e. slump, fine aggregate factor, cement content, and air content).

It is noted that for simplification an average value of 1.0 was suggested for constant α, representing a flatter hyperbolic form. However, the specific mathematical form of Eq.(4.4) is able to capture the time-dependent characteristic of a drying shrinkage curve, which starts from the origin and converges at an asymptote. To manipulate (curve fitting) three parameters (ε_{sh} , α, and f), Eq.(4.4) is able to describe any “drying-shrinkage-like” hyperbolic-like curves with high accuracy ($R^2 > 0.99$). A non-linear Levenberg-Marquardt least squares fitting tool was used in curve fitting. In most cases, the curve fitting tool is applied to a set of drying shrinkage data, and then the three curve fitting parameters are stable after 10 to 20 iterations. A similar procedure has been successfully used in predicting long-term chemical shrinkage (*Xiao et al. 2009; Fu 2011; Fu et al. 2011*).

To determine the minimum required testing duration, a sensitivity study was conducted using the proposed prediction model. Table 4.5 shows a summary of the predicted ultimate drying shrinkage strain using experimental measurements. The sensitivity study was performed to determine a minimum testing period for different concrete mixtures in order for the prediction model to be valid. It should be noted that for concrete without SRA, after approximately 50 days from initiation of drying, the predicted ultimate values are stable. Also, there is an assumption that the prediction from data recorded from a longer testing period would yield a more accurate ultimate shrinkage strain value. Most ultimate values (except Mix 3A-3 day cure) derived from data up to 50 day are around 5% when comparing to the ultimate value derived from 180 day data. This is acceptable and within the coefficient of variation of other testing parameters. For concrete with the incorporation of SRA, it was observed that a longer measurement period was needed due to delayed shrinkage development. A testing period of 84 day was selected as a cut-off date for Mix 1-SRA and Mix 3C-SRA.

Figure 4.7 shows the comparison between experimental measurements, the improved ACI 209 model, and the GL2000/ALSN model. One mixture and curing regime was randomly selected to represent a group of test, which are the control HPC, HPC with LWFA, HPC with SRA, and HPC with SRA and LWFA. It showed that the proposed procedure better predicts the shrinkage for all of the concrete mixtures when compared to the GL2000 model. Therefore, the conclusion can be drawn that it is possible to obtain a stable and more accurate ultimate shrinkage stain. The minimum 50 day testing duration works well for the control HPC and the internally cured HPC with pre-wetted LWFA. A longer testing duration of 84 days was selected to predict shrinkage strain of concrete with the incorporation of SRA. To apply this method to local or novel concrete, it is recommended that an individual cut-off date should be chosen. The proposed procedure is briefly summarized as follow:

Table 4.5 Sensitivity study using measurement up to different age to predict ultimate shrinkage
(Blocked area indicates selected cut-off date, NC-non converging)

Mixture	Curing period (day)	Number of days from initiation of drying										Difference between selected cut-off date and 180 day
		28	35	42	49	56	70	84	98	120	180	
1	1	-0.00091	-0.00090	-0.00094	-0.00092	-0.00094	-0.00094	-0.00094	-0.00094	-0.00094	-0.00094	-2.1%
	7	-0.00106	-0.00097	-0.00093	-0.00089	-0.00087	-0.00087	-0.00087	-0.00089	-0.00088	-0.00087	2.3%
	14	-0.00109	-0.00105	-0.00100	-0.00095	-0.00097	-0.00098	-0.00099	-0.00095	-0.00093	-0.00091	4.4%
1-SRA	1	NC	-0.00166	-0.00113	-0.00102	-0.00096	-0.00086	-0.00082	-0.00079	-0.00079	-0.00080	-1.3%
	7	NC	-0.00346	-0.00152	-0.00128	-0.00115	-0.00096	-0.00083	-0.00078	-0.00079	-0.00076	2.6%
	14	NC	-0.00285	-0.00168	-0.00130	-	-0.00086	-0.00078	-0.00078	-0.00079	-0.00079	-1.3%
2A	1	-0.00116	-0.00107	-0.00101	-0.00100	-0.00097	-0.00097	-0.00095	-0.00095	-0.00095	-0.00095	5.3%
	7	-0.00093	-0.00093	-0.00094	-0.00092	-0.00094	-0.00091	-0.00089	-0.00086	-0.00095	-0.00087	5.7%
	14	-0.00091	-0.00098	-0.00091	-0.00089	-0.00087	-0.00086	-	-	-0.00084	-0.00086	3.5%
2B	1	-0.00084	-0.00083	-0.00083	-0.00082	-0.00081	-0.00079	-0.00078	-0.00079	-0.00079	-0.00082	0.0%
	7	-0.00091	-0.00083	-0.00080	-0.00076	-0.00076	-0.00075	-0.00076	-	-0.00078	-0.00080	-5.0%
	14	-0.00076	-0.00073	-0.00073	-0.00072	-0.00071	-0.00072	-0.00074	-0.00074	-0.00075	-0.00078	-7.7%
2C	1	-0.00084	-0.00082	-0.00082	-0.00080	-0.00079	-0.00080	-0.00078	-0.00079	-0.00078	-0.00082	-2.4%
	7	-0.00091	-0.00085	-0.00080	-0.00079	-0.00078	-0.00078	-0.00078	-0.00079	-0.00081	-0.00080	-1.3%
	14	-0.00073	-0.00075	-0.00074	-0.00075	-0.00075	-0.00077	-0.00076	-0.00077	-0.00077	-0.00078	-3.8%
3A	1	-0.00068	-0.00070	-0.00069	-0.00069	-0.00069	-	-0.00069	-0.00069	-0.00069	-0.00076	-9.2%
	7	-0.00089	-0.00086	-0.00082	-0.00080	-0.00081	-0.00079	-0.00078	-0.00079	-0.00076	-0.00079	1.3%
	14	-0.00088	-0.00082	-0.00081	-0.00080	-0.00078	-0.00078	-0.00078	-0.00078	-0.00078	-0.00082	-2.4%
3B	1	-0.00080	-0.00081	-0.00081	-0.00081	-0.00081	-0.00079	-0.00079	-0.00078	-0.00078	-0.00081	0.0%
	7	-0.00137	-0.00097	-0.00088	-0.00084	-0.00082	-0.00081	-0.00081	-0.00080	-0.00080	-0.00083	1.2%
	14	-0.00092	-0.00090	-0.00086	-0.00085	-0.00084	-0.00084	-0.00084	-0.00084	-0.00085	-0.00086	-1.2%
3C	1	-0.00084	-0.00080	-0.00078	-0.00077	-0.00077	-0.00076	-0.00076	-0.00076	-0.00076	-0.00079	-2.5%
	7	-0.00090	-0.00090	-0.00090	-0.00085	-0.00083	-0.00081	-0.00081	-0.00082	-0.00083	-0.00086	-1.2%
	14	-0.00087	-0.00088	-0.00084	-0.00084	-0.00083	-0.00083	-0.00083	-0.00083	-0.00087	-0.00088	-4.5%
3D	1	-0.00116	-0.00107	-0.00101	-0.00100	-0.00097	-0.00097	-0.00095	-0.00095	-	-0.00095	5.3%
	7	-0.00093	-0.00093	-0.00094	-0.00092	-0.00094	-0.00091	-0.00089	-0.00086	-	-0.00087	5.7%
	14	-0.00091	-0.00098	-0.00091	-0.00089	-0.00087	-0.00086	-	-	-	-0.00086	3.5%
3C-SRA	1	-0.00117	-0.00117	-0.00094	-0.00080	-0.00085	-	-0.00069	-0.00065	-0.00065	-0.00069	0.0%
	7	NC	-0.00157	-	-0.00079	-0.00077	-	-0.00067	-0.00066	-0.00065	-0.00065	3.1%
	14	NC	-	-	-	-0.00082	-	-0.00067	-0.00067	-0.00064	-0.00066	1.5%

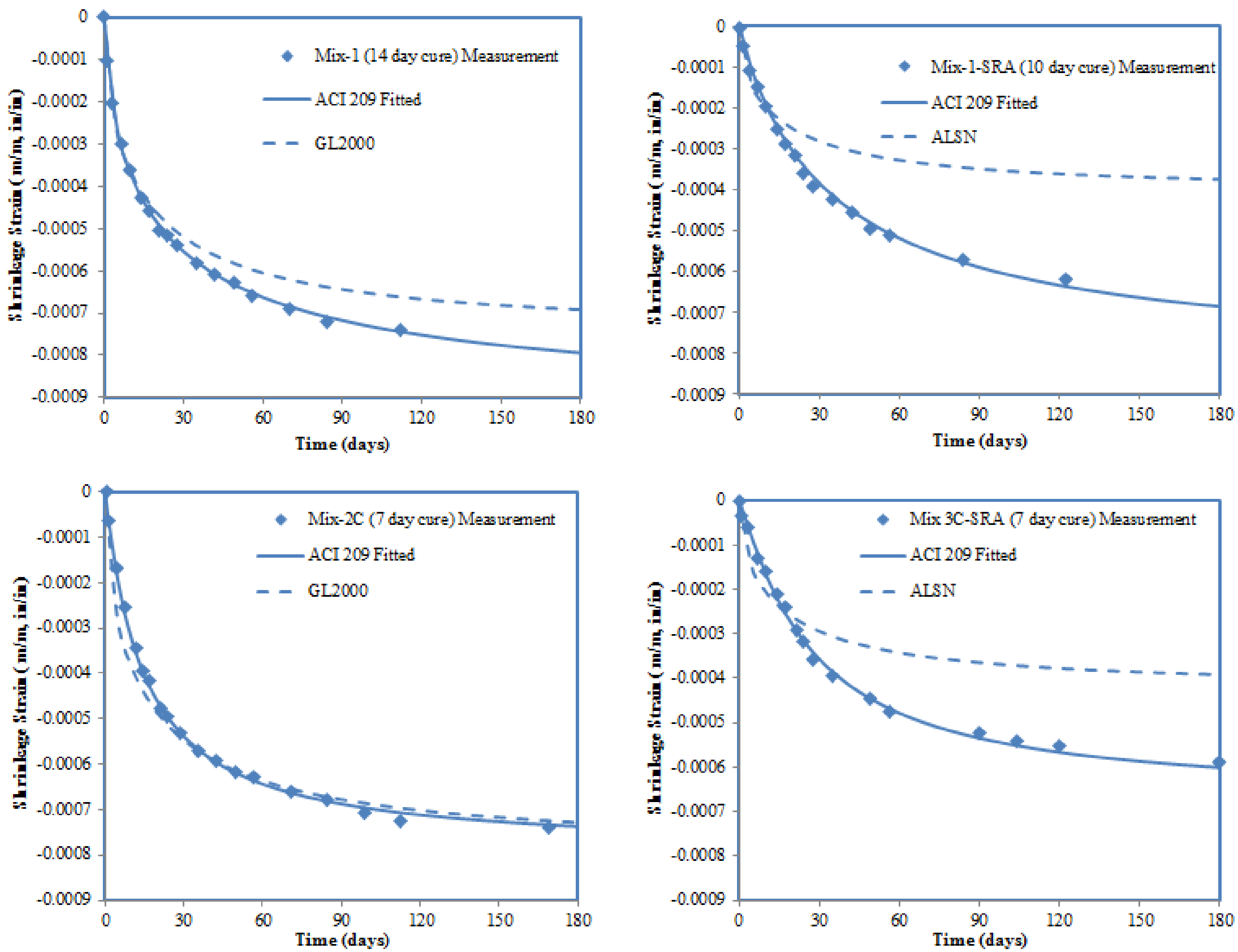

Figure 4.7 Comparison between experimental measurement, ACI 209 fitted curve using selected cut-off data, and GL2000/ALSN calculated shrinkage strain

- Perform ASTM C157 test, track the length change in a weekly basis (daily basis for the first week of drying);
- After each measurement starting from 28 days of drying, perform curve fitting to all data at hand using Eq. (4.4), determine the three parameters (ε_{sh} α, and f);
- Keep tracking the shrinkage development till the fitted ε_{sh} is stable at certain drying period (cut-off date), take the last fitted ε_{sh} as the ultimate shrinkage value;
- For the HPC studied in this research, a cut-off date of 50 day is reliable for the control HPC and the HPC internally cured by pre-wetted LWFA.A longer cur-off date of 84 day is selected for the HPC incorporated with 2% SRA.

4.7 CONCLUSIONS AND RECOMMENDATIONS

Ten different HPC concrete mixtures with LWFA or/and SRA were cast and drying shrinkage was monitored by the ASTM C157 test. Data collected was used to evaluate six existing shrinkage prediction models, including the ACI 209 model, the CEB90 model, the AASHTO model, the B3 model, the GL2000 model and the ALSN model. Several conclusions can be drawn as follows:

- All models, except the GL2000 model, underestimated shrinkage compared to experimental measurements in this research;
- The GL2000 is the best model to predict shrinkage, especially for the HPC control mixture, at the later ages, followed by the CEB90 model. The GL2000 model also gave an acceptable performance predicting HPC internally cured by pre-wetted LWFA; and,
- Although developed to predict shrinkage for concrete with SRA, the ALSN model did not perform satisfactorily as expected in this research.

A procedure to prediction long-term shrinkage stain using short-term experimental measurements was proposed based on current ACI 209 model. The comparison results indicated that the prediction using the proposed procedure outperformed all existing shrinkage models. A 50 day test period was recommended for HPC and internally cured HPC with pre-wetted LWFA. A longer test period of 84 day was recommended for concrete with SRA.

4.8 ACKNOWLEDGMENTS

The authors would like to thank the Oregon Department of Transportation which provided the funding for this research (SPR711 and SPR728).

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5 MANUSCRIPT 3

Assessing Drying Shrinkage Related Cracking Potential of High Performance Concrete for Bridge Decks

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To be submitted to

Journal of Cement and Concrete Composites

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5.1 ABSTRACT

Cracking at an early-age of high performance reinforced concrete structures, in particular bridge decks, results in additional maintenance costs, burden on serviceability and reduced long-term performance and durability. The causes behind cracking in high performance concrete are well known and documented in the existing literatures. However, appropriate shrinkage limits and standard laboratory/field tests which allow proper criteria to ensure crack-free or highly cracking-resistant high performance concrete are not clearly established either in the technical literature or in specifications. The purpose of this research is to provide shrinkage threshold limits for specifications and to provide a robust test procedure which allows easy determination of compliance with specified threshold limits. It has been shown that the "ring" tests (ASTM C1581 and AASHTO T334) are the most comprehensive accelerated laboratory tests to accurately identify cracking potential. In addition, acceptable correlation between the ring test and the field test has been observed and documented. However, a more simple and robust test procedure is in demand from materials suppliers and Departments of Transportation. A data analysis of current research shows that the ratio of free shrinkage to shrinkage capacity (theoretical strain related to tensile strength and modulus of elasticity), or "cracking potential indicator", is a promising assessment of cracking resistant performance. In this way, only the free shrinkage test (ASTM C157) and basic mechanical properties are required to assess cracking risk of candidate concrete mixture designs. This research investigation shows that a CPI less than 2.5 indicates low cracking risk.

5.2 INTRODUCTION

Among 605,000 bridges across the country monitored by the United States Department of Transportation (USDOT), 26.9% of them were reported "structurally deficient" or "functionally obsolete" in 2010, which refers to the bridge either having major deterioration and cracks that reduce its load-carrying capacity, or no longer meeting the current design standards (*U.S. Department of Transportation 2010*). In 2013, a grade of C+ was given to the national bridge system by the American Society of Civil Engineers (ASCE), and an annual investment of $20.5 billion estimated to improve current bridge conditions (*ASCE 2013*). In 2003, a nationwide state DOTs survey conducted by Michigan DOT (*Aktan et al. 2003*) on early-age bridge deck cracking issues indicated that 78% of the 31 responding states identified transverse cracking, which indicates the presence of drying shrinkage. Cracking, especially at early age, in high performance concrete (HPC) may result in a significant decrease in concrete durability and service life of the structure. Concrete bridge decks demand qualities from HPC such as low permeability, high abrasion resistance, superior durability, and long design life. To meet these requirements, bridge deck concrete is usually produced with low water to cementitious material ratio (w/cm) typically less than 0.40, high overall cement contents, inclusion of supplementary cementitious materials (SCMs) e.g. silica fume, fly ash and slag, and smaller maximum aggregate size (due to reinforcement constraints). All these features in the mixture design make the HPC bridge decks inherently susceptible to shrinkage and increased cracking risk (*Holt 2001, Bentz and Jensen 2004*). A comprehensive report on factors that affect shrinkage of hardened concrete can be found in literature (*ACI Committee 209 2005*).

The most significant challenge, from a concrete materials perspective, to overcoming cracking risk is to reduce the shrinkage, and ultimately the stresses generated as a result of such shrinkage. To mitigate the cracking issues due to shrinkage, many methods have been studied and documented. During the last 10 years, internal curing with pre-wetted fine lightweight aggregate (FLWA) has also been proved effective in mitigating concrete cracking potential (*Mohr and Bentz 2008, Schindler et al. 2012, Bentz and Weiss 2011*), and has been steadily progressing from laboratory research (*Lura 2003, Friggle and Reeves 2008, Paul and Lopez 2011, Slatnick et al. 2011*) to field applications (*Villarreal and Crocker 2007, Delatte et al. 2008, Friggle and Reeves 2008, Villarreal 2008, Cusson et al. 2010*). Another focus over the last 10 to 15 years has been shrinkage reducing admixtures (SRAs), which have also proved to be successful in reducing shrinkage induced cracking (*Tazawa and Miyazawa 1995, Folliard and Berke 1997, Bentz et al. 2001, Bentz 2005, Rongbing and Jian 2005, Bentz 2006, Saliba et al. 2011*). Some other techniques that have proven effective in controlling cracking in concrete bridge decks are fiber reinforced concrete (*Dubey and Banthia*), shrinkage-compensating concrete (*ACI Committee 223 2010*), and special construction practices (i.e. extended curing duration, controlled slump, and proper environment conditions of placement). Moreover, the type of aggregate has a significant impact on the amount of shrinkage in concrete. Research showed that sandstone aggregate concrete exhibited the higher drying shrinkage, while concrete made from limestone aggregate proved to be the most cracking-resistant (*Krauss and Rogalla 1996, Burrows 1998*). Other authors have shown that higher aggregate content (in volume or/and in maximum size) could reduce shrinkage due to relatively low cement paste content (*Burrows 1998, Nilson et al. 2004*).

To assess the cracking potential of concrete, the restrained ring test has given comprehensive and reliable results, and it has been used by many researchers. This test is also standardized as ASTM C1581 (*ASTM C1581 2004*) and AASHTO T334 (*AASHTO T334-08 2008*) (formerly known as AASHTO PP34-98). ACI Committee 213 report on early-age cracking provides a comprehensive overview of the qualitative use of the restrained ring test (*ACI Committee 231 2010*). However, the restrained ring test requires a complicated equipment setup, such as steel ring instrument, strain gauges, and data collection system. It also requires expertise to process and explain the results. All these features have hindered its potential application by concrete contractors and small material testing laboratories. Therefore, a simplified testing method or testing protocol is needed. The free dying shrinkage test specified in ASTM C157(*ASTM Standard C157 2008*) is a simple test to assess shrinkage of a given concrete mixture, but it does not give a comprehensive assessment of cracking risk as does the restrained ring test. The free drying shrinkage measured in ASTM C157 has been shown a weak correlation to the cracking potential measured in restrained ring test. Nevertheless, due to its simplicity, free drying shrinkage limits have been set up based on ASTM C157 test by many agencies, such Unified Facilities Guide Specifications (UFGS)

(*Unified Facilities Guide Specifications 2012*) and some state DOTs (*Mokarem et al. 2005, Nassif et al. 2007, Qiao et al. 2010, Ramniceanu et al. 2010*). The Federal Highway Administration (FHWA) is also considering implementing a single value shrinkage limit in the new specifications (FP-12) (*Dale P 2012*). However, there is no shrinkage threshold limit commonly agreed upon to ensure a crack-free or highly cracking-resistant concrete. The next section of this paper summarizes the previous efforts on linking free drying shrinkage of HPC to cracking performance.

5.3 SHRINKAGE LIMITS AND CRACKING POTENTIAL ASSESSMENT

A review on the most recent research efforts on establishing cracking potential assessment is given in this section. Most of these studies agreed that the restrained ring test could provide comprehensive estimation of the cracking potential of concrete materials. In addition, since the free drying shrinkage test is much simpler, some attempted to link the free shrinkage to the restrained ring test or field performance and subsequent free shrinkage limits have been recommended.

See et al. (*See et al. 2004*) investigated a wide range of concrete and mortar mixtures using the ASTM ring test. Based on the results, they suggested a cracking potential classification (as shown in Table 5.1) on the basis of either time-to-cracking or stress rate development in the concrete ring specimen. This classification was also adopted by ASTM C1581.

Table 5.1 Cracking potential classification (Based on stress rate at time-to-cracking). (*ASTM C1581 2004, See et al. 2004*)

Time-to-Cracking, t_{cr}, Days	Stress Rate at Cracking, S, MPa/Day	Potential for Cracking
$0 < t_{cr} \leq 7$	$S \geq 0.34$	High
$7 < t_{cr} \leq 14$	$0.17 < S < 0.34$	Moderate-High
$14 < t_{cr} \leq 28$	$0.10 < S < 0.17$	Moderate-Low
$t_{cr} > 28$	$S < 0.10$	Low

Time-to-cracking is the difference between the age at cracking and the age drying was initiated. It can be used to assess the relative cracking performance of specimens that cracked during the test. If not cracked, the stress rate at the age the test was terminated can be compared between tested materials.

During the last decade, Virginia DOT (VDOT) also actively sought appropriate limits on drying shrinkage for performance specifications (*Mokarem et al. 2003, Mokarem et al. 2005, Ramniceanu et al. 2010*). In earlier research, VDOT recommended drying shrinkage limits of 300 μm/m 28 days and 400 μm/m at 90 days for ordinary portland cement concrete (*Mokarem et al. 2003*), and 400 and 500 μm/m for supplementary cementitious materials (SCMs) blended concrete respectively (*Mokarem et al. 2003, Mokarem et al. 2005*). The shrinkage was measured on ASTM C157 specimen with 7 days wet curing instead of standard 28 day wet curing. This was done by comparing free shrinkage results (ASTM C157) to the restrained ring test (AASHTO). However, specimens that performed better in the ring test did not subsequently have lower free shrinkage strain. In recent research (*Ramniceanu et al. 2010*), VDOT proposed a series of shrinkage limits on bridge deck overlays which are currently used in the field in Virginia. The limits were based on the observation of free shrinkage results and scaled bridge deck overlay specimens in laboratory testing. The researchers concluded that the scaled bridge deck specimen closely mimicked the field condition, thus the free shrinkage could be linked to bridge deck cracking-resistant performance. For example, if the 28 day free shrinkage of the scaled overlay was 750 micron strain and no crack was found in the scaled specimen, the shrinkage limits for that particular mixture was set to 800 micron strain at 28 day. In this way, a series of shrinkage limits were set for specific mixtures.

New Jersey DOT (NJDOT) performed a research project from 2005 to 2007, to investigate the cracking potential of the HPC mixes for bridge decks in New Jersey (*Nassif et al. 2007*). Comprehensive laboratory tests were conducted, including compressive strength, splitting tensile strength, modulus of elasticity, free shrinkage, and restrained shrinkage. For restrained shrinkage testing, AASHTO PP34-99 was utilized with modifications to better capture the cracking performance by monitoring the relative displacement within the ring specimen. They found that high coarse aggregate to fine aggregate ratio (over 1.5) with high coarse aggregate content (over 1110 kg/m^3) could help significantly reduce cracking potentials. By correlating free shrinkage to restrained shrinkage performance, a free shrinkage limit of 450 μm/m at 56 days was recommended to ensure a high cracking resistant performance for HPC bridge deck.

From 2002 to 2006, Texas DOT (TxDOT) conducted a research project to identify effective materials-based methods of controlling drying shrinkage (*Folliard et al. 2003, Brown et al. 2007*). Innovative methods included the use of fiber-reinforced concrete, SRA, shrinkage compensating concrete, and high-volume fly ash (HVFA) were investigated in laboratory tests, including free shrinkage test, restrained ring test (AASTHTO), compressive strength, splitting tensile strength, and modulus of elasticity. In addition, several large-scale bridge decks (LSBD) representing real bridge detailing were cast and monitored at an outdoor exposure site. The results showed that the free shrinkage test did not provide enough information to assess cracking potential, because it did not agree well with the LSBD performance. On the contrary, the AASHTO test agreed well with the LSBD. Given a closer look at the mixtures that performed well in LSBD, the researchers claimed that an ideal crack-free or highly cracking-resistant mixture should be the one which shows no crack in AASHTO ring test, and has a relatively low free shrinkage strain (300 μm/m or less) with low early-age modulus of elasticity but high early-age tensile strength (*Brown et al. 2007*). Nevertheless, no shrinkage limit or assessment criterion was proposed, due to the complicated interaction among all materials properties.

In 2005, the Kansas DOT (KDOT) conducted a study on evaluating shrinkage and cracking behavior of concrete (*Tritsch et al. 2005*). In this study, free shrinkage tests (ASTM C157) and restrained ring tests (AASHTO) were use. The researchers found that the free shrinkage was a weak predictor of restrained shrinkage. This was due to that only one ring specimen (out of 39) cracked, which was likely due to lack of restraint provided by the inner steel ring. Although no shrinkage limit was recommended in this study, it laid down the groundwork for the following research which lead to low cracking high performance mixture design, termed “KU Mix”, featuring moderate strength (25 to 30 MPa), aggregate gradation optimization, low over all cement content (320 kg/m^3), larger maximum aggregate size (25mm), maximum slump (90mm), and limited placement temperature (13°C to 21°C) (*Lindquist et al. 2008, McLeod et al. 2009*). It has proven effective in reducing cracking in bridge decks in field applications in Kansas (*Darwin et al. 2010*).

Recent research conducted by the Washington DOT (WDOT) studied early-age cracking mitigation strategies for concrete bridge decks (*Qiao et al. 2010*). The goal of this research was to identify effective early-age cracking mitigation strategies for concrete bridge decks in Washington State. In addition to the free shrinkage test (ASTM C157) and the restrained ring test (AASHTO), hardened concrete properties, i.e. compressive strength, splitting tensile strength, flexural strength, and modulus of elasticity were tested at 7 and 28 days of age. Based on the results, the researchers attempted to link the free shrinkage to cracking performance, and concluded that lower free shrinkage strain with acceptable flexural strength generally indicated relatively good restrained shrinkage cracking resistance. WSDOT also adopted the “KU Mix” design methodology in this research, which resulted in a significant reduction in drying shrinkage. Bridge decks free of early-age shrinkage cracking were achieved in the field. Currently, WSDOT is implementing a free shrinkage limit of 320 μm/m strain at 28 day of drying (*Qiao et al. 2010*).

Sponsored by West Virginia Division of Highways (WVDOH), a recent research project conducted by Ray and co-workers (*Ray et al. 2012*) studied the correlation between shrinkage and cracking of HPC for bridge decks. Using the AASHTO ring test and ASTM C157 free shrinkage test, 18 HPC mixtures with different SCMs (i.e. fly ash, microsilica, slag, and metakaolin) and different w/cm were investigated. Based on the test results, a correlation was established between “cracking index” and time of cracking in ring tests. The “cracking index” is a factor combining compressive strength, modulus of elasticity, and free shrinkage strain. Similar to the approach used by VDOT, representative highway bridge decks which showed satisfactory cracking-resistant performance on the field were selected to establish the limit based on the AASHTO ring tests. It was concluded that for their local materials, any concrete mixture that cracked later than 30 days in the AASHTO ring test could be conservatively considered acceptable in cracking-resistant performance on the field, and the “cracking index” could be applied elsewhere using the same concepts.

From all previous research endeavors the current understanding of cracking risk assessment can be summarized by: 1) it is well agreed upon that the retrained ring test (both ASTM and AASHTO) provide the most comprehensive estimation of concrete cracking potentials; 2) high-cracking-resistant concrete is concrete that has low free shrinkage, relatively low modulus of elasticity, and high tensile (or flexural) strength; 3) there is no commonly agreed up on testing method and subsequent shrinkage limits to control cracking risk; 4) most shrinkage limits developed by state DOT, if any, only applied to specific mixture designs.

The deficiencies in previous approaches are listed as follows:

- The shrinkage limits are specific to mixtures. They are limited by the concrete mixture designs use to develop those limits. A general approach is needed.
- The restrained ring test could be used to access cracking risk. However, the ring test, by its nature, requires complicated instrumentation and a well facilitated laboratory. Therefore, a simpler yet as reliable test method/protocol is needed to assess concrete potentials.

In this research, internal curing with FLWA and a shrinkage reducing admixture were selected as shrinkage reducing methods. Two different aggregate types, siliceous sandstone and limestone, were also included in the investigation. A “cracking potential indicator” (CPI), accompanied with a simple testing protocol, is proposed based on results of this study to assess cracking potential without restrained ring test. Only free shrinkage test (ASTM C157) and basic mechanical property tests (splitting tensile strength and static elasticity modulus) are required to satisfactorily assess concrete cracking potential. Theoretically, this approach could be generally applied to any given concrete mixture design.

5.4 EXPERIMENTAL

5.4.1 Materials

5.4.1.1 Binders

The cement and SCMS used in this research were an ASTM Type I/II ordinary Portland cement, an ASTM C618 Class F fly ash, and an ASTM C1240 silica fume. The dioxide analysis of the cement and fly ash is shown in Table 5.2. The silica fume contains nearly pure silica dioxide in noncrystalline form with approximately 1% crystalline silica.

Table 5.2 Cement and fly ash oxide analysis (wt %)

	CaO	SiO_2	Al_2O_3	Fe_2O_3	MgO	Na_2O	K_2O	TiO_2	MnO_2	P_2O_5	SrO	BaO	SO_3	Total Alkalies as Na_2O	Loss on Ignition
OPC	63.57	19.95	4.71	3.50	0.85	0.25	0.27	0.24	0.09	0.09	0.16	0.06	3.19	0.43	3.19
Fly Ash	10.20	55.24	15.77	6.27	3.64	3.64	2.08	0.94	0.12	0.23	0.32	0.62	0.70	-	0.23

5.4.1.2 Admixtures

A high-range water-reducer and an air-entrainer were used and adjusted in all mixtures to achieve consistent workability (target 150mm slump) and appropriate air content (5±1.5%) to ensure proper freeze/thaw resistance. One SRA which is compatible with the air entrainer was used in some mixtures, at 2% of the total cementitious materials.

5.4.1.3 Aggregates

Local siliceous river gravel and natural siliceous river sand were used in most mixtures. The maximum size of the river gravel was 19 mm. A siliceous limestone was used as the coarse aggregate in one mixture, with 19 mm maximum size.

One ASTM fine lightweight aggregate (i.e. expanded shale) was used as an internal curing agent in certain mixtures to partially replace normal weight fine aggregate. Absorption and desorption test were performed to characterized this FLWA in literature (*Deboodt 2011*). The replacement level of FLWA was based on the Bentz Equation (*Bentz et al. 2005*) and the calculation can be found in literature (*Fu et al. 2012*). A summary of aggregates properties is given in Table 5.3.

Table 5.3 Aggregates properties

	Specific Gravity	Absorption Capacity (%)	Desorption Capacity (%)	Fineness Modulus
River Sand	2.41	3.08	-	3.0
River Gravel	2.44	2.58	-	7.1
Limestone	2.68	0.58	-	6.5
Expanded Shale	1.55	17.50	16.0	2.7

5.4.2 **Mixture Design**

All mixtures in this study were based on Oregon DOT HPC mixture design for bridge decks. The target compressive strength was 34.5 MPa. A w/cm of 0.37 was used for all mixtures except for one OPC mixture which used 0.42 w/cm. The total cementitious materials content in all mixtures was 375 kg/m^3, including 30% replacement of class F fly ash and 4% silica fume, except for two OPC mixtures which contained only portland cement. For mixtures incorporating with FLWA, approximately 40% river sand was replaced volumetrically by expanded shale. Two different wet curing durations (i.e. 3 day and 14 day) were used. A detailed mixture proportioning and description is given in Table 5.4. SYN is an HPC mixture with incorporation of both FLWA and SRA (SYN short for synergy). OPC1 and OPC2 are two mixtures without fly ash or silica fume. LS is a HPC mixture with limestone as coarse aggregate rather than local river gravel. CRM is a proprietary repair mortar which is typically used for structure repairs such as mending bridge deck cracking. It was mixed according to manufacturer instructions.

Table 5.4 Concrete mixture proportioning and descriptions

Mixture	Curing Duration (day)	Cement (kg/m^3)	Fly ash (kg/m^3)	Silica fume (kg/m^3)	Water (kg/m^3)	Coarse aggregate (kg/m^3)	Sand (kg/m^3)	FLWA (kg/m^3)	SRA (kg/m^3)
HPC1	3	249	112	15	139	1074	659	-	-
HPC2	14	249	112	15	139	1074	659	-	-
SRA1	3	249	112	15	131	1074	659	-	7.5
SRA2	14	249	112	15	131	1074	659	-	7.5
FLWA1	3	249	112	15	139	1074	400	164	-
FLWA2	14	249	112	15	139	1074	400	164	-
SYN	14	249	112	15	131	1074	400	164	7.5
OPC1	14	375	-	-	139	1074	659	-	-
OPC2	14	375	-	-	158	1074	659	-	-
LS	14	249	112	15	139	1100	740	-	-
CM*	3	-	-	-	-	-	-	-	-

*CM – a proprietary repair mortar without coarse aggregate.

Note that although the mixture proportioning of HPC1 and HPC2 are exactly the same, the curing duration is different. In the ring test, the curing duration are important factors to affect testing results, therefore HPC1 and HPC2 are treated as two mixtures in this study. This is also true to SRA1/SRA2, and FLWA1/FLWA2. For OPC1 and OPC2, they have different w/cm but same curing duration. A curing duration of 14 day was chosen to match ODOT standard. In addition, according to previous research (*Deboodt 2011*) , a shortened curing duration of 3 days was also selected to make a distinguishable different in terms of free drying shrinkage.

5.4.3 **Methods**

For each mixture, the following specimens were prepared accordingly:

- Cylinders (ϕ100 × 200 mm) for compressive strength (6 replicates), splitting tensile (6 replicates), and static modulus of elasticity (4 replicates) for both 28 day wet cured condition and 28 day match cured condition.
- ASTM C157 prism (3 replicates);
- ASTM C1581 ring specimens (3 replicates).

Note that for mechanical properties tested on cylinders, two curing durations (matched cure with rings, and standard 28 day wet cure) were used. The ASTM C157 prisms were match cured with ring specimens.

5.4.3.1 Mechanical Properties

Compressive strength, splitting tensile strength, and static modulus of elasticity were tested according to ASTM standards, at 28 days of age. For each mixture, $\phi 100 \times 200$ mm cylindrical samples were cured in two conditions: standard 28 day wet cure, and 28 day matched cure. For standard curing, samples were demolded 24 hours after casting and stored in standard moisture room (23°C and 100% RH) until testing. For matched curing, samples were demoded 24 hours after casting and stored in the standard moisture room until the end of desired wet curing periods. Then these samples were moved to the drying chamber (23°C and 50% RH) and stored near the ring specimens until testing. This was to ensure the measured mechanical properties are representative of ring specimens.

5.4.3.2 Free Shrinkage

Free shrinkage was measured using the ASTM C157 test. The concrete prisms dimension is 75×75×280 mm. All specimens were cast and sealed in the molds until demolding at 24 hours, then moved to the moisture room for curing until the end of desired wet curing periods. Instead of standard curing duration which is 28 days wet curing, all prisms were match-cured with the ring specimen. This was also to ensure the measured free shrinkage is representative of corresponding ring specimen.

5.4.3.3 Restrained Shrinkage

All restrained shrinkage ring specimens were prepared according to ASTM C1581. Four strain gauges were attached 90° apart to the inner surface of the steel sing at mid-height. The strain was recorded using a data acquisition system. After concrete placement, all specimens were immediately moved to a drying chamber (23ºC and 50% RH) and covered with wet burlap and plastic sheet until 24 hours of age. Then, the outer PVC rings were removed and all concrete surfaces were covered with wet burlap and plastic sheeting for the entire desired curing period. During the curing period, to maintain the moisture condition, the burlap was re-wetted every 48 hours. At the end of the desired curing period, the burlap and plastic sheet were removed. Then the top concrete surface was sealed by a waterproof sealant to ensure drying only circumferentially. All rings specimen were inspected every 24 hours until cracks were observed in all three replicates. By examining the strain gauge recording, the exact time of cracking can be determined. All tests were terminated at 60 days regardless of whether cracks were observed or not.

5.5 RESULTS AND DISCUSSIONS

5.5.1 Mechanical Properties

Table 5.5 shows the summary of compressive strength, splitting tensile strength, and modulus of elasticity of all mixtures. Most of the mixtures met the 34.5 MPa strength target. For curing, in addition to the standard 28 day wet cure method, cylinder samples were also match cured with ring specimen to match the exact curing duration. For instance, cylinders using 28 day match cured condition, were wet cured for 3 day (the first 24 hours in the mold), and exposed to drying environment for 25 days before tested.

Table 5.5 Concrete Mechanical Properties

Mixture	Curing Duration (days)	28 Day Wet Cured			28 Day Match Cured		
		Compressive Strength (MPa)	Tensile Strength (MPa)	Modulus of Elasticity (GPa)	Compressive Strength (MPa)	Tensile Strength (MPa)	Modulus of Elasticity (GPa)
HPC1	3	28.8	3.42	22.9	30.3	3.49	24.4
HPC2	14	35.4	4.06	28.7	39.9	4.40	27.5
SRA1	3	33.2	3.97	28.0	-	-	-
SRA2	14	36.4	3.78	29.3	39.1	4.08	27.4
FLWA1	3	36.6	3.72	24.2	-	-	-
FLWA2	14	45.4	5.17	29.6	53.5	5.48	29.7
SYN	14	26.1	2.76	22.0	24.2	2.93	21.1
OPC1	14	44.7	3.67	32.2	45.7	4.29	33.1
OPC2	14	34.5	3.42	30.0	-	-	-
LS	14	34.2	3.90	32.4	35.9	4.12	25.6
CM	3	-	-	-	61.2	5.86	29.4

One interesting observation that can be made from Table 5.5 is that test on matched cured cylinder test yielded almost consistently higher compressive (except for SYN) and tensile strength, but a similar modulus of elasticity. Note that matched cured cylinders went through significant drying duration (14 days), which is considered unfavorable for strength gain for concrete by classic theories. The reason is still unknown and further investigation of this phenomenon is underway.

Table 5.6 shows a summary of five compressive strength test results and variation. Each set of test were from a HPC control mixture, casted during two years' time when the research project was conducting. The mixture proportioning of each mix was the same among all five mixtures. And same materials, such as cementitious materials, chemical admixtures, and fine and coarse aggregates were used in all of these mixtures. Same mixture procedure was followed.

Table 5.6 Variation of 5 Compressive Strength Tests

Test #	Compressive Strength (MPa)			Ave. (MPa)	Standard Deviation (MPa)	Max difference within test
	A	B	C			
1	29.0	26.4	28.0	27.8	1.3	9.6%
2	34.7	35.2	31.5	33.1	2.3	9.8%
3	31.6	29.7	32.9	31.3	2.2	10.0%
4*	28.4	29.7	28.3	29.0	1.0	4.7%
5**	34.5	33.9	37.7	35.8	2.7	10.5%
	Average of 5 tests			31.4	3.2	

*HPC1; **HPC2

According to ASTM C39, the acceptable range (d2s% described in ASTM C670) of individual cylinder (100 × 200 mm) strength is 10.6% (*ASTM C670*). It can be seen that all five mixtures met the requirement. However, it is difficult to control the viability between different mixtures, even in laboratory conditions. Possible reasons are aggregate sampling, environment conditions (temperature and RH), and testing operators. And also the variability of concrete lies in itself as a composite material.

5.5.2 **Free Shrinkage**

Table 5.7 gives a summary of free shrinkage measurements of all mixtures at increasing age. HPC2 represents the control mixture and curing condition. The percentage in the brackets show the relative scale of certain shrinkage compare to HPC2 at the same age. The free shrinkage at the early age was effectively reduced for mixtures using mitigation methods (SRA, FLWA, or synergy of both). However, given the high shrinkage nature of this HPC mixture, using the FLWA alone was not as effective as the other two methods, especially at later age. The synergy of SRA and FLWA most significantly reduced the free shrinkage. This effect is also shown in Figure 5.1 and Figure 5.2.

Table 5.7 Summary of Free Shrinkage (μm/m) and % relative to HPC2

Mixture	Curing Duration (days)	7 day	28 day	56 day	90 day	180 day
HPC1	3	340 (91)	600 (87)	727 (90)	780 (85)	863 (90)
HPC2	14	**373 (100)**	**693 (100)**	**810 (100)**	**917 (100)**	**960 (100)**
SRA1	3	133 (36)	337 (49)	443 (55)	497 (54)	-
SRA2	14	190 (51)	447 (65)	573 (71)	640 (70)	710 (74)
FLWA1	3	280 (75)	535 (77)	633 (78)	703 (77)	-
FLWA2	14	323 (87)	663 (96)	800 (99)	870 (95)	917 (96)
SYN	14	140 (38)	345 (50)	465 (57)	530 (58)	-
OPC1	14	360 (97)	600 (87)	690 (85)	750 (82)	830 (86)
OPC2	14	300 (80)	557 (80)	677 (84)	747 (81)	837 (87)
LS	14	240 (64)	380 (55)	430 (53)	457(50)	-
CM	3	207 (55)	447 (65)	610 (75)	740 (81)	853(89)

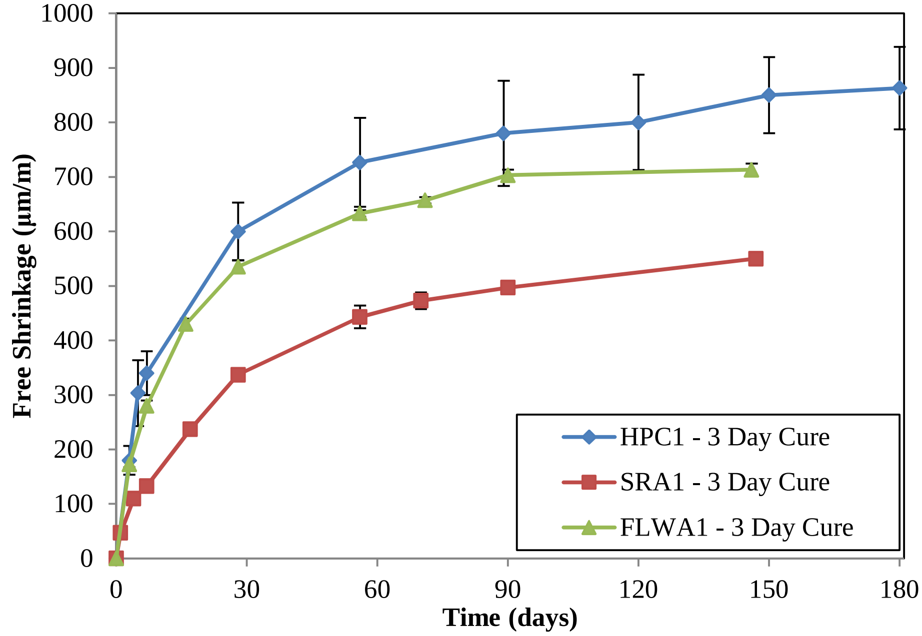

Figure 5.1 Free shrinkage versus drying time, 3 day cure, effect of shrinkage mitigation methods

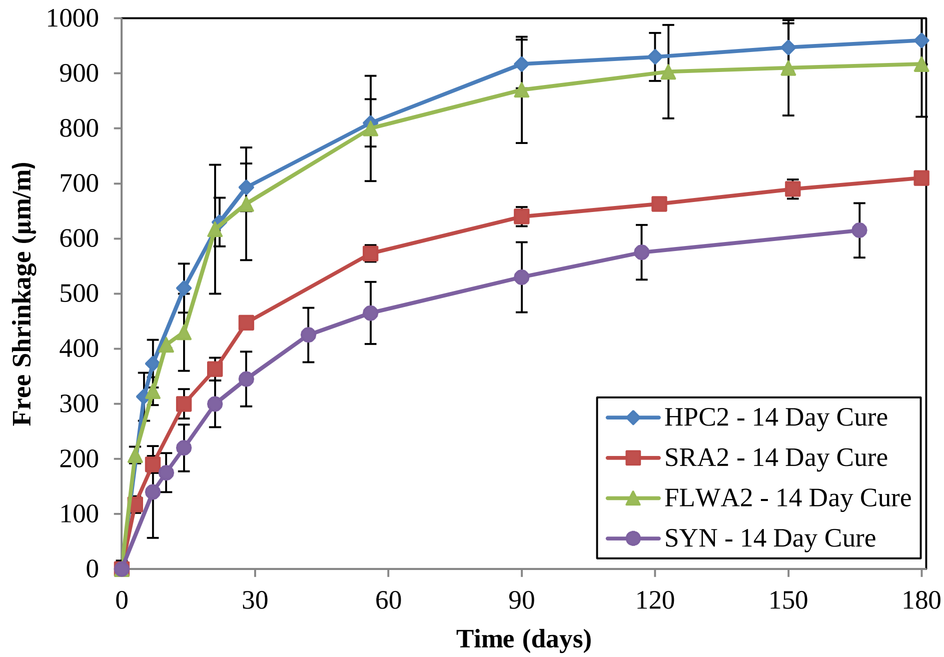

Figure 5.2 Free shrinkage versus drying time, 14 day cure, effect of shrinkage mitigation methods

Figure 5.3 shows the effect of water to cement ratio as well as incorporation of SCMs. OPC1 and OCP2 showed over 800 μm/m shrinkage at 180 day of age, which is considered high shrinkage. Moreover, the presence of SMCs (30% fly ash and 4% silica fume) may have further augmented shrinkage of the control HPC mixture.

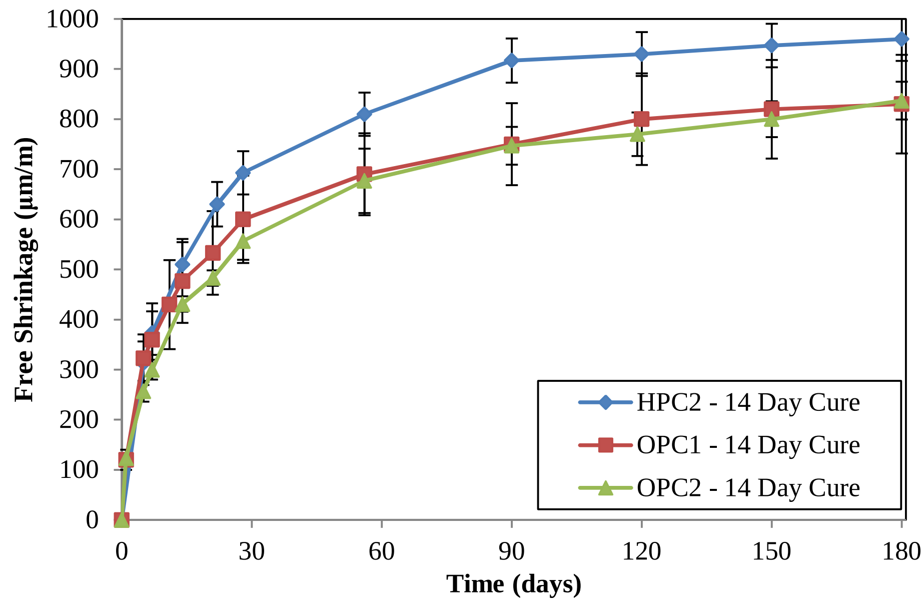

Figure 5.3 Free shrinkage versus drying time, 14 day cure, effect of w/cm and SCMs

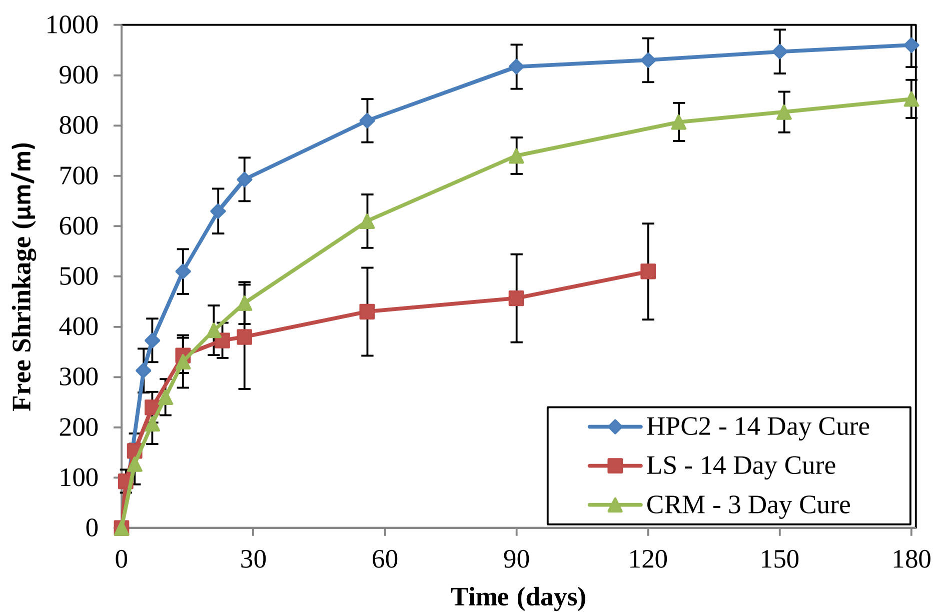

Figure 5.4 Free shrinkage versus drying time, 14 day cure, effect of aggregates

As discussed previously, aggregate type also has significant impact on shrinkage behavior. As shown in Figure 5.4, free shrinkage was reduced by 45% at 28 days after drying using limestone instead of siliceous river gravel as coarse aggregates. From a point of view of aggregate type, CM, as a mortar mix, still showed superior shrinkage performance, likely due to that the mixture contained high volume of quartz sand, which is believed to perform best in shrinkage behavior in concrete.

5.5.3 **Restrained Drying Shrinkage**

Table 5.8 gives a summary of the ASTM C1581 ring results, including time-to-cracking and the corresponding stress rate.

Table 5.8 Summary of time-to-cracking and stress rate

Mixture	Curing Duration (days)	Time-to-Cracking, Days				Stress Rate, MPa/Day				Cracking Potential Classification*
		A	B	C	Ave.	A	B	C	Ave.	
HPC1	3	4.0	5.5	5.2	4.9	0.380	0.315	0.338	0.344	H
HPC2	14	4.4	4.6	3.6	4.2	0.343	0.281	0.482	0.369	H
SRA1	3	13.9	18.4	18.8	17.0	0.094	0.073	0.094	0.087	L
SRA2	14	16.1	14.9	11.6	14.2	0.104	0.093	0.139	0.112	ML
FLWA1	3	6.5	7.0	7.3	6.9	0.238	0.213	0.284	0.245	MH
FLWA2	14	7.4	7.9	n/a	7.7	0.245	0.263	n/a	0.254	MH
SYN	14	19.7	14.0	14.0	15.9	0.115	0.070	0.060	0.081	L
OPC1	14	4.0	5.6	5.3	5.0	0.278	0.383	0.314	0.325	MH
OPC2	14	4.2	4.6	3.6	4.1	0.257	0.266	0.359	0.294	MH
LS	14	40.9	no crack	23.1	32.0	0.045	no crack	0.099	0.072	L
CM	3	23.0	28.0	33.0	28.0	0.084	0.072	0.063	0.073	L

* H – High; ML – Moderate High; ML – Moderate Low; L – Low.

Time-to-cracking is the time elapsed between initiation of drying and the cracking in the rings. Upon cracking, a sudden change will show in two or more strain gauges recording, which can also be confirmed by visual inspection. Stress rate at time-to-cracking was calculated according to ASTM C1581. Based on time-to-cracking or stress rate, a cracking potential can be given to each mixture. The authors believe that when determining the cracking potential classification, high priority should be given to stress rate at cracking. On the one hand, the stress rate better quantifies the stress of the concrete, which is directly related to cracking issues. On the other hand, time-to-cracking is involved in stress rate calculation. In other words, stress rate indicates a more comprehensive evaluation. Figure 5.5 shows a good relationship of time-to-cracking with stress rate, with correlation coefficient over 0.94. The power relationship indicates that with the decrease of stress rate, the time-to-cracking would be significantly prolonged.

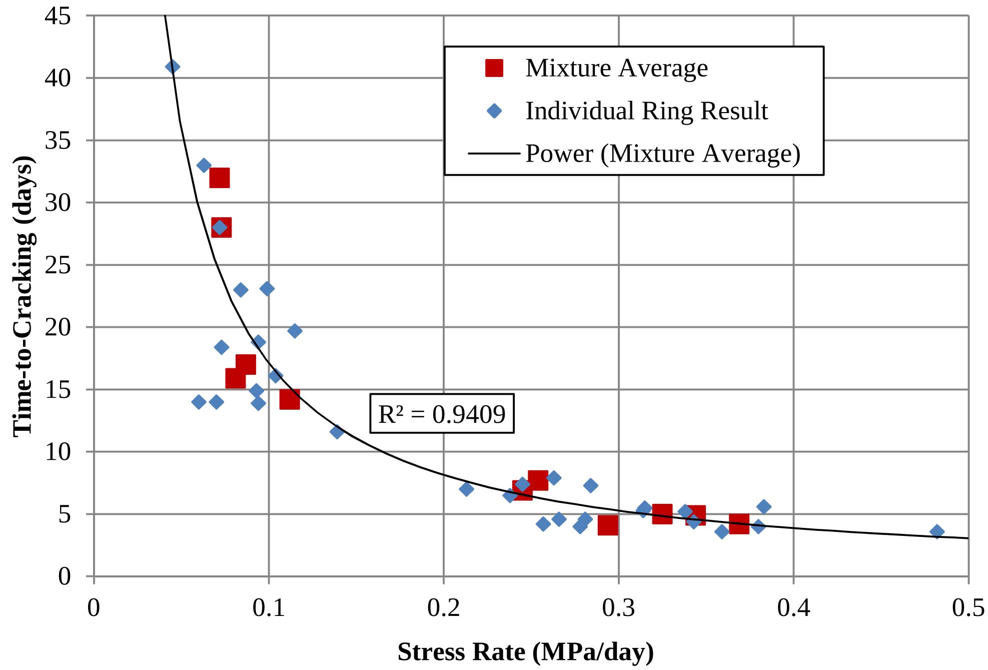

Figure 5.5 Time-to-cracking versus Stress rate

It is noted that SRA most significantly prolonged the time-to-cracking. FLWA also effectively prolonged the time-to-cracking. SYN showed the lowest free shrinkage and similar time-to-cracking to SRA. In addition, CM also exhibited superior cracking-resistance, which indicates drying shrinkage related cracking are likely not a concern when repairing an HPC bridge deck using this material.

Among all 11 mixtures, mixture LS, which is a limestone HPC, lasted the longest before cracking. In fact one LS ring specimen showed no crack at 60 days after initiation of drying, when the test was terminated. One set of strain gauge data is presented in Figure 5.6, showing the strain development in three individual ring specimen of mixture LS.

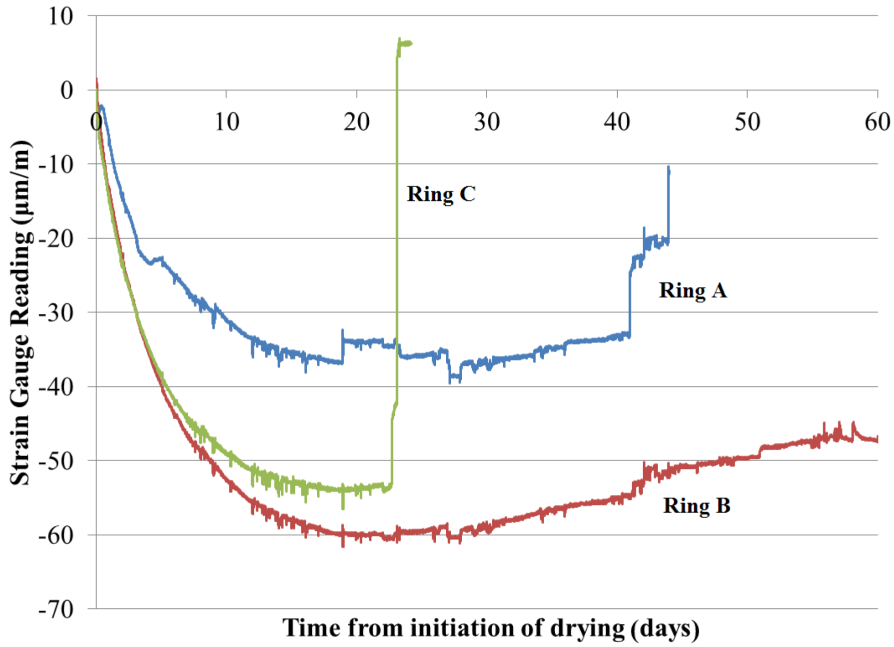

Figure 5.6 Strain development versus time, three individual rings of mixture LS

Ring C showed a typical response as most tests, which consist of an increase in strain and a "sharp jump" toward zero strain at the end. However, ring B did not show this sudden change, but rather a slow decrease in strain indicating stress relaxation in the ring. This can be seen in the later age of ring A as well. One explanation is that after a certain period of time, about 28 days in this case, the effect of stress relaxation starts to impact the cracking behavior of ASTM ring specimens. In other words, if a concrete mixture survives 28 days or longer in the ASTM ring, the cracking potential could be further lowered due to stress relaxation. However, ring A and C did not sustained the ring test as long as ring B, which is likely due to materials properties variability. This phenomenon is quite usual in restrained ring test (*Folliard et al. 2003, See et al. 2004, Radlinska et al. 2007, Qiao et al. 2010*).

As discussed previously, mixture LS, lasted longest in the ring test. It should be noted that, the only difference between LS and other HPC mixtures is the coarse aggregate used. In mixture LS, an angular limestone coarse aggregate with similar gradation and maximum size was used, while a pure silicious round river gravel aggregate was used was used in other HPC mixtures. By only replacing the coarse aggregate, the time-of cracking of concrete with limestone was significantly improved, from 4.2 days (HPC2) to 32 days, which is even more effective than mitigation techniques such as incorporation of FLWA (7.7 days) and SRA (14.2 days). This might well relates to the interfacial transition zone (ITZ) theory. Due to the fact that the limestone aggregate was angular in shape and rough in surface, more bonding surface and better mechanical bonding formed in ITZ could help to improve the cracking resistance. This aggregate effect was more significant than expected, more study is underway.

5.5.4 **Cracking Potential Indicator (CPI)**

As outlined previously, a high cracking-resistance should come from combined properties: 1) low free shrinkage; 2) relative high tensile strength to resist tensile stress developed within concrete, and, 3) relative low modulus of elasticity so less stress development on same amount of shrinkage. Thus, a “cracking potential indicator” (CPI) is proposed to assess cracking potential, taking account of free shrinkage as well as mechanical properties (i.e. splitting tensile strength and static modulus of elasticity). The equation is given as follows:

$$CPI = \frac{free\ shrinkage}{nominal\ tensile\ strain\ capacity} = \frac{\epsilon_{free}}{f_t/E_c} \tag{5.1}$$

Where:

- ε_{free} is free shrinkage (m/m) measured at 28 day from initiation of drying;
- f_t is splitting tensile strength (MPa) measure at 28 day age, and;
- E_C is static modulus of elasticity (GPa) measured at 28 day age.

The ratio of f_t to E_C is named nominal tensile strain capacity (m/m). This ratio does not have any physical meaning, but is used as a relative comparison between materials. A larger nominal tensile strain capacity indicates the material is able to accommodate more tensile deformation before cracking occurs. Since 28 days is a common industrial practice used for quality control for concrete properties, it was selected as the testing age. Note that for mechanical properties tests, concrete specimens were test at 28 day age, while for free shrinkage tests the 28 day from initiation of drying is equivalent to age of 28 day plus curing duration. Using the data listed in Table 5.5 and Table 5.7, the CPI for all mixtures can be calculated using both standard cure 28 day and matched cure 28 day mechanical properties, and shown in Figure 5.7.

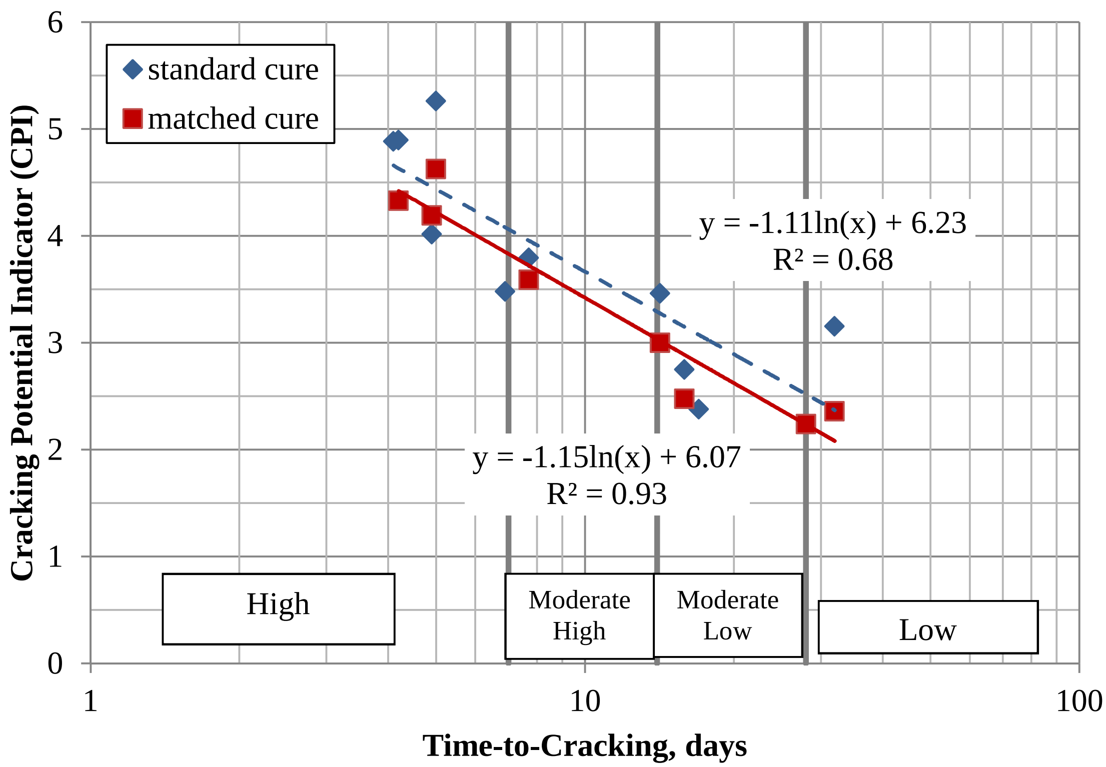

Figure 5.7 CPI versus time-to-cracking

Figure 5.7 shows the relationship between CPI and time-to-cracking. According to cracking potential classification given in Table 5.1, the chart was divided into four zones by time-to-cracking. A general trend can be observed that mixtures with lower CPI tend to fall into lower cracking risk zone. The logarithmic relations are also given in the chart with equations and correlation coefficient. One interesting finding is that CPI calculated using matched cured concrete properties showed better correlation with time-of cracking than CPI calculated using standard curing or “28 day wet cure”. This is likely due to concrete samples match cured with ring specimens more accurately represented concrete rings. Therefore, match cured concrete samples, when available, should be used to estimate the cracking risk of given concrete mixture. Figure 5.8 shows the relationship between CPI and time-to-cracking. It suggests similar trend: the CPI correlated well with stress rate. A low CPI indicates a lower stress rate in the ring test.

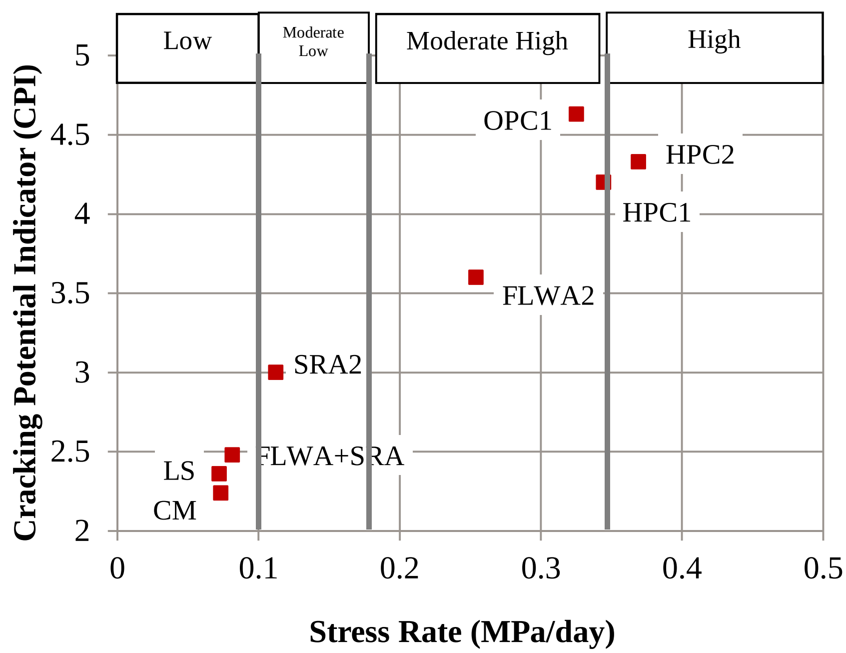

Figure 5.8 CPI versus stress rate

Based on the relationship shown in Figure 5.7 and 5.8, a preliminary cracking potential classification based on the CPI is proposed in Table 5.9.

Table 5.9 Cracking potential classification based on the CPI

Cracking Potential Indicator (CPI)	Potential for Cracking
CPI ≥ 4.0	High
3.0 ≤ CPI < 4.0	Moderate-High
2.5 ≤ CPI < 3.0	Moderate-Low
CPI < 2.5	Low

According to the proposed CPI theory, a combination of high tensile strength and low modulus of elasticity is preferred. However, these two properties are usually not independent of each other; therefore it might be difficult to manipulate in practice to achieve desired values. Generally speaking, the coarse aggregate type could impact the tensile strength (round v.s. angular) and modulus of elasticity (stiffness of coarse aggregate). But this is also limited by aggregate availability. Therefore, the factor that could most significantly affect the cracking potentials is still free drying shrinkage.

5.6 CONCLUSIONS AND RECOMMENDATIONS

According to the results of ASTM C1581 restrained ring tests, by incorporating FLWA and/or SRA the cracking resistance of ODOT HPC was significantly improved. And HPC with SRA showed most significant benefits in improving the cracking resistance. The results also indicate that HPC mix using river gravel showed significant higher shrinkage and cracking prosperity than HPC mix using limestone.

The ASTM C1581 rings test is the most comprehensive way to evaluate the cracking performance of different HPC mixtures in this study. With an attempt to simplify this test, a “cracking potential indicator” (CPI) was proposed in this study. And a good correlation was found between CPI and ring test results. To use CPI, only free shrinkage test (ASTM C157) and basic mechanical properties are required. And match cured mechanical properties under match cured conditions with representing field curing condition should be used when available. A recommended test protocol based on CPI is summarized as follows:

- Cast ASTM C157 prisms and wet cure until required curing time according to specification of HPC bridge decks, then measure free shrinkage strain at 28 days from initiation of drying;
- Using the same batch of material cast minimum nine (6 replicates for each tests) 150 × 300 mm cylinders and test mechanical properties (ft and Ec) at 28 day from casting. Use the same curing regime as ASTM C157;
- Use Eq. (5.1) to calculate CPI;
- Refer to Table 5.9 to identify the potential for cracking of testing concrete mixture.

A preliminary data analysis showed that a CPI less than 2.5 indicates low cracking risk. Theoretically, the CPI concept is not limited to the materials used in this research, and could be easily applied universally using other "local" materials. Further verification is needed.

5.7 ACKNOWLEDGEMENTS

The authors would like to acknoledge the Oregon DOT for supporting the research project (SPR 728). This research was partically funded by Portland Cement Association Education Fundations.

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6 GENERAL CONCLUSIONS

Cracking, especially at early age, in high performance concrete (HPC) may result in a significant decrease in concrete durability and service life of the structure containing it. Concrete bridge decks demand qualities from HPC such as low permeability, high abrasion resistance, superior durability, and long design life. To meet these requirements, bridge deck concrete is usually produced with low water to cementitious material ratio (w/cm), typically less than 0.40, high overall cement contents, inclusion of supplementary cementitious materials (SCMs) such as silica fume, fly ash and slag, and smaller maximum aggregate size (due to reinforcement constraints). All these features in the mixture design make the HPC bridge decks inherently susceptible to several types of shrinkage and thus increased cracking risk.

To mitigate the shrikage and cracking issues in HPC, internal curing by incorporation pre-wetted fine lightweight aggregate (FLWA) has proven effective. To determine the optimum FLWA content, information about the propensity for shrinkage in the cement paste, specifically the chemical shrinkage value, is needed. However, there is a lack of information on how to determine the long-term chemical shrinkage value for HPC with supplementary cementitious materials (SCMs) and/or shrinkage reducing admixture (SRA). Manuscipt 1 identified a simple procedure to determine long-term chemical shrinkage values for given cementitious systems with SCMs and/or SRA. Several improvements to the ASTM C1608 (dilatometry procedure) were investigated. An experimental prediction model was adopted and verified to estimate long-term chemical shrinkage values for portland cement systems containing SCMs and/or SRA. A recommended procedure was also proposed to determine the long-term chemical shrinkage values for HPC systems containing SCMs and/or SRA, and a modification to a commonly used FLWA proportioning equation is suggested.

Concrete durability is closely related to effects of drying shrinkage, therefore it is important to develop proper prediction models for modern HPC. The ACI 209 model is recommended by American Concrete Institute and widely used in the U.S. for normal strength concretes using conventional aggregates. It recommends performing short-term testing on concrete to calibrate the model to improve predictions for local materials. However, the calibration procedure is not clearly stated in the document. In Manuscript 2, shrinkage data collected from ten different high performance concrete (HPC) mixtures internally cured by pre-wetted lightweight fine aggregate (LWFA) and/or shrinkage reducing admixture (SRA) was used to evaluate current shrinkage prediction models. Those models were the ACI 209 model, CEB90 model, AASHTO model, B3 model, GL2000 model and ALSN model. It was shown that each model was limited by the data source used to develop the model. The study found that the GL2000 model showed the best overall performance in predicting shrinkage strain for internally cured HPC using local materials. Nevertheless, a major concern for each model is whether the data source used to develop the model is representative of all concretes. It was shown that more accurate long-term shrinkage prediction could be achieved based on the current ACI 209 model with experimental measurements. This proposed procedure was capable to predict long-term drying shrinkage for concrete using local materials mixture by using short-term experimental measurements.

Although the causes behind cracking in high performance concrete are well known and documented in the existing literatures, appropriate shrinkage limits and standard laboratory/field tests which allow proper criteria to ensure crack-free or highly cracking-resistant high performance concrete are not clearly established either in the technical literature or in specifications. The purpose of Manuscript 3 is to provide shrinkage threshold limits for specifications and to provide a robust test procedure, which allows easy determination of compliance with specified threshold limits. It has been shown that the “ring” tests (ASTM C1581 and AASHTO T334) are the most comprehensive accelerated laboratory tests to accurately identify cracking potential. In addition, acceptable correlation between the ring test and the field test has been observed and documented. However, a more simple and robust test procedure is in demand from materials suppliers and Departments of Transportation. Data analysis of current experimental data showed that the ratio of free shrinkage to shrinkage capacity (theoretical strain related to tensile strength and modulus of elasticity), or “cracking potential indicator”, is a promising assessment of cracking resistant performance. In this way, only free shrinkage test (ASTM C157) and basic mechanical properties are required to assess cracking risk of certain concrete mixture designs. The results indicate that a CPI lower than 2.5 indicated low cracking risk.

Several future directions this work could lead to:

- Identify the impact of coarse aggregate. It seems the aggregate effect on cracking is much more significant than previously expected. More research should be conducted to understand the impact of coarse aggregate, such as aggregate shape, surface textures, absorption capacity strength, toughness, and mineralogy, etc;

- Apply more effective/aggressive shrinkage mitigation techniques such as : higher FLWA content (meaning bring in more than need to counter autogenous shrinkage), fiber reinforcement, and combined techniques (e.g. SRA plus fiber), and;

- Apply and verify CPI theory in the field. Oregon DOT is working on a draft specification with CPI incorporated for contractors and materials suppliers.

APPENDIX A: DRYING SHRINKAGE PREDICTION MODELS

A.1 PREDCITING DRYING SHRINKAGE

This section briefly covers the main methodologies used for predicting drying shrinkage. The ACI 209 model is predominately used to predict drying shrinkage in the United States, and the CEB 90 model is used in Europe. Other methods (B3 model and GL 2000 model) have been developed through research to better predict drying shrinkage. With developments in SRAs, correction factors to account for the SRAs have been developed in the ALSN 2004 model. Within these models, ACI 209, CEB90, B3 and GL 2000 model are included in the report by ACI committee 209 Creep and Shrinkage in Concrete (ACI Committee 209.2R 2008). In this section, a brief description of each model evaluated in Manuscript 2 was presented. Also some most recent updates and related research work is also provided in this appendix.

A.2 DRYING SHRINKAGE PREDICTING MODELS

A.2.1 **ACI 209**

The ACI 209 model is recommended by the American Concrete Institute, and has been incorporated into many of the building codes in the United States. This model was developed empirically based on shrinkage data obtained prior to 1968 (Al-Manaseer and Lam 2005). The equations can be used to predict the shrinkage of normal weight, lightweight sand and all lightweight concrete. ACI 209 is effective with Type I or Type III cement and for concrete that is either moist cured or steam cured under standard conditions outlined in Table A.1. A correction factor (γ_{sh}) is needed to adjust the ultimate shrinkage strain, $(\varepsilon_{sh})_u$, for conditions other than what is represented in Table B.1 (ACI Committee 209 1992).

Table A.1 Factors Affecting Creep and Shrinkage (Inspired by ACI Committee 209 1992)

Factors			Variables considered	Standard conditions
Concrete (creep and shrinkage)	Concrete composition	Cement paste content Water-cement ratio Mixture proportions Aggregate characteristics Degrees of compaction	Type of cement	Type I and III
			Slump	70 mm
			Air content	≤ 6%
			Fine aggregate percentage	50%
			Cement content	279 to 446 kg/m^3
	Initial curing	Length of initial curing	Moist cured	7 days
			Steam cured	1 to 3 days
		Curing temperature	Moist cured	23.2 ± 2 °C
			Steam cured	≤ 100 °C
		Curing humidity	Relative humidity	≥ 95%
Member geometry and environment (creep and shrinkage)	Environment	Concrete temperature Concrete water content	Concrete temperature	23.2 ± 2 °C
			Ambient relative humidity	40%
	Geometry	Size and shape	Volume-surface ratio or minimum thickness	*V/S* = 38 mm
				150 mm

The ACI 209 model equations are:

Shrinkage after 7 days of moist curing:

$$(\varepsilon_{sh})_t = \frac{t}{t+35} \cdot \ (\varepsilon_{sh})_u$$

Shrinkage after 1-3 days of steam curing:

$$(\varepsilon_{sh})_t = \frac{t}{t+55} \cdot \ (\varepsilon_{sh})_u$$

Where:

t = time after the end of initial wet curing, and;

$(\varepsilon_{sh})_u$ = 780γ_{sh} μm/m.

A.2.2 **CEB 90**

Europeans typically use the prediction method developed in 1990 by the Comité Euro-International du Béton (CEB). The CEB 90 model was derived using mathematical functions instead of tables and figures, and has been optimized from information from a data bank of structural concrete. This model predicts the time dependent deformation of ordinary normal weight concrete exposed to a temperature range of 5 °C and 30 °C and a relative humidity of 40 to 100 %. The CEB 90 equations are:

$$\varepsilon_{cs}\,(t, t_s) = \varepsilon_{cs0} \cdot \beta_s(t - t_s)$$

$$\varepsilon_{cs0} = \varepsilon_s(f_{cm}) \cdot \beta_{RH}$$

$$\varepsilon_s(f_{cm}) = [160 + \beta_{sc} \cdot (90 - f_{cm})] \cdot 10^{-6}$$

$$\beta_{SC} = \begin{cases} 4 & for\ slowly\ hardening\ cement \\ 5 & for\ normal\ or\ rapid\ hardening\ cement \\ 6 & for\ rapid\ hardening\ high\ strength\ cemnt \end{cases}$$

$$\beta_{RH} = \begin{cases} -1.55\beta_{sRH} & 40\% < RH < 99\% \\ 0.25 & RH > 99\% \end{cases}$$

$$\beta_{sRH} = 1 - (\frac{RH}{100})^3$$

Where:

ε_{CS0} = notional shrinkage coefficient;
β_S = coefficient to describe the development of shrinkage with time;
t = age of concrete, days;
t_S = age of concrete at the beginning of shrinkage or swelling, days, and;
β_{SC} = coefficient that depends on cement type.

Factors that can be used to predict drying shrinkage are the compressive strength, dimensions of the member, duration of drying and the relative humidity and temperature of the environment (Muller and Hilsdorf 1990).

Also it should be noted that an updated version of this model (CEB 2012) model will be released in the FIB code soon.

A.2.3 **B3**

The B3 model was developed by Bažant and Baweja was “calibrated by a computerized data bank comprising practically all the relevant test data obtained in various laboratories throughout the world”. Bažant and Baweja state the coefficient of variations for the B3 model are much lower than the CEB 90 model as well as the ACI 209 model. The B3 model prediction equations were designed under certain concrete conditions. These conditions are restricted to portland cement concrete. Other conditions include the w/cm ratio, the aggregate to cementitious ratio and the compressive strength of the concrete at 28 days (Bažant and Baweja 2000).

The B3 model equations are:

Mean Shrinkage Strain:

$$\varepsilon_{sh}(t,t_0) = -\varepsilon_{sh}\infty \cdot k_h \cdot s(t)$$

Time Dependence:

$$s_{(t)} = tanh\sqrt{\frac{t-t_0}{\tau_{sh}}}$$

Humidity Dependence:

$$k_h = \begin{cases} 1-h^3 & for\ h \leq\ 0.98 \\ -0.2 & for\ h = 1.0\ (swelling\ in\ water) \\ linear\ interpolation & for\ 0.98\ \leq h \leq 1.0 \end{cases}$$

Size Dependence:

$$\tau_{sh} = k_t(k_s \cdot D)^2$$

$$D = \frac{2V}{S}$$

$$k_s = \begin{cases} 1.00 & for\ an\ infinite\ slab \\ 1.14 & for\ an\ infinite\ cylinder \\ 1.23 & for\ an\ infinite\ square\ prism \\ 1.30 & for\ a\ sphere \\ 1.55 & for\ a\ cube \end{cases}$$

kt = 8.5t0-0.08fc-1/4 days/mm2

Where:

ε_{sh} = shrinkage strain;
$\varepsilon_{sh\infty}$ = ultimate (final) shrinkage strain;
S(t) = time function for shrinkage;
τ_{sh} = shrinkage half time, days;
f_c = 28-day compressive strength of concrete;
t = age of concrete, days;
t_0 = age of concrete when drying begins;
V/S = volume to surface ratio

h = relative humidity of the environment ($0 \leq h \leq 1$);
k_s = cross section shape factor;
k_t = parameter used to calculate τ_{sh}, and;
k_h = humidity correction factor for final shrinkage.

A.2.4 **GL2000**

The GL 2000 was developed by Gardner and Lockman (Gardner and Lockman 2001) to better predict the shrinkage of concrete. They developed this method since there was not a widely accepted method in existence at the time. This model is effective at predicting shrinkage in normal strength concrete with a 28-day compressive strength of 82 MPA and a w/cm ratio of 0.4 to 0.6.

The GL 2000 equations are:

$$\varepsilon_s(t) = \varepsilon_{shu}\beta(h)\beta(t)$$

$$\varepsilon_{shu} = 1000k \cdot \sqrt{\frac{30}{f_{cm28}}}\ \mu m/m$$

$$k_h = \begin{cases} 1.00 & Type\ I\ cement \\ 0.70 & Type\ II\ cement \\ 1.15 & Type\ III\ cement \end{cases}$$

$$\beta_{(t)} = \sqrt{\frac{t - t_c}{t - t_c + 0.15 \cdot ({}^{V}/_{S})^2}}$$

Where:

h = humidity expressed as a decimal;

t = age of concrete, days;

t_c = age drying commenced, end of moist curing, days;

K = 1 Type I cement;

K = 0.70 Type II cement;

K = 1.15 Type III cement;

V/S = volume-surface ratio, mm, and;

f_{cm28} = concrete mean compressive strength at 28 days, MPa.

If fly ash or slag is blended in the concrete mix, the compressive strength of the concrete should be used to determine which value of K to be used. Gardner and Lockman state that the GL 2000 method can be used to accurately predict the shrinkage regardless of which admixtures, mineral by-products, curing regime or casting temperature (Gardner and Lockman 2001).

A.2.5 **ALSN**

With the introduction of SRAs the GL 2000 equations have become less useful at predicting shrinkage since there is no provision for admixtures. Al-Manaseer and Ristanovic (Al-Manaseer and Ristanovic 2004) have modified the GL 2000 equations to more accurately predict shrinkage of concrete containing SRA. Tests were performed on 27 different concrete mixtures with variations that include fly ash, metakaolin, silica fume, high-range water-reducing admixture, normal-range water-reducing admixture, and SRA.

The ALSN 2004 equations are:

$$\varepsilon_s(t) = \varepsilon_{shu} \cdot \beta(h)\beta(t)\beta(SRA)$$

$$\varepsilon_{shu} = \sqrt{900k\left(\frac{30}{f_{cm28}}\right)}\ \mu m/m$$

$$\beta(h) = 1 - 1.18\left(\frac{h}{100}\right)^4$$

$$\beta(t) = \sqrt{\frac{t - t_c}{t - t_c + 0.12\left(\frac{V}{S}\right)^2}}$$

$$\beta(SRA) = \frac{2}{2 + SRA^{0.7}}$$

Where:

ε_{shu} = ultimate shrinkage strain;
β(h) = humidity correction factor;
β(t) = age correction factor;
β(SRA) = SRA correction factor;

K = shrinkage constant;
f_{cm28} = average 28-day concrete compressive strength (MPa)
H = relative humidity, %;
t = age of concrete after casting, days;
t_c = age the concrete drying commenced, days;
V/S = volume-surface area ratio, mm, and;
SRA = percentage of SRA.

The equation was developed with a mixture design of Type II portland cement w/cm ratio was 0.33 and SRA dosage was 2.5% for all tests. The ALSN 2004 predicts the shrinkage of concrete containing a SRA dosage between 0 and 2.5%. (Al-Manaseer and Ristanovic 2004)

A.3 SUMMARY

The previous sections introduce five different models for predicting shrinkage. It shows different model requires different input factors, in which the limitations of each model lies. Table A.2 gives a summary of the input factors required for predicting shrinkage in each model.

Table A.2 Input factors for predicting shrinkage (inspired by *Al-Manaseer and Lam 2005*)

	ACI209	CEB90	B3	GL2000	ALSN
Relative humidity	X	X	X	X	X
Specimen size	X	X	X	X	X
Specimen shape			X		
Compressive strength fc at 28 day		X	X	X	X
Cement types			X	X	X
Curing types	X		X		
Age at end of curing	X	X	X	X	X
SRA					X
Total parameters required	4	4	7	4	5

A study by Al-Manaseer and Lam (*Al-Manaseer and Lam 2005*) compared the ACI 209 mode, the CEB 90 model, the B3 model, and the GL 2000 model to determine the accuracy of each. The prediction models were compared to the RILEM experimental data bank. Some data points were eliminated because they did not fall into the criteria for the prediction model. Results from this study indicate that ACI 209 and GL 2000 overestimate shrinkage, whereas the CEB 90 and B3 models underestimate shrinkage. Al-Manaseer states that the CEB 90 underestimates because it was designed using finer European cements. When comparing these four methods to the data bank, the B3 and GL 2000 methods performed the best at predicting the shrinkage. (*Al-Manaseer and Lam 2005*) These prediction models are valid for portland cement concrete with limited w/cm ratios.

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