EARLY AGE CREEP AND SHRINKAGE
OF EMERGING CONCRETE MATERIALS

BY

MATTHEW D. D’AMBROSIA

DISSERTATION

Submitted in partial fulfillment of the requirements
for the degree of Doctor of Philosophy in Civil Engineering
in the Graduate College of the
University of Illinois at Urbana-Champaign, 2011

Urbana, Illinois

Doctoral Committee:

Professor David A. Lange, Chair and Director of Research
Professor Leslie J. Struble
Professor Jeffery R. Roesler
Associate Professor John S. Popovics
Professor Emeritus William L. Gamble
Concrete pavements and structures are especially vulnerable to cracking at early age. The volumetric instability of concrete at early age is a frequent cause of cracking. The primary components of volume change are external drying shrinkage, autogenous shrinkage, and thermal dilation. When concrete is restrained, tensile stress develops due to shrinkage and increases the probability of cracking. Early age properties, such as tensile creep, are not well understood and the availability of literature on the subject is limited. The goal of this research is to improve the understanding of early age behavior in emerging materials in order to improve long term durability.

The early age volume changes of self-consolidating concrete (SCC), high-performance concrete (HPC), and concrete with shrinkage reducing admixture (SRA), or shrinkage-reduced concrete (SRC) were studied in order to understand mechanical behavior and develop guidelines for practice. A restrained uniaxial testing frame was previously developed for the purposes of understanding of early age mechanical properties and it was used to explore the role of tensile creep for relaxation of shrinkage stress in materials that are outside the scope of many current prediction models and design guidelines. Tensile creep was compared to compressive creep and up to a tenfold increase was observed, indicating an urgent need for updating models. Other observations, such as non-linearity of creep at early age and under restrained conditions, led to new insights regarding the use of superposition for long term deformations. Experimental characterization of early age behavior aided the development of a new modeling approach based on the utilization of relative humidity (RH) as the driving force for shrinkage. This approach was validated using new experiments developed to characterize tensile creep and autogenous shrinkage, and results demonstrate that RH is a powerful parameter for modeling shrinkage stress development and drying gradients.

Based on the experimental work and modeling efforts, practical guidelines were developed for specifications, mixture proportioning, and acceptance testing, and mitigation strategies were suggested to minimize the potential for shrinkage cracking. Improvements were also suggested for existing prediction models to account for early age behavior. These research contributions enable practitioners to implement new concrete materials technology and realize the benefits of innovative concrete materials without sacrificing long term durability.
To Mom, Dad, and Margie
ACKNOWLEDGMENT

It is with sincere gratitude and appreciation that I wish to acknowledge all the individuals who contributed to the completion of this dissertation. First, to my advisor and mentor, David Lange, thank you for showing me the way and always being optimistic. To my committee members Leslie Struble, John Popovics, Jeff Roesler, and Bill Gamble, thank you for the guidance and support you have given me over the years and for always keeping your doors open.

There were many fellow graduate students who helped me along the way. Thanks to Salah Altoubat, Cheolwoo Park, Anne Werner, Nathan Rau, and Wilkins Aquino for helping me get started by showing me the ropes. Thanks to Zachary Grasley, Chang Joon Lee, Andrew Brinks, and Fernando Tejeda, for the helpful discussion along the way and for contributing their thoughts and ideas to my work. Thanks to Benjamin Birch and Jacob Henschen who helped me to accomplish some of the experimental work as I was nearing completion.

Thanks go to Tim Prunkard, Alan Knell, and all the machine shop staff who helped make my ideas a reality. Thanks also go to Dr. Grzegorz Banas for supervising an exceptional student shop and helping keep away the instrumentation and servo-hydraulic gremlins. Thanks to the outstanding administrative support staff of the department of Civil and Environmental Engineering for always being friendly and helpful.

Thank you to the Illinois Department of Transportation, the Center for Advanced Cement Based Materials (ACBM), BASF (Masterbuilders, Inc.), W.R.Grace, and the Center of Excellence for Airport Technology for the financial support to complete this work.

Finally, thanks to my wife, Margie, and my family: Mark, John, Mom, and especially my Dad. Without your love, guidance, and never-ending support, this accomplishment would not have been possible.
# TABLE OF CONTENTS

1. INTRODUCTION ...................................................................................................................... 1
   1.1 Research Objective ............................................................................................................... 1
   1.2 References ............................................................................................................................. 3

2. BACKGROUND ........................................................................................................................ 4
   2.1 Shrinkage Mechanisms ......................................................................................................... 4
   2.2 Creep Mechanisms ................................................................................................................ 5
   2.3 Emerging Concrete Materials ............................................................................................... 7
      2.3.1 Self-Consolidating Concrete .......................................................................................... 7
      2.3.2 High Performance Concrete ......................................................................................... 10
      2.3.3 The Role of Water to Cementitious Material Ratio and Cement Content ................... 11
      2.3.4 Water Reducers and Superplasticizers ......................................................................... 12
      2.3.5 Supplementary Cementitious Materials ....................................................................... 13
      2.3.6 Shrinkage Reducing Admixtures ................................................................................. 14
   2.4 Mitigation of Cracking ........................................................................................................ 16
   2.5 References ........................................................................................................................... 16

3. MATERIALS ............................................................................................................................ 20
   3.1 SCC Mixture Proportions ................................................................................................... 20
   3.2 HPC Mixture Proportions ................................................................................................... 21
   3.3 SRA Mixture Proportions ................................................................................................... 23
   3.4 Mixing Procedure ................................................................................................................ 24
   3.5 Constituent Materials Characterization ............................................................................... 24

4. EXPERIMENTAL METHODS................................................................................................ 25
   4.1 Development of the Restrained Stress Test Machine (RSTM) ........................................... 25
   4.2 Constant Compressive and Tensile Creep .......................................................................... 29
   4.3 Compressive and Tensile Strength ...................................................................................... 31
   4.4 Elastic Modulus of Concrete in Compression and Tension ................................................ 31
   4.5 Drying and Autogenous Shrinkage Measurements ............................................................. 31
   4.6 Internal Temperature and Relative Humidity ..................................................................... 32
   4.7 References ........................................................................................................................... 34
5. RESULTS ........................................................................................................................................ 37
5.1 SCC ........................................................................................................................................ 37
  5.1.1 Strength Development ........................................................................................................ 38
  5.1.2 Shrinkage and Stress Development .................................................................................... 39
  5.1.3 Creep and Stress Relaxation ............................................................................................. 44
  5.1.4 Summary ........................................................................................................................... 47
5.2 HPC ......................................................................................................................................... 48
  5.2.1 Unrestrained Shrinkage ..................................................................................................... 48
  5.2.2 Weight Loss and Drying Rate ......................................................................................... 55
  5.2.3 Tensile Strength Development ....................................................................................... 58
  5.2.4 Thermal Stress and Hydration Kinetics .......................................................................... 62
  5.2.5 Restrained Stress Development ...................................................................................... 64
  5.2.6 Evolution of Elastic Modulus at Early Age ....................................................................... 67
  5.2.7 Early Age Tensile Creep ................................................................................................. 71
  5.2.8 Discussion of Creep Parameters ...................................................................................... 72
    5.2.8.1 Restrained Tensile Creep Strain .............................................................................. 73
    5.2.8.2 Restrained Tensile Creep Coefficient and Specific Creep ..................................... 74
  5.2.9 Conclusions of HPC Tests .............................................................................................. 75
5.3 SRA ......................................................................................................................................... 85
  5.3.1 Shrinkage and Stress Development ................................................................................ 85
  5.3.2 Discussion of Early Expansion ......................................................................................... 86
  5.3.3 Weight Loss and Drying Rate ......................................................................................... 88
  5.3.4 Hydration Kinetics ........................................................................................................ 89
  5.3.5 Tensile Strength Development ....................................................................................... 89
  5.3.6 Restrained Stress Development ...................................................................................... 89
  5.3.7 Evolution of Elastic Modulus at Early Age ..................................................................... 90
  5.3.8 Early Age Tensile Creep ................................................................................................. 91
  5.3.9 Constant Load Tensile Creep Strain .............................................................................. 92
  5.3.10 Constant Load Tensile Creep Coefficient .................................................................... 92
  5.3.11 Restrained Tensile Creep Strain .................................................................................. 93
  5.3.12 Restrained Tensile Creep Coefficient .......................................................................... 93
1. INTRODUCTION

1.1 Research Objective

The volumetric instability of concrete at early age is a frequent cause of cracking in concrete structures. The primary components of volume change are external drying shrinkage, autogenous shrinkage, and thermal dilation [1-1]. When concrete is restrained, tensile stress develops and increases the probability for cracking. Strength development is critical at early ages when concrete is vulnerable to cracking. Drying shrinkage and tensile creep are especially important if concrete is restrained. Drying shrinkage is the primary driving force for tensile stress development. Tensile creep is beneficial as a stress relaxation mechanism, relieving a portion the stress that develops due to shrinkage. Autogenous shrinkage is significant at early age in low w/cm ratio concrete, thus adding to stress development. Cracking reduces concrete durability by allowing the ingress of water and aggressive ionic species such as deicing salts, thereby enabling other degradation mechanisms such as corrosion, sulfate attack, and alkali-silica reactivity, which cause further cracking, and the cycle progresses. A better understanding of the interactions of the various volume change mechanisms is needed to improve the long term-durability and sustainability of civil infrastructure constructed with concrete.

Recent use of self-consolidating concrete (SCC) and high performance concrete (HPC) has increased, elevating concerns about cracking at early age. High performance concrete is a general term that can apply to many types of concrete mixtures. Concrete is typically considered high performance when it is designed to have properties such as high compressive strength (greater than 6000 psi), high elastic modulus, low permeability, high abrasion resistance, or high flowability. SCC and HPC share several characteristics. Both typically have relatively high paste content and low water to cementitious materials ratio. The cement paste portion affects shrinkage and creep, as well as other mechanical properties such as strength and elastic modulus. Generally, compressive strength is often the only mechanical property evaluated in concrete construction. Shrinkage and creep properties are potentially different and, if not evaluated as part of engineering design, can cause detrimental cracking and loss of long term durability.

Shrinkage reducing admixtures (SRA) are a relatively new type of chemical admixture gaining popularity for their ability to reduce the driving force for shrinkage and mitigate cracking. SRAs
have recently been incorporated into the ASTM C 494-10 specification in a new class called Type S, Specific Performance. Shrinkage reduced concrete (SRC) has many benefits in practice and is especially fitting for use in industrial flooring to reduce the number of joints by increasing spacing. SRAs have been used in structures that are corrosion sensitive in order to reduce the risk of cracking. They are allowed by many state transportation agencies in bridge structures and have been found to reduce both early age and long term shrinkage. SRAs have also been shown to be effective against both drying shrinkage [1-2, 1-3] and autogenous shrinkage [1-4].

A goal of this work is to relate the behavior of these new classes of concrete materials to modern design methods and prediction capabilities. An experimental approach was taken, followed by mechanical modeling of material behavior and interpretation with respect to design and practice.

A comprehensive understanding of early age volume change is essential for improving the durability of concrete structures. To predict cracking, it is necessary to quantify how early age volume changes, such as drying shrinkage, induce stress and how creep mechanisms act to relax part of the stress. Models have been developed that evaluate the creep and shrinkage behavior of concrete. These models were originally intended for use in structural design to properly account for creep and shrinkage due to long term stresses in order to predict deflections. The two most current and widely accepted models are known as the ACI 209 model and the RILEM Draft Recommendation B3 model [1-5, 1-6]. The experimental data used to construct and validate these models was primarily compressive creep results from constant load tests on mature ordinary concrete. To predict early age cracking in concrete, however, tensile creep of early age concrete under restrained conditions should be considered for new classes of materials that redefine the boundaries of practice.

The main research objective of this study is to explore our understanding of mechanical behavior at early ages, particularly creep and shrinkage, of new concrete materials such as SCC, HPC, and SRC. Very little quantitative information is available in the literature regarding tensile creep of concrete. This research establishes the role of tensile creep in relaxation of shrinkage stress for new materials that are outside the scope of many current code models and design guidelines. The experimental work on early age behavior forms the basis for development of a new modeling approach based on the current theories and models in literature. With better prediction
models and guidelines for practice, practitioners may realize the benefit of innovative concrete materials without sacrificing long term durability.

1.2 References


1-5. American Concrete Institute (ACI) 209R-92 (Re-approved 1997) Prediction of Creep Shrinkage and Temperature Effects in Concrete Structures, Farmington Hills, MI.

2. BACKGROUND

Concrete undergoes volumetric shrinkage during external and internal drying. Since aggregate in concrete is typically volumetrically stable, the total shrinkage is a function of the volume of cement paste. External drying is the loss of moisture to the environment through evaporation and diffusion. Internal drying is due to self-desiccation caused by chemical shrinkage during the hydration of portland cement. The mechanisms of shrinkage are not fully understood, due to the dependence on composition and microstructure of cement hydration products. Current accepted theories for shrinkage in cement paste shrinkage state that stress depends on the internal relative humidity. For the typical range of measured internal relative humidity between 50% to 100%, two mechanisms are proposed. Capillary stress is a mechanism that occurs in micropores less than 50 nm and disjoining pressure acts on particles in very close proximity (~2 nm) when attraction results from van der Waals’ forces. The capillary stress mechanism is active until the capillary menisci break, which occurs somewhere between 40-60% RH depending on pore fluid chemistry [2-1]. Concrete is not expected to have internal humidity lower than 50% in most structures.

2.1 Shrinkage Mechanisms

The primary mechanism of shrinkage behavior is described by the model of capillary pressure reduction in cement paste. Hydrostatic tension (under-pressure) develops when a meniscus forms in a capillary as shown in Figure 2-1. The resulting differential pressure depends on surface tension and the radius of the meniscus. The developing pressure is given by the Gauss-Laplace equation according to

\[ P_c = \frac{2\gamma}{R} \]  

(2.1)

where \( P_c \) is the capillary pressure, \( R \) is radius of curvature, and \( \gamma \) is the surface tension of the pore fluid [2-1]. As the internal relative humidity decreases during drying or self-desiccation, the radius of the meniscus decreases, decreasing the internal capillary pressure and drawing the pore walls together. Shrinkage is also affected by disjoining pressure created by water adsorbed on
the surface of the C-S-H gel. As the internal relative humidity decreases, the adsorbed water layer decreases in thickness, reducing the disjoining pressure and allowing the pore surfaces to come closer together due to attraction by van der Waals’ forces. A more thorough description of the mechanisms of shrinkage is given in a review by Grasley et al. [2-3].

Figure 2-1. Capillary pressure mechanism for drying shrinkage in concrete

The driving mechanism for shrinkage is the reduction in internal relative humidity over time due to self-desiccation and external drying. Autogenous shrinkage is driven by the same physical mechanism as drying shrinkage, except that RH reduction is caused by the phenomenon of chemical shrinkage due to the hydration of cement [2-4]. The rate of external drying is a diffusion-controlled process, which is highly dependent on the pore microstructure of hardened cement paste. Therefore, any change in the hardened cement paste that affects the pore microstructure consequently affects drying shrinkage. Early age concrete has a microstructure that is continuously evolving. The relative amounts of capillary porosity and solid hydration product are change as microstructure evolves. As a result, shrinkage rates change over time and early shrinkage trends may be substantially different from long-term trends. Therefore, it is vital to study early age shrinkage behavior in addition to long-term behavior.

2.2 Creep Mechanisms

Creep in concrete is caused by the viscoelastic deformation of the bulk porous hydrated cement paste due to applied stress. Stresses can originate due to internal or external forces, either caused by changes in pore pressure or temperature, or by applied structural loads. Creep is a property of
the paste portion of concrete and is restrained by aggregates, which are relatively stable and do not undergo significant creep [2-1]. The mechanisms of concrete creep have been studied by many researchers and several different mechanisms are thought to be responsible for time dependent deformation [2-5, 2-6]. They are:

- redistribution of capillary water or interlayer water movement (*seepage*),
- sliding or shearing of gel particles lubricated by adsorbed water (*viscous flow*),
- sliding or shearing movement of the molecular structure (*plastic flow*),
- diffusion of solid material, or
- permanent plastic deformation caused by microcracking, re-crystallization or the formation of new physical bonds in the hydration products.

Many of the same concrete properties that effect shrinkage also effect creep, such as internal moisture content, drying rate, and temperature. In literature, creep mechanisms are divided into separate components for drying creep and basic creep. Drying creep was first observed by Pickett and is often referred to as the Pickett effect [2-7]. Although several mechanisms for drying creep have been proposed, none of the theories are universally accepted [2-8]. Basic and drying creep have been separated by experimental methods using superposition analysis, however separation is only valid if the material behaves in accordance with linear viscoelasticity theory (i.e., viscoelastic strain (creep) is linearly proportional to applied stress) [2-9, 2-10].

From a durability viewpoint, creep is beneficial because it reduces tensile stress development that leads to cracking. Admixtures or proportioning methods that reduce creep capacity could have a negative impact on durability by increasing the restrained stress development in a structure. Creep can also be a detrimental behavior, especially for prestressed concrete structures. Creep deformation over time leads to larger deflections and loss of prestressing force, so it may be unfavorable for structural serviceability requirements. Creep is especially critical in super tall buildings and long-span bridges, where creep deflections can play a crucial role in the long term serviceability of a structure.
2.3 Emerging Concrete Materials

2.3.1 Self-Consolidating Concrete

SCC is a high-performance material that is designed to flow into formwork under its own weight. It is considered a new class of concrete due to the need for new methodologies in order to determine the constituent proportions. In the early 2000s, interest in SCC led to the development of dedicated committees in the American Concrete Institute (ACI 237) and the American Society for Testing and Materials (ASTM), as well as dedicated conferences and symposia.

SCC was first used in Japan in the 1980’s to reduce the labor cost associated with consolidation and placement and to reduce defects caused by poor consolidation, with the intent of improving long term durability [2-11]. Since then it has gained international popularity and has been a focus of research interest in North America since the mid-1990s. As of 2011 it is still a popular research topic, but has not gained wide acceptance in the commercial ready mix industry in the United States. It is used in some urban regional markets and has been specified for many large infrastructure projects. It is very popular in precast operations around the nation. Research and development of SCC materials is widespread, but the goal of uniformity and acceptance for practical use has not yet been fully realized.

SCC can be easily placed without vibration or mechanical consolidation, due to high flowability and resistance to segregation. Flowable properties are typically achieved with one or more of the following mix design attributes: high cementitious materials content, superplasticizer or high-range water reducer (HRWR), viscosity modifying admixtures (VMA), mineral admixtures, and careful selection of aggregate volume and gradation. Low aggregate volume and small coarse aggregate top-size are often needed to improve flow between steel reinforcement and to reach restricted areas. To reduce the potential for segregation, mineral admixtures such as silica fume, fly ash, ground-granulated blast furnace slag (GGBFS), calcined clay, and pulverized limestone may be added. VMA can also be added to enhance resistance to segregation or reduce bleeding. The hardened properties of SCC may be significantly influenced by the increase in cementitious materials content, which may affect strength gain, elastic modulus, creep, and shrinkage (autogenous and drying). Although chemical admixtures have been shown to have little impact on hardened properties other than setting time, segregation that occurs in the fresh state can lead
to a poorly consolidated or inhomogeneous material that will have diminished strength and durability.

This study was implemented to investigate the impact of SCC strategies on the hardened properties of SCC. The early age shrinkage and creep as well as strength gain and modulus of elasticity were measured to assess the characteristic behavior of SCC and identify properties, design codes, and model predictions that may need improvement for practical application of this material.

A database of over 150 SCC mixtures obtained from the literature and from industry application was compiled for understanding the major categories of SCC proportioning. The database was used to identify proportioning strategies which may impact hardened properties. Figure 2-2 shows the fine aggregate to coarse aggregate ratio vs. aggregate content for SCC database mixtures. Specific materials that were examined this study are also identified. A shaded oval on the figure represents what are considered normal concrete proportions, according to the ACI method for mixture proportioning. Most notably, the average water to cementitious material ratio ($w/cm$) was 0.41, and the average water to powder ($w/p$) was 0.35, indicating a trend towards lower water contents for SCC. The aggregate content tends to be lower for SCC than for typical concrete, as demonstrated in Figure 2-2. Additional paste volume is needed for improving the flowability and passing ability of SCC. There is also a tendency for a higher proportion of sand in the mixture, which improves segregation resistance.
The hardened properties of SCC mixtures are expected to vary depending on the strategy and proportions for a particular material. Rols et al. measured drying shrinkage of SCC and showed it was 50% greater than ordinary concrete with similar cement content [2-12]. Kim et al. also showed that drying shrinkage can be 30-50% greater for SCC [2-13]. These trends indicate that some SCC mixtures may be susceptible to cracking. Poppe and DeSchutter studied compressive creep and shrinkage of SCC and compared the experimental results with several models from literature [2-14]. They found that the model did not fit for several materials, and related the differences to the presence of superplasticizer and the cement content.

Segregation that occurs in the fresh state can lead to a poorly consolidated or inhomogeneous material that will have diminished strength and durability. Differential shrinkage may occur due to higher paste content in the surface layers of segregated concrete. Abrasion resistance, freeze-thaw durability, and toughness may be affected as well leading to potential long term durability problems. A lack of literature or experimental data on the durability of segregated concrete makes prediction of durability difficult and more research is required to assess the nature of this problem.

Research has shown that VMA does not significantly affect hardened properties. MacDonald et al. studied concretes containing VMA at 3 to 6 times the manufacturers recommended dosage
rate and they observed no significant effects on chemical shrinkage or compressive strength [2-15]. Khayat noted that a slight reduction in strength may be caused by the addition of VMA, but that was most likely due to an increase in entrapped air voids [2-16].

2.3.2 High Performance Concrete

HPC has gained popularity in recent decades in part due to the increasing desire to build more durable concrete structures. Although high performance is often associated with high strength, the terms are not interchangeable. HPC, as defined by the American Concrete Institute (ACI) is “concrete meeting special combinations of performance and uniformity requirements that cannot always be achieved routinely using conventional constituents and normal mixing, placing, and curing practices.” [2-17]. Concrete designed to have enhanced properties over ordinary portland cement concrete (OPC) should be considered high performance. HPC commonly has a lower $w/cm$ ratio than normal concrete. Chemical and mineral admixtures are often added to enhance or control properties such as strength, workability, and setting time [2-18]. As a result, HPC may have drastically different creep and shrinkage behavior than normal concrete. Cracking of HPC may negatively influence durability if shrinkage and creep are not properly accounted for in the design of a structure.

One of the most common reasons for engineers to specify HPC is to improve durability. Many concrete deterioration mechanisms involve the ingress or migration of water containing aggressive ions. Therefore, a material with low permeability is thought to improve concrete durability and lengthen the service life of a structure. A reduction in material permeability is easily achieved with HPC by lowering the $w/cm$ ratio or adding chemical and mineral admixtures that improve consolidation or create a dense microstructure. Unfortunately, cracking may counteract the reduction in permeability provided by HPC, and could eventually lead to a higher overall permeability and reduce the durability of a structure. Cracks provide interconnected channels that facilitate infiltration of water and aggressive ions. The relationship between crack width and permeability was studied by Aldea et al. [2-19]. The water permeability of cracked normal and high strength concrete increased significantly with increasing crack width. For cracks less than 0.2 mm, the permeability coefficient increased within an order of magnitude compared with that corresponding to uncracked material, whereas for crack widths greater than 0.2 mm, water permeability increased rapidly. The effect of cracking on durability is significant
due to a rapid increase in permeability. If crack width is limited, by reducing shrinkage stress or using fiber reinforced concrete, then the impact of cracking on durability is limited.

Cracking of HPC has been attributed to the combined effects of autogenous shrinkage, lower tensile creep and higher tensile stiffness. The effect of autogenous shrinkage is negligible under drying conditions for normal portland cement concrete, but is more significant when considering HPC with low $w/cm$ ratio [2-20]. Autogenous shrinkage is not typically measurable until the $w/cm$ ratio is 0.42 or lower. Thermal stress may also cause cracking in concrete structures due to the relatively high cementitious materials content of HPC. During hydration, thermal stresses are generated due to temperature differences, which are greatest after heat evolution has peaked and the ambient temperature drops rapidly on a cold evening, or when a large temperature gradient is present, such as in mass concrete.

2.3.3 The Role of Water to Cementitious Material Ratio and Cement Content

The most important characteristics of concrete relative to mechanical performance are the $w/cm$ ratio and paste content. A common characteristic of HPC and SCC is a relatively low $w/cm$ ratio. It is well known that as the $w/cm$ ratio decreases, paste capillary porosity decreases, and strength and impermeability increase [2-1]. It is also known that, all other variables held constant, drying shrinkage decreases as the microstructure is densified and the rate of diffusion is reduced. These factors are often touted as major benefits of both HPC and SCC. However, when studying behavior at early age, a larger portion of potential volume change is driven by self-desiccation rather than external drying. Shrinkage and creep are proportional to the paste content and deviations from ordinary concrete proportions can have a significant effect on mechanical performance.

Reducing the cement paste content will reduce shrinkage because aggregate is usually a volumetrically stable material and acts to restrain both shrinkage and creep deformation. Hard and strong aggregates provide the greatest reduction in shrinkage and creep. However, when the $w/cm$ ratio is reduced to provide high strength, it is common to increase the paste content to provide the same level of workability. To minimize shrinkage, workability should be maintained without increasing paste content. Possible solutions are to improve the gradation of coarse aggregate through optimization methods or to add superplasticizer.
Autogenous shrinkage, driven by self-desiccation, contributes to shrinkage, particularly in low \( w/cm \) ratio concrete mixtures (below 0.42 \( w/cm \) ratio). The effect of autogenous shrinkage is particularly evident at early age, when the greatest percentage of cement hydration occurs. Self-desiccation can also occur at higher \( w/cm \) ratios due to the inability of capillary pore water to reach unhydrated cement grains. Diffusion through C-S-H gel is slow and capillary porosity is generally discontinuous at low \( w/cm \), limiting access to water and causing self-desiccation as the unhydrated cement reacts. However, bulk autogenous shrinkage is not usually measurable in concrete with \( w/cm \) greater than 0.40 unless the cement paste content is relatively high.

### 2.3.4 Water Reducers and Superplasticizers

Water reducers and superplasticizers (also called high range water reducers) allow for reduction in the \( w/cm \) by causing cement particles to disperse in the cement paste suspension in the fresh state. Many different types of chemicals are traditionally used for this purpose, including lignin, melamine, sulfonated naphthalene formaldehyde (SNF), and poly carboxylic acid ethers (CAE), also commonly referred to as polycarboxylates. The newest products are the polycarboxylates, reaching the market in the late 1990s in the United States. They weren’t common in commercial ready-mix concrete until midway through the next decade. This new family of superplasticizer is partially responsible for the increase in popularity of SCC, allowing for higher levels of cement dispersion without the severity of common side effects, such as excessive bleeding, segregation, and set retardation. These side effects still occur, only at much higher dosage rates. Polycarboxylates have also demonstrated less sensitivity to cement chemistry than older formulations, although there is still variability, primarily caused by changes in alkali content [2-21].

Various researchers have studied the effects of water reducers and superplasticizers on concrete creep and shrinkage [2-21 to 2-24]. The results of various types of testing performed are sometimes conflicting, but more often than not, water reducers and superplasticizers are associated with increased creep and shrinkage of concrete. To the extent that their purpose is also to allow for a lower \( w/cm \) ratio, the effect is probably offset somewhat by the development of a dense microstructure. The magnitude of this effect also varies depending on the admixture type. Low and mid-range water reducers are more often associated with higher shrinkage than
superplasticizer. The reason for this difference is not fully understood, but can possibly be attributed the molecular size and the thickness of the layer that forms around each cement particle during the dispersion process. This layer may then have an effect on pore structure or composition of hydration products.

The effect of water reducers and superplasticizers on creep is not well understood. Khan et al. observed that creep of concrete containing superplasticizer was much more sensitive to the age of loading than plain concrete [2-25]. A higher magnitude of early creep was observed, which has the potential to relieve shrinkage stress and reduce cracking.

Common practices may include a combination of admixtures to provide the greatest reduction in slump at the lowest cost [2-18]. Other admixtures are often used to control setting time and other concrete properties. Such practice makes the effects virtually impossible to predict and testing must be performed to ensure sufficient quality is met.

2.3.5 Supplementary Cementitious Materials

Supplementary cementitious materials (SCMs) such as fly ash, silica fume, ground granulated blast furnace slag, and metakaolin have been evaluated for their effect on creep and shrinkage of concrete. Conflicting results have been reported. Some researchers reported increases in drying shrinkage due to SCMs [2-23, 2-26-2-28], while others claim a significant decrease [2-24, 2-29, 2-30]. A possible explanation for the discrepancy is that several competing mechanisms affect shrinkage behavior. At lower w/cm ratio, autogenous shrinkage plays a much greater role in the overall shrinkage behavior. SCMs such as silica fume increase the amount of autogenous shrinkage considerably when compared to plain portland cement [2-20]. SCMs also lead to the development of finer capillary pores in mature concrete, which can increase capillary stress according to Equation 2.1, and lead to higher shrinkage [2-20]. Smaller pores, however, also reduce the diffusion rate and lead to less drying shrinkage. Therefore, the results are dependent on the age of testing and the w/cm ratio. SCMs in general will all have the same effect, due to pozzolanic reactivity, although the rate of densification of the cement paste with secondary C-S-H will vary depending on the reactivity of the pozzolan.
2.3.6 Shrinkage Reducing Admixtures

The driving force for shrinkage stress and cracking in concrete can be reduced physically through the use of SRA. The mechanism of shrinkage reduction is attributed to a reduction in the surface free energy of the liquid/solid interface (surface tension) in a partially saturated pore, which then causes a reduction in the stress generated by capillary tension during the drying process [2-46]. This mechanism can be demonstrated using Equation 2.1. It can be observed that as the surface tension (γ) decreases, the capillary pressure (Pc) decreases proportionally. Uniaxial, free shrinkage and restrained ring tests have demonstrated that SRA reduces the driving force for cracking [2-31 to 2-36]. However, the influence of SRA on early age stress development and creep mechanisms is not well understood.

SRA composition varies depending on the manufacturer, but it generally consists of a surface-active organic polymer solution that acts to reduce surface tension and thereby reduce capillary stress [2-37]. Nihon Cement and Sanyo Chemical companies of Japan developed the first admixture of this type in the early 1980’s. BASF Admixtures (formerly Master Builders, Inc.), now markets a similar chemical in the United States under the name Tetraguard. Later in the 1980’s W.R. Grace developed a similar admixture, which is marketed under the name Eclipse. The chemical systems are generally described as polyoxyalkylene ethers, low molecular weight oxyalkylene compounds and a comb-polymer having carboxylic acid groups and oxyalkylene units therein. The precise chemical compositions are beyond the scope of this work, but this information is disclosed in the patent documents [2-38, 2-39].

Early research on the effect of SRA on concrete focused on free shrinkage measurements and simple qualitative evaluation of cracking. Tomita et al. reported the benefits of shrinkage reduction due to mixing polyoxyalkylene alkyl ether in fresh concrete [2-40] and impregnating the admixture into the surface of hardened concrete [2-41]. Experiments included free shrinkage measurements, qualitative restrained shrinkage tests to measure time to cracking, and warping tests of concrete slabs. Variations in w/cm ratio, mix proportions, curing conditions, and admixture dosages were examined in this series of publications. Several formulations were tested and the developers optimized the admixture to maximize the shrinkage reduction while at the same time, reduce any side effects on other concrete properties. Reported effects are retardation of setting time, reduction in both early and long-term concrete strength and instability.
of the entrained air void system [2-41, 2-42]. In some cases, the air content reportedly decreased and caused a subsequent reduction in freeze thaw resistance. However, other researchers have reported additional entrained air and the dosage of air entraining agent may be reduced. Each product should be evaluated carefully to determine if there exists an effect on entrained air and assistance should be sought from the manufacturer for difficulties entraining air. The true nature of any effect on air entrainment is likely related to cement chemistry, as well as supplementary cementitious materials content, especially alkali levels.

In testing performed by Shah et al. using free shrinkage and ring-shaped restrained specimens, the results clearly demonstrated the effectiveness of SRA for reduction of both shrinkage strain and cracking. The results indicated that the addition of three different SRA types caused a significant reduction in free shrinkage and width of cracking due to restrained shrinkage. Folliard and Berke measured the effect of a propylene glycol type SRA on HPC containing silica fume and also found a significant reduction in free shrinkage and restrained cracking using the ring test [2-42]. They also found a slight strength loss with SRA, as well as some water reduction ability. A synergistic relationship between SRA and silica fume was reported, which could be attributed to the ability of SRA to reduce autogenous shrinkage as well as drying shrinkage. Superplasticizing effects of SRA have also been reported by Gettu et al. using a polyethylene glycol formulation [2-43]. In another paper by Berke et al., larger restrained slabs were tested and qualitative results (cracking tendency) indicated both a significant reduction in restrained shrinkage cracking and shrinkage gradient [2-44].

In a study of the effect of SRA on the setting time of concrete, Brooks et al. determined that there was no significant effect [2-45]. Others have confirmed this finding. The SRA type was not reported in this case, as in some other cases. Since there are several different SRA chemical types with a wide range of formulations, it is difficult to determine which SRA chemicals are responsible for a particular behavior, as is also the case with the effect on air entrainment.

Bentz et al. reported that SRA accelerated the drying of a bulk liquid solution but reduced the drying rate of cement paste [2-46]. The surface tension measurements of the bulk solution were significantly lower than distilled water. This explains the possibility of accelerated drying of bulk solution, but does not explain the reduced drying rate of cement paste. Autogenous shrinkage was also reduced when SRA was present. The degree of hydration was reportedly
unaffected in this study. Other researchers have reported little or no effect on the drying rate [2-36].

2.4 Mitigation of Cracking

High early shrinkage combined with lower creep and higher stiffness (all characteristics of low w/cm concrete) can significantly increase the possibility of cracking. Autogenous shrinkage can be significant when considering low w/cm ratio and can be increased even further with the addition of silica fume. On the other hand, increasing the hardened cement paste portion of concrete can produce increases in both shrinkage and creep. It is evident from this review that the design of concrete to minimize cracking is not trivial. A balance must be obtained when selecting materials. While strength is an important design parameter for concrete, shrinkage and creep behavior can be just as important (if not more) when considering concrete that is restrained at early age. SRA is a proven method for shrinkage and crack reduction, but is not always cost effective. So in order to improve the crack resistance of new concrete materials, laboratory testing is required to evaluate a concrete mixture for performance. Test methods must be properly selected so that cracking performance can be evaluated. This study utilizes several effective test methods to evaluate early age creep and shrinkage and give suggestions for modeling the behavior and prediction of field performance.

2.5 References


3. MATERIALS

The concrete mixtures used in this study were proportioned using guidelines obtained from industry publications, research sponsors such as IDOT, and published literature. Constituent materials were chosen based on the locally available sources in Illinois and the IDOT approved materials list. For the high performance concrete portion of the study, the mixtures correlated with relevant IDOT bridge deck construction projects. For the SCC work, mixtures were selected to represent different strategies for proportioning SCC, based on examples and guidelines in literature and were developed with input from IDOT. For the SRA study, the mixtures were developed in conjunction with the sponsors of the work. Mixtures used Class C fly ash to represent typical SCM usage in the state of Illinois.

3.1 SCC Mixture Proportions

Four SCC mixtures were selected that represent different strategies of SCC proportioning. The proportions of the mixtures to be examined in this study and the fresh concrete properties are shown in Table 3-1. SCC 1 is a mixture with two coarse aggregate sizes used to obtain a uniform gradation, SCC 2 has increased paste content using mineral filler (fly ash, slag or limestone powder), SCC 3 contains a VMA, and SCC 4 is a high strength mixture with two coarse aggregates. Also included in this study is a standard precast beam mixture for comparison, designated OPC1. The precast mixture was designed to sufficiently flow into beam formwork and through tight reinforcement with the aid of mechanical vibration. It has a similar cement paste content and similar strength to most of the SCC materials studied. All of the SCC mixtures used polycarboxylate-based (CAE) superplasticizers to attain flowable characteristics without mechanical vibration and without segregation. The mixtures do not contain air entraining agent, but entrapped air content was measured.
<table>
<thead>
<tr>
<th></th>
<th>SG</th>
<th>UNIT</th>
<th>OPC 1</th>
<th>SCC 1</th>
<th>SCC 2</th>
<th>SCC 3</th>
<th>SCC 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement (Type I)</td>
<td>3.15</td>
<td>lb/yd³</td>
<td>726</td>
<td>661</td>
<td>601</td>
<td>685</td>
<td>679</td>
</tr>
<tr>
<td>Fly Ash (Class C)</td>
<td>2.65</td>
<td>lb/yd³</td>
<td>0</td>
<td>157</td>
<td>325</td>
<td>0</td>
<td>151</td>
</tr>
<tr>
<td>Coarse Aggregate, 3/4&quot; (20mm)</td>
<td>2.70</td>
<td>lb/yd³</td>
<td>1853</td>
<td>367</td>
<td>1365</td>
<td>1627</td>
<td>579</td>
</tr>
<tr>
<td>Coarse Aggregate, 3/8&quot; (10mm)</td>
<td>2.70</td>
<td>lb/yd³</td>
<td>0</td>
<td>1075</td>
<td>0</td>
<td>0</td>
<td>1018</td>
</tr>
<tr>
<td>Fine Aggregate (FM = 2.57)</td>
<td>2.64</td>
<td>lb/yd³</td>
<td>1192</td>
<td>1403</td>
<td>1336</td>
<td>1389</td>
<td>1389</td>
</tr>
<tr>
<td>Water</td>
<td>1.00</td>
<td>lb/yd³</td>
<td>290</td>
<td>311</td>
<td>301</td>
<td>278</td>
<td>267</td>
</tr>
<tr>
<td>Superplasticizer (CAE)</td>
<td>1.06</td>
<td>fl oz/yd³</td>
<td>22</td>
<td>63</td>
<td>29</td>
<td>49</td>
<td>36</td>
</tr>
<tr>
<td>Viscosity Modifying Admixture (VMA)</td>
<td>1.00</td>
<td>fl oz/yd³</td>
<td>22</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Slump flow (standard slump for OPC)</td>
<td></td>
<td>in</td>
<td>5</td>
<td>30</td>
<td>28</td>
<td>26</td>
<td>27</td>
</tr>
<tr>
<td>Paste content by Volume</td>
<td></td>
<td>%</td>
<td>32</td>
<td>37</td>
<td>40</td>
<td>33</td>
<td>34</td>
</tr>
<tr>
<td>FA/CA ratio</td>
<td></td>
<td></td>
<td>0.64</td>
<td>0.97</td>
<td>0.98</td>
<td>0.85</td>
<td>0.87</td>
</tr>
<tr>
<td>w/cm</td>
<td></td>
<td></td>
<td>0.40</td>
<td>0.38</td>
<td>0.33</td>
<td>0.41</td>
<td>0.32</td>
</tr>
</tbody>
</table>

3.2 HPC Mixture Proportions

The HPC mixtures used in this study reflect current IDOT practice for HPC and conventional (OPC) projects. In 2001, IDOT BMPR identified five mixtures representing the current use of HPC as well as one OPC conventional deck mixture. These mixtures are presented in Table 3-2. The mixtures are denoted with codes IHPC1 through IHPC4 and then ISTD to represent a standard concrete mixture. IHPC1 and IHPC2 are the same mixture except that silica fume was used in place of high reactivity metakaolin (HRM). IHPC3 had lower cementitious materials content, higher fine aggregate proportion, and a higher percentage of silica fume by weight of cementitious materials. IHPC4 had more cement and no fly ash, with a slightly higher proportion of fine aggregate. The conventional mixture, ISTD, did not contain mineral admixtures, water reducer, or superplasticizer.
Table 3-2. High Performance Concrete Mixtures

<table>
<thead>
<tr>
<th></th>
<th>SG</th>
<th>Unit</th>
<th>IHPC1</th>
<th>IHPC2</th>
<th>IHPC3</th>
<th>IHPC4</th>
<th>ISTD</th>
<th>IDL44</th>
<th>ISL44</th>
<th>IBL44</th>
<th>IKL44</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement (Type I)</td>
<td>2.65</td>
<td>lb/yd³</td>
<td>465</td>
<td>465</td>
<td>445</td>
<td>565</td>
<td>605</td>
<td>515</td>
<td>465</td>
<td>545</td>
<td>445</td>
</tr>
<tr>
<td>Fly Ash (Class C)</td>
<td>2.50</td>
<td>lb/yd³</td>
<td>120</td>
<td>120</td>
<td>90</td>
<td>140</td>
<td>145</td>
<td>90</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Metakaolin</td>
<td>2.70</td>
<td>lb/yd³</td>
<td>27</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Coarse Aggregate, 3/4” (20mm)</td>
<td>2.67</td>
<td>lb/yd³</td>
<td>1820</td>
<td>1820</td>
<td>1820</td>
<td>1820</td>
<td>1863</td>
<td>1866</td>
<td>1820</td>
<td>1811</td>
<td></td>
</tr>
<tr>
<td>Fine Aggregate</td>
<td>2.60</td>
<td>lb/yd³</td>
<td>1095</td>
<td>1095</td>
<td>1200</td>
<td>1150</td>
<td>1130</td>
<td>1108</td>
<td>1130</td>
<td>1240</td>
<td>1230</td>
</tr>
<tr>
<td>Water</td>
<td>1.00</td>
<td>lb/yd³</td>
<td>269</td>
<td>268</td>
<td>246</td>
<td>260</td>
<td>266</td>
<td>288</td>
<td>279</td>
<td>251</td>
<td>246</td>
</tr>
<tr>
<td>AEA (Grace Daravair 1400)</td>
<td>1.10</td>
<td>fl oz/yd³</td>
<td>3.06</td>
<td>3.1</td>
<td>2.8</td>
<td>3.0</td>
<td>1.2</td>
<td>4.7</td>
<td>6.4</td>
<td>5.8</td>
<td>5.7</td>
</tr>
<tr>
<td>Type A WR (Grace Daracem 65)</td>
<td>1.14</td>
<td>fl oz/yd³</td>
<td>17.7</td>
<td>17.7</td>
<td>16.2</td>
<td>20.7</td>
<td>9.6</td>
<td>8.6</td>
<td>8.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Type D WR+R (Grace Daratard 17)</td>
<td>1.17</td>
<td>fl oz/yd³</td>
<td>26.3</td>
<td>26.2</td>
<td>24.1</td>
<td>25.4</td>
<td>18.2</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Type F HRWR (Grace Daracem 19)</td>
<td>1.20</td>
<td>fl oz/yd³</td>
<td>28.2</td>
<td>30.5</td>
<td>28.0</td>
<td>29.5</td>
<td></td>
<td>39.3</td>
<td>35.3</td>
<td>34.7</td>
<td></td>
</tr>
<tr>
<td>Paste content by Volume</td>
<td>%</td>
<td></td>
<td>0.60</td>
<td>0.60</td>
<td>0.66</td>
<td>0.63</td>
<td>0.62</td>
<td>0.59</td>
<td>0.61</td>
<td>0.68</td>
<td>0.68</td>
</tr>
<tr>
<td>w/cm</td>
<td></td>
<td></td>
<td>0.44</td>
<td>0.44</td>
<td>0.44</td>
<td>0.44</td>
<td>0.44</td>
<td>0.44</td>
<td>0.44</td>
<td>0.44</td>
<td>0.44</td>
</tr>
</tbody>
</table>

IHPC1 and IHPC2 were used in the construction of two concrete bridge decks on US-51 in IDOT District 5 (Macon, IL). IDL44 was used in the Duncan Rd bridge over I-72 in IDOT District 5 (Champaign, IL), ISL44 was used in the I-55 Bridge over Lake Springfield in IDOT District 6 (Springfield, IL), IBL44 was used in the I-70 bridge over Big Creek in IDOT District 5 (Clark Co.), and IKL was used in the US-51 bridge over the Kaskaskia River in IDOT District 7 (Vandalia, IL). The cement, mineral admixtures, and aggregates used in the bridge projects were retrieved for laboratory testing. Chemical admixtures were obtained directly from the supplier and not from the ready mix plant. Specimens tested using field materials were given the designation IHPC1-F, IHPC2F, etc.

The Duncan Rd mixture is a conventional design containing fly ash and does not contain silica fume or superplasticizer. The Lake Springfield mixture had a lower cementitious material content and contained silica fume in addition to cement. Big Creek and Kaskaskia mixtures are optimized aggregate mixtures, with 50% of coarse aggregate passing the ½” sieve. Improving the aggregate gradation allowed for lower paste content to achieve the same workability.

The mixtures in this portion of the study do not contain a set retarding admixture. This admixture, when used in practice, allows more time for placement and finishing, especially in warmer weather. However, when used at the same dosage in laboratory conditions, problems arose with low early strength gain. Hydration and strength gain were more severely retarded in the laboratory, due to the lower temperatures (~73°F) compared to the field conditions during
some bridge deck installations (80-90°F). Early shrinkage stress development exceeded tensile strength in the first hours of testing and the specimens fractured. The decision was made to compare the remaining mixtures without retarder to avoid early test failure.

### 3.3 SRA Mixture Proportions

Fourteen different concrete mixtures were examined in a two-part study. Four mixtures were tested in Phase I with mixture proportions shown in Table 3-3. They consisted of one OPC mixture and one HPC mixture containing silica fume, Class F fly ash, and superplasticizer. These mixtures were studied with and without SRA at 0.75 gal/yd³ (~ 1% cwt). Type I portland cement was used with a \( \text{w/cm} \) of 0.45 and 0.35. Both concrete mixtures contained an air-entraining admixture.

Concrete with \( \text{w/cm} \) ratios of 0.40 and 0.50 were tested with two SRA types and two dosages for each type. The mixtures have equal paste content to evaluate the effect of SRA on different \( \text{w/cm} \) ratio concrete. To maintain constant slump, a superplasticizer was used in the 0.40 \( \text{w/cm} \) mixture. The proportions are presented in Table 3-3. Two SRA types were used in this phase of the project.

**Table 3-3. Concrete Mixture Proportions for SRA Study**

<table>
<thead>
<tr>
<th></th>
<th>Phase I</th>
<th>Phase II</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>SG</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cement (Type I)</td>
<td>3.15 lb/yd³</td>
<td>674</td>
</tr>
<tr>
<td>Fly Ash (Class F)</td>
<td>2.5 lb/yd³</td>
<td>50</td>
</tr>
<tr>
<td>Silica Fume (dry densified)</td>
<td>2.2 lb/yd³</td>
<td>76</td>
</tr>
<tr>
<td>Coarse Aggregate, 3/4&quot; (20mm)</td>
<td>2.67 lb/yd³</td>
<td>1710</td>
</tr>
<tr>
<td>Fine Aggregate (FM = 2.57)</td>
<td>2.6 lb/yd³</td>
<td>1112</td>
</tr>
<tr>
<td>Water</td>
<td>1 lb/yd³</td>
<td>280</td>
</tr>
<tr>
<td>1% SRA 1 (MBT Tetraguard AS21)</td>
<td>0.99 fl oz/yd³</td>
<td>125</td>
</tr>
<tr>
<td>2% SRA 1 (MBT Tetraguard AS21)</td>
<td>0.99 fl oz/yd³</td>
<td>256</td>
</tr>
<tr>
<td>1% SRA 2 (Grace Eclipse Plus)</td>
<td>0.96 fl oz/yd³</td>
<td>128</td>
</tr>
<tr>
<td>2% SRA 2 (Grace Eclipse Plus)</td>
<td>0.96 fl oz/yd³</td>
<td>256</td>
</tr>
<tr>
<td>AEA (MBT Microair)</td>
<td>1.01 fl oz/yd³</td>
<td>1.6</td>
</tr>
<tr>
<td>Type F Superplasticizer</td>
<td>1.2 fl oz/yd³</td>
<td>33.3</td>
</tr>
<tr>
<td>Paste content by Volume</td>
<td>%</td>
<td>39, 34</td>
</tr>
<tr>
<td>FA/CA ratio</td>
<td>%</td>
<td>0.65, 0.71</td>
</tr>
</tbody>
</table>

\[ \text{w/cm} \]

<table>
<thead>
<tr>
<th>0.35</th>
<th>0.45</th>
<th>0.40</th>
<th>0.50</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.35</td>
<td>0.45</td>
<td>0.40</td>
<td>0.50</td>
</tr>
</tbody>
</table>
3.4 Mixing Procedure

The laboratory mixing procedure for all testing was based on the procedure recommended by IDOT BMPR. The sequence is used by IDOT for silica fume or HRM mixtures, but for this study it was used for all laboratory mixtures. The procedure is based on AASHTO T 126 (3 minute mixing, 3 minute rest, and 2 minute mixing), varied slightly to simulate the sequence of mixing in the field and to ensure that the silica fume/HRM is thoroughly distributed. A three cubic foot Lancaster pan mixer was used in this study.

The following procedure was followed for each batch of concrete:

- Add CA, FA, and most of mixing water, and mix for 30 seconds to allow water to be absorbed into any aggregate that may be drier than SSD. This prevents AEA from being absorbed into dry aggregate
- Add AEA and wash out beaker with remaining mix water, then mix for 30 seconds
- Add cement and mix for 3 minutes
- Within first 1½ minutes, add water-reducer, retarder and wash out beakers (if applicable)
- Rest for 3 minutes
- Resume mixing
- Begin adding silica fume or HRM at 30 seconds
- Mix for 1 minute
- Add superplasticizer SLOWLY until desired slump is attained
- Mix for additional 2 minutes
- Take air content and slump readings and record
- Adjust as necessary; if water or admixtures are added, then mix for additional 1 – 2 minutes

3.5 Constituent Materials Characterization

The aggregates and cementitious materials used in this study were characterized both physically and chemically. The chemical oxide compositions for portland cement and supplementary cementitious materials were determined by x-ray fluorescence. Aggregate gradation was determined by sieve analysis according to ASTM C 136. The results for x-ray florescence analysis and aggregate gradation are presented in Appendices A and B respectively.
4. EXPERIMENTAL METHODS

The state of the art technique for quantifying stress associated with externally restrained shrinkage is a uniaxial frame that is capable of applying a simulated, fully restrained load. These systems allow determination of the stress accumulation and creep relaxation under restrained condition. Passive systems, such as ring tests and solid test frames are not included in this review because they are unable to simulate full restraint and quantify the resulting stresses, although they are useful for quantifying the relative time to cracking between materials under the same testing conditions.

4.1 Development of the Restrained Stress Test Machine (RSTM)

The measurement of restrained shrinkage stress in concrete is critical to determining susceptibility to early age cracking. Conventional methods for measuring tensile creep under sustained loads, such as the experiment by Bissonnette and Pigeon are not capable of providing full restraint and the measurements are dependent on the stiffness of the testing apparatus [4-1]. However, in the past 15 years, new testing techniques to measure restrained shrinkage stress have been developed.

Paillère et al. [4-2] used a uniaxial system to measure the stress developed due to restrained shrinkage. The system was developed at Laboratorie Central des Ponts et Chaussées (LCPC). A uniaxial specimen with flared ends was cast into a frame that applied a restraining force by means of an air pump. Tensile stress was then measured with a load cell and deformation was monitored as the load was applied manually to produce a restrained condition. This test was performed both vertically and horizontally depending on the age of the specimen. It was determined that a vertical test was problematic due to the dead load and fragility of the specimen.

Bloom and Bentur [4-3] developed a similar system in which an electronic step motor was used to apply the restraining load. Two flared end specimens were measured for simultaneous determination of free shrinkage and stress development. Creep was calculated as the difference in strain accumulation between the two specimens. Kovler [4-4] further modified this system to include a closed-loop computer control system, and measured deformation with LVDT sensors instead of conventional dial gages. When the deformation reached a predefined threshold, a
restraining force was applied automatically to deform the specimen to its original length. Many researchers have since adopted this method of simulating full restraint in a laboratory test. This experimental device has been used to investigate drying creep, autogenous shrinkage, and internal curing with lightweight aggregates [4-4 to 4-10].

A test developed by Pigeon et al. [4-11] based on Kovler’s system measured the stress due to restrained autogenous shrinkage. This experiment also used a computer controlled loading system. Deformation was measured using a direct current displacement transducer. Springenschmid et al. [4-12] developed the Temperature Stress Testing Machine (TSTM) to measure the tensile stress in concrete due to the heat of hydration and external temperature change. Attached to one end of a uniaxial concrete specimen was an adjustable crosshead. A computer controlled step motor applied a load to control the deformation of the concrete specimen as it reached a threshold of 0.001 mm (0.00004 in).

Van Breugel and de Vreis developed a TSTM similar to Springenschmid et al, except that it used a hydraulic actuator to apply load [4-13]. The device was used in conjunction with an autogenous deformation testing machine (ADTM) to optimize HPC mixture proportions based on creep and shrinkage performance. This device has been used by Lura et al. to study the effect of curing temperature and type of cement on early-age shrinkage of HPC [4-14].

The Restrained Stress Testing Machine (RSTM) currently in use at the University of Illinois at Urbana-Champaign uses a specimen size of 1000x75x75 mm (39x3x3-in.) to accommodate concrete with up to 25mm (1 in) maximum size coarse aggregate. The applied load is generated using a servo-hydraulic actuator with high load stability that is capable of load application up to 90 kN (20 kip). An extensometer anchored in the specimen was used to avoid grip-specimen interaction, which caused inaccurate strain measurements in preliminary tests [4-17]. Deformation is measured with an LVDT and fed into a closed loop system that controlled the applied load to the specimen, which was measured with a load cell. A threshold value of 0.005 mm (8 microstrain) is used to simulate restraint. This value was determined experimentally to be the minimum effective value within the limitations of the measuring equipment and environmental conditions. This system has been used to study drying creep, fiber reinforced concrete, and the effects of SRA on creep and stress development [4-15 to 4-20].
The RSTM provides simultaneous measurement of unrestrained drying shrinkage and restrained drying shrinkage and creep deformation. The RSTM can also be used to perform conventional constant load tensile and compressive creep tests. For each experiment, two companion specimens were cast in a temperature and humidity controlled environmental chamber. The unrestrained specimen, shown in Figure 4-1, is used to measure free shrinkage, while the restrained specimen, shown in Figure 4-2, is manipulated to simulate full restraint. The specimen is connected to a hydraulic actuator through specially designed end grips that transmit load without causing stress concentrations. The system operates with a feedback controlled computer program written in LabView, which communicates with an Instron controller. The conditions during testing were maintained at 23°C (±0.5°C) and 50% (±5%) relative humidity. The dimensions of each specimen are given in Figure 4-3. Typical test data, shown in Figure 4-4, can then be analyzed using a subroutine written in Visual Basic called *Restrained Test Analysis (ReTA)*. The program calculates creep using iterative superposition and then computes useful parameters such as creep compliance and creep coefficient. The code for this program is given in Appendix C.

![Figure 4-1. RSTM free shrinkage specimen and LVDT extensometer](image)

**Figure 4-1.** RSTM free shrinkage specimen and LVDT extensometer
Figure 4-2. RSTM assembly showing stainless steel formwork, instrumentation, and servo-hydraulic system

Figure 4-3. RSTM companion specimens for determination of shrinkage and creep
4.2 Constant Compressive and Tensile Creep

Constant load compressive creep was measured according to ASTM C 512. The apparatus for this experiment utilizes a static pressure hydraulic ram to compress heavy gauge springs which are fixed in a frame with the concrete cylinders. As the hydraulic pressure increases, compressive load is applied to the concrete cylindrical specimens, as shown in Figure 4-5, and then the plates are tightened to lock the spring in place. The spring is necessary to maintain constant stress in the concrete samples as creep deflection occurs. The load was checked periodically with the hydraulic ram and a pressure gauge to verify the applied stress level was constant.

To measure constant load tensile creep, a new device was constructed using a dead load (DL) lever arm as shown in Figure 4-6. The DL apparatus consists of heavy gauge steel plate welded to a ½-in. thick steel channel section. The lever arm is constructed with aluminum tube to apply a constant static load using 18:1 magnification. Using a reasonable amount of weight applied to
the frame (up to 200 lb.), the magnification allows for applied loads of up to 3600 lb. Using a specimen size of 3x3x11-in. (75x75x250 mm), an applied stress of 400 psi in tension is possible. Strain measurements were obtained using embedment strain gages and collected continuously for both experiments. Two types of strain gages were used, depending on the specimen size, as shown in , the 4-in. gage length was used for 6x12-in. and 4x8-in. cylinders (Vishay EGP-5), and a 2-in. gage length (Texas Measurement Inc., PMFL-50-2LT) for 3x6-in. cylinders.

Figure 4-5. Compressive creep test frames and data acquisition system

Figure 4-6. DL tensile creep test frames and data acquisition system
4.3 Compressive and Tensile Strength

Early age strength development of 4x 8-in. (100x200 mm) cylinders were measured according to ASTM C39 and C496 for compressive strength and indirect tensile strength respectively. Test specimens were kept in a temperature and humidity controlled environmental chamber at 23°C (±0.5° C) and 50% (±5 %) relative humidity. Compressive strength was also measured for some mixtures on a separate set of 4 x 8-in. (100x200 mm) cylinders kept at moist cured conditions at 23°C (±2° C).

4.4 Elastic Modulus of Concrete in Compression and Tension

The elastic modulus was measured in two ways. Compressive elastic modulus was determined using 4x8-in. cylinders according to ASTM C 469. The evolution of stiffness over time was studied at 1, 3, 5, 7, 14, and 28 days for each material. Elastic modulus in tension and compression was also studied in the RSTM, allowing for the investigation of stiffness corresponding to variable stress histories and ages.

4.5 Drying and Autogenous Shrinkage Measurements

Long term drying shrinkage for concrete mixtures in this study were measured in general accordance with ASTM C 157, with a modification made to the procedure to allow early age measurement by removing the curing requirement. Standard prism specimens were fabricated, having dimensions 3x3x11.25” (76.2x76.2x285mm). The wet curing period was not provided so drying shrinkage could be measured starting at 24 hrs. Prisms were sealed on two sides to
provide two sided drying and an identical surface to volume ratio as the RSTM free shrinkage specimen. Length change was measured using a standard commercial length change comparator from Humboldt Manufacturing and a Mitutoyo digital indicator with 0.0001-in resolution. Standard end pins were embedded in the prisms to provide a reference gage point and an invar rod was used prior to each measurement to zero the length change comparator. It is important to note that when early shrinkage measurements are performed, care should be taken not to damage the end pins. Samples should be carefully checked to ensure that end pins are not damaged prior to the initial reading. A fast setting epoxy can be used to repair damaged pins, but this must be done prior to the initial reading so that all subsequent readings are consistent.

Autogenous shrinkage was measured using 3x3x11-in. (75x75x250 mm) prisms or 4x8-in cylinders in a sealed plastic or foil covered container and embedment type strain gages. The specimens are sealed at casting with adhesive aluminum foil to prevent moisture loss to the environment and continuously maintained at 23°C. It was determined in preliminary experiments that even brief drying or evaporative cooling during demolding of specimens can cause significant deformation (20-30 με) in early age concrete relative to autogenous dilation. Therefore it is preferred that specimens be permanently sealed from casting. Temperature is measured in the specimens so that thermal dilation due to hydration heat evolution may be included in the analysis.

### 4.6 Internal Temperature and Relative Humidity

The internal temperature and relative humidity were measured using a computer controlled measurement system designed by Grasley et al. [4-21]. The probes are relatively inexpensive compared with other testing techniques, which allows them to be embedded into concrete without the need for recovery. They can also be reused after a conditioning process [4-21], which consists of oven drying the sensors to remove all condensed vapor from the polymer film. Although lab experience has shown this reconditioning may not always be reliable, the sensor calibration can be verified prior to use after the reconditioning process is complete. Some sensors, after reconditioning were unable to register RH above 85% and they were discarded. Measurements were sampled every ten minutes from the time of casting. The internal specimen temperatures were used to observe hydration heat evolution and evaporative cooling after
demolding. Semi-adiabatic temperature of concrete can be measured in an insulated container to determine the approximate setting time and evaluate changes in hydration kinetics. Internal relative humidity measurements were used to study the driving forces for shrinkage stress: Drying and self-desiccation. Drying profiles can be observed to study the drying gradient and its subsequent effect on stress development.

Figure 4-8. Strain sensor mounted in a cylinder mold and shown with data collection system

Figure 4-9. Digital relative humidity sensor and packaging system for use in cementitious materials
4.7 References


5. RESULTS

Previous research has been conducted to investigate concrete behavior at early age using the methods presented. Previous work has demonstrated the validity of the RSTM for determination of retrained stress development and evaluation of creep, as well as application of the experiment for various materials [5-1 to 5-9]. Preliminary results have also shown the need for predictive modeling at early age and the possibility of modifying the current code models for early age behavior. [5-10 to 5-12]

5.1 SCC

The growing use of self-consolidating concrete (SCC) in North America has led to many practical questions about the mechanical performance of this material. SCC is designed to have fresh concrete properties that allow the material to consolidate under its own weight and completely fill formwork without the need for external vibration [5-13]. SCC is highly fluid, but must be sufficiently cohesive to prevent segregation. Much attention is paid to achieving the required flowability and stability, while more research is needed on the resulting impact on hardened properties. Mechanical properties can be influenced by SCC strategies such as using higher paste content with or without mineral additives, lower w/cm ratio, and uniformly graded aggregate with a greater portion of fine particles. The early age mechanical performance was investigated with experiments that measure shrinkage, creep under tensile stress, and strength development. Results indicate that SCC mechanical properties depend on strategy and mixture proportions. As with ordinary concrete, self-consolidating concretes can be produced with a wide range of volume stability and mechanical properties. Implications for early age cracking and long-term durability of different SCC strategies are discussed.

This study was implemented to investigate the impact of SCC strategies on the hardened properties of SCC. The early age shrinkage and creep, strength development, and modulus of elasticity were measured to assess the characteristic behavior of SCC. Results were used to identify acceptable performance limits and model predictions that may need improvement for practical application.
5.1.1 Strength Development

Compressive strength development, shown in Figure 5-1, is similar for most of the materials in this study, with the exception of SCC2. This mixture has a lower w/cm ratio and higher paste content to improve segregation resistance, which also led to higher strength. SCC mixtures in general tend towards low w/cm and high paste content, thus higher strength is typically achieved. The rate of strength gain is also relatively high due to the high paste content. Indirect tensile strength results are shown in Figure 5-2. The tensile strength specimens were kept in a drying environment with the shrinkage and creep specimens in order to closely follow the same curing regime. The tensile strength does not increase very much after three days, which is expected for concrete that is not given an external source of water for curing. Lack of external water during curing may increase the risk for cracking at early age by limiting strength development; however this was not specifically investigated. Indirect tensile strength does not correspond directly to uniaxial tensile strength and may be as much as 5 to 12% higher [5-14]. The presence of a drying stress gradient is likely to cause failure at stress that is apparently lower than measured strength [5-15].

Figure 5-1. Compressive strength development of moist-cured 4x8-in. (100x200 mm) cylinders
5.1.2 Shrinkage and Stress Development

The reduction in pore pressure caused by internal reduction of relative humidity due to hydration (self-desiccation) and reduction of relative humidity due to diffusion and evaporation of moisture (drying) lead to micro-scale compressive stress in the solid cement hydration products. It is this compressive stress in the solid that causes the time dependent viscoelastic response called shrinkage of the cement paste. When concrete is prevented from shrinking by aggregate, structural restraint, geometry, steel reinforcement, or self-weight (as in a curling slab on grade), tensile stress develops in the cement paste. If the tensile stress exceeds 40% to 50% of the tensile strength, microcracks begin to form. As the stress increases further, microcracks will eventually coalesce into macrocracks, which can be detrimental to durability and structural behavior. Because the drying process occurs from the surface, greater stress is generated in the surface layer, causing a stress gradient and a higher probability of microcracking at the surface.

Autogenous shrinkage was measured in concrete prism specimens to assess the potential for stress development and cracking at early age. Results are shown in Figure 5-3 for a period of 30 days, demonstrating that significant autogenous shrinkage develops when the w/cm ratio is reduced to 0.40 or lower. The cement paste content also significantly increases shrinkage, as is
evident in SCC2, which was 40% cement paste by volume. Results for the entire test period were not obtained for SCC4 because of a sensor malfunction, but it would be expected to lie between SCC1 and SCC2 according to $w/cm$ and paste content.

Early age total shrinkage measurements including both autogenous shrinkage and drying shrinkage are shown in Figure 5-4. In the first 30 days after casting, the highest shrinkage occurs in SCC2. Low $w/cm$ and high paste content caused a significant amount of autogenous shrinkage, which is a major contributor to overall early age shrinkage and subsequent stress development and cracking. Total shrinkage in OPC1 and SCC3 is mostly due to drying and is relatively low in comparison.

Mixture SCC2 was studied using different types of SCMs and then compared to 100% portland cement and inert limestone filler, as shown in Figure 5-5. In all cases, replacement of portland cement with other powder types produced lower early age shrinkage. This result indicates that SCMs and inert fillers should be strongly considered in all SCC as a strategy to obtain higher powder content for stability and fluidity in the fresh state.

Shrinkage measurements were carried out for a period of up to 3.5 years using standard ASTM C 157 prisms. The results are shown in Figure 5-6 through Figure 5-8 on a logarithmic scale. The results show that cementitious paste content is the dominant factor in long term shrinkage. Despite having low $w/cm$ ratio and supplementary cementitious materials in the form of fly ash, generally considered to reduce long term shrinkage, SCC2 has almost double the long term shrinkage as the most other mixtures. Other mixtures have total shrinkage that is proportional to the amount of cement paste. Shrinkage versus mass loss measurements show that a change in the shrinkage mechanism is taking place after approximately two years. The change proportion of mass loss to shrinkage is consistent with other reported changes due to the breaking of the capillary menisci and the loss in surface tension in the pore fluid, which releases stress. Some samples show a slight expansion thereafter, which is an indication that the capillary stress has relaxed substantially.

Stress development under fully restrained conditions is shown in Figure 5-9. The results indicate that to reduce the risk of early age cracking in SCC, using a $w/cm$ ratio of at least 0.40 to 0.42
can prevent autogenous shrinkage from causing significant stress, while at the same time minimizing drying shrinkage. A continuous moist curing strategy for 7 days is thought to prevent the stress from occurring at too early age when tensile strength is still low, which may reduce the risk for cracking. Stress-strength ratios shown in Figure 5-10 demonstrate that microcracking and damage may be caused as early as one or two days after drying at early age.

Figure 5-3. Autogenous shrinkage of 3x3x11.25-in. (75x75x250mm) prisms sealed at 23°C

Figure 5-4. Early age total (drying plus autogenous) shrinkage of 3x3x24.5-in. (75x75x600mm)
RSTM prisms during drying at 50%RH, 23°C

Figure 5-5. Shrinkage of SCC with different mineral filler types

Figure 5-6. Long-term shrinkage measurements of SCC mixtures from ASTM C 157 tests (3x3x11.25")
Figure 5-7. Long-term mass loss measurements of SCC mixtures from ASTM C 157 tests (3x3x11.25"")

Figure 5-8. Long-term shrinkage versus mass loss measurements of SCC mixtures from ASTM C 157 tests (3x3x11.25"")
Figure 5-9. Shrinkage stress development of 3x3x24.5-in. (75x75x600mm) restrained prisms during drying at 50% RH, 23°C

Figure 5-10. Stress-strength ratio of 3x3x24.5-in. (75x75x600mm) restrained prisms during drying at 50% RH, 23°C

5.1.3 Creep and Stress Relaxation

Shrinkage and creep of concrete are very closely related behaviors. Both are time dependent viscoelastic deformations, and the difference lies primarily in how stress is generated. Shrinkage can be defined as the viscoelastic deformation of concrete due to the generation of internal
micro-scale compressive stress in the solid cement hydration product as a result of pore pressure reduction. This is actually one form of creep. However, the term creep is often reserved for the time dependent viscoelastic response to stress generated from externally applied loads. The uniaxial experiment used in this study was designed to separate the mechanisms of creep and shrinkage using superposition. The tensile creep strain is calculated according to

$$\varepsilon_{cr} = \varepsilon_r - \varepsilon_{sh}, \quad (5.1)$$

where $\varepsilon_{cr}$ is the total creep strain, $\varepsilon_r$ is the cumulative restrained deformation and $\varepsilon_{sh}$ is unrestrained shrinkage. Creep compliance, $J_c$, is also referred to as specific creep or creep per unit stress, and can then be calculated from the experimental measurements according to

$$J_c(t,t') = \frac{\varepsilon_{cr}}{\sigma}, \quad (5.2)$$

where $\sigma$ is the average applied stress in the specimen, calculated as the applied load divided by the specimen area. The limitations of this superposition analysis are that it assumes a linear relationship between stress and creep and it assumes an even distribution of stress throughout the cross-section. The linearity of creep has been established for stress levels up to 40% of the ultimate strength for mature concrete in compression [5-16]. However, this is often not the case in early age concrete subjected to tensile stress from restrained shrinkage. Indeed, the stress-strength ratios in this study exceeded 40% on the first day of drying. The tensile creep reported here may include microcracking, which contributes to nonlinear behavior. Non-uniformity of stress in the cross-section is caused by the drying shrinkage gradient. The gradient tends to be most severe during the first few days of the drying period. Generally microcracking is thought to occur only at high levels of applied stress, but it has been shown that due to the presence of a drying gradient, microcracking can occur at the surface even in unrestrained shrinkage specimens [5-17, 5-18, 5-19]. Figure 5-11 shows the specific creep response of four of the mixtures in this study. A correlation in the results is evident between creep and the rate of stress development. The creep capacity is directly proportional to paste content and w/cm ratio. The high stress-strength ratio of SCC1 during drying probably induced microcracking damage, which increases the apparent calculated creep. Stress relaxation occurs due to microcracking as well,
but the risk for cracking may still exist due to the damage induced by shrinkage stress. However, in the case of SCC4 the stress develops rapidly due to the lack of relaxation by creep; as a result the risk of cracking is severe. The calculated creep only indicates the ability to reduce cracking if it does not include microcracking damage. The capability for relaxation by creep is indicated by the creep-shrinkage or relaxation ratio, shown in Figure 5-12. This ratio can be used to obtain order of magnitude level estimates and as a rough approximation for creep in structural models in the absence of more accurate data.

![Figure 5-11. Specific creep of 3x3x24.5-in. (75x75x600mm) restrained prisms during drying at 50% RH, 23°C](image-url)
Figure 5-12. Creep-shrinkage ratio for SCC mixtures

5.1.4 Summary

An investigation of the early age mechanical behavior of SCC material has revealed a potentially high risk for cracking. Analysis of a database of SCC mixture proportions indicates a trend to use high cement paste content and a relatively low w/cm ratio. As a result, autogenous shrinkage may cause significant stress at early age and the creep capacity may be diminished in low w/cm ratio materials. These factors contribute to increasing the cracking risk at early age. On the other hand, the tendency for SCC to have higher early age strength gain and ultimate strength decreases the cracking risk. SCC mixtures are often developed with certain flow characteristics in mind, but caution should be used so that cracking is avoided. A strategy that minimizes cement paste content to achieve the necessary flow characteristics is beneficial. Using a w/cm ratio that avoids significant autogenous shrinkage will reduce the potential for shrinkage stress development. Providing external water during curing in field applications will also delay shrinkage stress development at early age until tensile strength has matured. In general, it is best not to treat SCC as a group of materials with comparable mechanical behavior. Different strategies for mixture proportioning may lead to SCC materials that have the common ability to flow into formwork without mechanical vibration, but have very different behavior when considering mechanical performance and early age cracking risk.
5.2 HPC

The results presented in the following sections are organized in several ways. The experiments were performed on nine different concrete mixtures. The mixtures were previously presented and the mixture identification code designations defined in Chapter 3. Five mixtures containing set retarding admixture were tested and are grouped together for comparison purposes. The remaining mixtures did not contain set retarder. The \( w/cm \) ratio was varied for the Lake Springfield and Duncan Rd. materials, and these comparisons are presented separately. Finally, the mixtures containing laboratory materials are compared directly with mixtures containing field materials to study the effect of material source.

5.2.1 Unrestrained Shrinkage

Unrestrained shrinkage is often described as a potential driving force for cracking in concrete. The unrestrained shrinkage results for IDOT concrete mixtures are shown in Figure 5-13. The HPC mixtures containing laboratory materials performed similarly and had slightly higher shrinkage than the conventional mixture after one week of drying. The \( w/cm \) ratio was 0.44 for all mixtures in this group and the paste content was similar, therefore the increase in shrinkage can possibly be attributed to the presence of mineral admixtures (fly ash, silica fume, HRM). Pozzolanic mineral admixtures cause an increase in autogenous shrinkage that is most pronounced at early age [5-20]. The pozzolanic reaction consumes additional water as the silica reacts with the hydration product calcium hydroxide to form secondary C-S-H. Mixtures containing silica fume (IHPC2, IHPC4) had slightly higher shrinkage than IHPC1, which contained HRM. Silica fume has the greatest impact on autogenous shrinkage due to its high silica content, which chemically uses a greater amount of water, and due to a smaller particle size, which speeds up the hydration process and the rate of autogenous shrinkage.

Unrestrained shrinkage of the remaining IDOT concrete mixtures from years two and three with laboratory materials are shown in Figure 5-14 with a consistent \( w/cm \) ratio of 0.44. These mixtures had similar free shrinkage behavior despite changes in cementitious materials, aggregate composition, and paste percentage. The free shrinkage after one week of drying correlated with the portland cement content for these mixtures. All mixtures in this group contained a pozzolan, so this relationship may be related to autogenous shrinkage. Overall
magnitudes after one week of drying are similar for all IDOT concrete mixtures at the same $w/cm$ ratio. In Figure 5-16, the $w/cm$ ratio was varied for the Duncan Rd and Lake Springfield concrete mixtures. The change from 0.44 to a lower $w/cm$ ratio, 0.39 in the case of Lake Springfield or 0.41 in the case of Duncan Rd, did not produce much change in the free shrinkage result. The reduced drying shrinkage from a lower $w/cm$ ratio is probably counteracted by the increase in autogenous shrinkage. Therefore, it may be stated that the balance between autogenous shrinkage and drying shrinkage is optimized in this range. Increasing the $w/cm$ ratio from 0.44 to 0.50 produced an increase in shrinkage, as expected.

The mixtures containing field materials consistently exhibited lower shrinkage than mixtures containing laboratory materials. The comparisons can be observed in Figure 5-13 and Figure 5-15. It is reasonable to attribute this difference to the source of cementitious materials, since the aggregates were similar (crushed limestone). The role of aggregate as a restraint to volumetric changes indicates that if they are similar materials, they should not change shrinkage behavior significantly. The cement and fly ash combination was chemically different for the laboratory and field sources and the subsequent hydration products that form had different shrinkage potentials. The chemical compositions determined by XRF are given in Table A-1, Table A-2, and Table A-3 of Appendix A for the cement, fly ash, and other supplementary cementitious materials respectively.

Long-term shrinkage measurements of 3-in. ASTM C 157 standard prism specimens are presented in Figure 5-17 through Figure 5-20. Most of the specimens were measured up to 40 days and many were measured for longer periods up to a maximum of almost three years. Trends that were apparent in the early age (8 day) measurements are not always consistent when considering the longer-term (40 day) measurements. The dominant driving force of shrinkage changes over time as hydration reduces porosity and autogenous shrinkage ceases to play a role in the overall shrinkage behavior. The ASTM C 157 prism measurements also do not always have the same magnitude of free shrinkage as the RSTM uniaxial specimens, despite the drying surface areas (surface to volume ratios) being identical. Samples were always drying on two sides only to maintain symmetry. The differences may be attributed to the somewhat higher experimental error in the prism tests relative to the more sensitive uniaxial measurement, different time of first measurement, or location in the environmental chamber. The concrete
mixtures made with field materials have higher overall shrinkage in the long-term measurements when compared to lab materials. The difference is attributed to the cement and fly ash sources. As the microstructure develops, the shrinkage may become more dependent on the pore structure and degree of hydration. The pore size dependence can be closely linked to \( w/cm \) ratio and the presence of pozzolans. The conventional concrete mixture, which did not contain pozzolans, has higher shrinkage than the HPC mixtures, demonstrating that pozzolans have a more substantial benefit for decreasing long term shrinkage than for early age.
Figure 5-13. Free shrinkage of uniaxial specimens, \( w/cm = 0.44 \)

Figure 5-14. Free shrinkage of uniaxial specimens, \( w/cm = 0.44 \), laboratory materials
Figure 5-15. Free shrinkage of uniaxial specimens, laboratory vs. field materials

Figure 5-16. Free shrinkage of uniaxial specimens, variations in w/cm
Figure 5-17. Long-term free shrinkage of prisms, $w/cm = 0.44$

Figure 5-18. Long-term free shrinkage of prisms, $w/cm = 0.44$, laboratory materials
Figure 5-19. Long-term free shrinkage of prisms, variation in $w/cm$

Figure 5-20. Long-term free shrinkage of prisms, laboratory vs. field materials
5.2.2 Weight Loss and Drying Rate

The initial amount of water in a concrete mixture influences how much shrinkage potential exists for that material. Simple weight loss measurements can be performed to measure the amount of water that evaporates from a concrete specimen. Both the rate of weight loss and the amount of weight loss per unit of shrinkage relate to the shrinkage potential of the material. Pozzolanic mineral admixtures generally increase the amount of autogenous shrinkage, which may cause an increase in the amount of shrinkage per unit of weight loss. The rate of weight loss results are shown in Figure 5-21 and Figure 5-22 and weight loss per unit of shrinkage is shown in Figure 5-23 and Figure 5-24. Strong trends are not evident from the measured data, despite the presence of mineral admixtures. The conventional concrete mixture has significantly less weight loss and more shrinkage per unit weight loss than the HPC mixtures. This may indicate that this mixture has more remaining shrinkage potential and could have significantly higher ultimate shrinkage. It was observed to have the steepest shrinkage slope in Figure 5-17, which supports this theory. Mixtures containing SCMs would be expected to have denser pore structure (less porosity) at later ages, but not within the first few weeks, as the pozzolanic reaction occurs more slowly than cement hydration. Porosity would be slowly densified versus plain cement-only concrete after a period of 2-3 weeks. Therefore, the later age shrinkage potential for an OPC mixture would be greater than for mixtures containing SCMs.
Figure 5-21. Weight loss of concrete prisms, $w/cm = 0.44$

Figure 5-22. Weight loss of concrete prisms, laboratory vs. field materials
Figure 5-23. Free shrinkage vs. weight loss, $w/cm = 0.44$

Figure 5-24. Free shrinkage vs. weight loss, laboratory vs. field materials
5.2.3 Tensile Strength Development

The split tensile strengths of 4x8-in. cylinders were measured during the first week after casting. The specimens were sealed for one day, and then cured at 50% RH and 23°C for the remainder of the week. Tensile strength was measured as opposed to compressive strength because of the relationship of tensile stress to shrinkage cracking in concrete. It may be assumed that the tensile strength is 10% of compressive strength for estimation purposes. Tensile strength results are shown in Figure 5-25. This group of concrete mixtures contains a set retarding admixture, which has reduced the early age tensile strength in some instances. The HPC mixtures do not have higher early age tensile strength than the IDOT conventional mix, ISTD. The presence of fly ash can be expected to reduce the early strength gain. Higher strengths are expected from the HPC mixtures at later ages. The early age strengths were relatively unaffected by the proportioning of mineral admixtures. The mixtures containing field materials had higher initial strengths than the corresponding mixtures with laboratory materials indicating the retarder may have interacted differently with the cement. Retarder effectiveness reportedly depends on the C₃A content of the cement. When the C₃A content of cement is high, more retarder is consumed by the C₃A and becomes unavailable to slow C₃S hydration [5-14]. As shown in Appendix A, the C₃A contents are similar for the laboratory and field cements, with the laboratory cement slightly higher, which does not support the argument. Sulfate content also may alter the effectiveness of retarder by changing C₃A hydration [5-14]. The presence of mineral admixtures further complicates this relationship. The importance of testing cement-admixture interactions is illustrated by these results. Chemical analysis may not be able to predict an interaction problem, but the strength testing of any combination of admixtures will reveal the potential for strength development.

Figure 5-26 shows the tensile strength measurements for the remaining mixtures with laboratory materials and a 0.44 w/cm ratio. The materials have similar strengths except IDL44R1 (Duncan Rd), which had lower strength due to the addition of retarding admixture. The decision to stop using retarder in the laboratory was made after this mixture was tested. The results of these tests showed that the use of retarding admixture in the laboratory was unwarranted and caused difficulty in determining early age behavior. Laboratory specimens were not wet cured, as was the case in actual bridge deck applications. Stress developed in the laboratory specimens much earlier and thus the need for higher early strengths. Low strengths often meant early failure (less than 2 days) in the uniaxial test, prohibiting the measurement of early age creep.
Figure 5-27 shows the strength comparisons of laboratory and field materials. The field materials did not have a consistent relationship to the laboratory materials, indicating a probable dependence on cement composition.

Figure 5-28 shows the strength comparisons of different $w/cm$ ratio mixtures from Lake Springfield and Duncan Rd. Lowering the $w/cm$ ratio produced the expected result of increasing strength in both mixtures. Although additional tensile strength is beneficial to resist cracking, the strength may not reflect the cracking potential of the material. Strength is related to creep and the reduction in creep that usually occurs with an increase in strength may counteract a resistance to cracking. A strong concrete material that also has high creep capacity is ideal to resist cracking due to shrinkage stress.

Early age tensile strength plays a critical role in the long-term durability of concrete structures. Shrinkage stresses that develop at early age will cause cracking much sooner if sufficient strength has not been achieved. High strength usually means low creep, which is detrimental to crack resistance. The importance of wet curing is to delay the start of drying shrinkage while allowing tensile strength to develop, which further reduces the risk of cracking. Concrete will continue to gain strength over time, so if shrinkage stress development is prevented during the very early age when the concrete is one or two days old, the risk for cracking is greatly diminished.
Figure 5-25. Split tensile strength, $w/cm = 0.44$

Figure 5-26. Split tensile strength, $w/cm = 0.44$, laboratory materials only
Figure 5-27. Split tensile strength, laboratory vs. field materials

Figure 5-28. Split tensile strength, variations in $w/cm$ ratio
5.2.4 Thermal Stress and Hydration Kinetics

Supplemental laboratory tests were performed to monitor the internal temperature of concrete. The internal temperature of a 3x3x11-in. prism, cast with each concrete batch, was measured by embedding polymer tubes in the top of the fresh concrete, and then placing temperature probes into the tubes. Five probes were placed in each temperature prism and it was sealed until an age of 23 hours, at which time it was demolded. Throughout the test, the temperature prisms were maintained in a climate controlled environment.

Once the internal relative humidity (RH) measurement system became available, the system was utilized to measure the internal temperature as well. The probe system was no longer used. The only differences between the two temperature measurement systems are that the old probe system involved placing straws in the top of a concrete prism with probes inside whereas the new system involved sensors embedded in ¼ diameter PVC tubes projecting into the prism from the sides.

The main purpose of this experiment was to monitor the internal temperature of the concrete just after the forms were removed. Removal of the forms from the moist concrete induced evaporative cooling. This evaporative cooling most likely causes thermal contraction. Since the uniaxial test involves monitoring shrinkage until a certain strain threshold is reached (8 με), this thermal contraction could induce stress. The purpose of the uniaxial test is to measure drying shrinkage, creep, and stress rather than any displacements caused by thermal events. Therefore, the start of the uniaxial data-acquisition and application of restraining forces was delayed one hour after demolding to ensure that most of the thermal contraction from evaporative cooling had occurred before the test began. Figure 5-29 shows the internal temperature data, illustrating the evaporative cooling that occurred after demolding each specimen. Figure 5-29 also indicates most of the cooling was complete within one hour of form removal.

In addition to measuring the internal temperature in the prism, the internal temperature was monitored using a small sample of 0.44 lbs. (200 g) of concrete enclosed in a thermos, which created semi-adiabatic conditions. Some typical semi-adiabatic internal temperature curves are shown in Figure 5-30. This was a supplemental test used mainly for quality control purposes.
Figure 5-29. Internal temperature of concrete prisms during demolding

Figure 5-30. Internal concrete temperature under semi-adiabatic conditions
5.2.5 Restrained Stress Development

Stress development due to shrinkage is shown for IDOT concrete mixtures in Figure 5-31 through Figure 5-34. All mixtures of like w/cm and material source developed similar magnitudes of stress (within a range of 100 psi). No significant trends were observed, leading to the conclusion that changing mixture proportions and mineral admixtures within the range of this study did not have much effect on stress development. It should also be noted that stress development did not follow the same trends as free shrinkage, emphasizing the importance of creep when considering stress. The stress development followed the tensile strength development trends in many cases. Creep of concrete is known to closely relate to strength, so this relationship was expected. In Figure 5-33, a trend is visible where higher stress develops in low w/cm ratio mixtures. This relationship is also related to strength.

Failure due to fracture of the uniaxial concrete specimen is indicated in each figure with a black “X.” Average shrinkage stress levels were approximately 80% of the concrete tensile strength at the time of failure, which is similar to typical values for compressive creep after long term sustained load application [5-26]. Previous work by Altoubat and others has shown that specimens in direct tension typically fail below their measured tensile strength. In his work, the rate of tensile strength gain was documented by split tensile strength measurements. Part of the difference may be attributed to this method for tensile strength measurement. The split cylinder test is an indirect measurement of tensile strength that is dependent on test conditions and may be as much as 5 to 12% higher than direct tensile strength, due to tri-axial stress confinement in the loading region [5-14]. Another factor in the stress at failure is the presence of a drying stress gradient in the specimen. As drying occurs, stress develops due to capillary tension in the pore microstructure. The stress, which is assumed constant in this study, is actually much greater at the surface where drying has occurred. The surface stress results in microcracking and causes damage to the specimen, which results in failure when the cracks propagate. The proper analysis for failure of concrete in direct tension should probably be based on a fracture energy approach. Comparison of the stress with the tensile strength at the macro level does not account for micro-mechanical processes that govern failure and the quasi-brittle nature of concrete [5-21].
Figure 5-31. Shrinkage stress development, $w/cm = 0.44$

Figure 5-32. Shrinkage stress development, $w/cm = 0.44$, laboratory materials only
Figure 5-33. Shrinkage stress development, variation in w/cm ratio

Figure 5-34. Shrinkage stress development, laboratory vs. field materials
5.2.6 Evolution of Elastic Modulus at Early Age

The elastic modulus was measured for concrete mixtures tested with the restrained test method each time the applied load was incremented. Stress-strain diagrams taken from test data of each uniaxial test are shown in Figure 5-35. The evolution of the elastic modulus for all IDOT concrete mixtures is shown in Figure 5-36. Early strains are relatively high, which is similar to other reports of concrete modulus measurements at small strain or under dynamic loading. Each loading increment results in a finite amount of elastic strain as the specimen returns to original length from the threshold strain of 8 \( \mu \)strain. The elastic modulus was calculated as the summation of accumulated stress divided by the summation of elastic strain according to

\[
E = \frac{\sum \sigma}{\sum \varepsilon_{el}} = \frac{\sum P}{\sum \varepsilon_{el} A},
\]

where \( \sigma \) is the total average stress in the concrete, \( \varepsilon_{el} \) is the summation of measured elastic strain, \( P \) is the summation of load applied at each increment, and \( A \) is the cross sectional area, which was equal to 9 in\(^2\) for all specimens. The load increment varied depending on the shrinkage rate and the time necessary to reach the threshold for restrained load application. The elastic modulus measurement decreases after the initial measurement for most mixtures. The initial measurements represent the initial tangent modulus of elasticity, which is expected to be higher than the subsequent tangent or secant modulus. The measurements either level out or begin to increase slightly over the test period of one week. The ACI equation was used to predict the elastic modulus, also shown in Figure 5-36, according to

\[
E = 57,000 \sqrt{f_c \left( \frac{t}{4 + 0.85t} \right)}, \text{ psi}
\]

where \( f_c \) is the compressive strength in psi and \( t \) is the age of concrete in days [5-27]. The measured values for elastic modulus correlate well with the magnitude or evolution of the ACI equation. The initial measurement of the elastic modulus from the first load application during the test, which is at relatively low stress and strain, corresponds to the initial tangent modulus, \( E_1 \) shown in Figure 5-37. At higher stress levels, the measurement corresponds to the chord
modulus, shown as $E_2$ in Figure 5-37 since it represents a linear slope of the current stress-strain measurements. Slope $E_3$ is a chord modulus at higher stress-strain levels, where the concrete exhibits highly nonlinear behavior. The initial tangent modulus is expected to be greater than the chord modulus. At higher levels of stress, the chord modulus decreases due to the non-linear behavior of concrete. In this testing program, stress levels frequently exceeded 40% of the measured tensile strength, which is cited by many researchers as the point at which nonlinear behavior typically begins [5-14]. Some elastic modulus measurements showed a gradual increase in stiffness after an initial drop. This behavior is specific to early age concrete in which the microstructure is continuously developing stiffness and strength, causing the stress-strain curve to increase over time. To illustrate this point, two hypothetical cases of stress-strain curves for early age concrete are presented. In case I, shown in Figure 5-38, the measured stiffness decreases with time, despite the increase in the stress strain curve with evolving microstructure. In case II, shown in Figure 5-39, the opposite effect is measured. By changing the rate of stiffness evolution over time, the elastic modulus appears to increase despite the nonlinear behavior at later measurements. Therefore, measurements depend on the rate of stiffness evolution, stress level at measurement, and the shape of the stress strain curve.
Figure 5-35. Stress strain diagram for uniaxial tests, all materials

Figure 5-36. Elastic modulus (secant) for all materials
Figure 5-37. Schematic diagram for measurement of elastic modulus

Figure 5-38. Measurement of elastic modulus at early ages, case 1
5.2.7 Early Age Tensile Creep

Creep of concrete has been studied extensively for mature concrete under compressive loading. The importance of creep in concrete was first established from structural concerns about long-term deformation and prestress loss. Creep coefficient and specific creep are parameters that usually describe creep under sustained loading. They were originally single values that were used in structural design to account for long-term deformation. The typical range for long-term creep coefficient of mature concrete in compression is 1.5 to 3.0 and specific creep (creep compliance) ranges from 0.1 to 0.4 με/psi (25 to 50 με/MPa) [5-14].

Recent research focuses on tensile creep of concrete, since it relates to the ability to resist cracking by relaxation of stress. Early age concrete in particular is sensitive to volumetric changes such as drying and autogenous shrinkage or thermal deformation. Stress relaxation is critical at early ages, since the material has not achieved full strength and is more susceptible to cracking. At early ages, the creep coefficient changes over time and is best represented as a curve. The shape of the curve depends on the evolution of microstructure and environmental conditions. Furthermore, the creep coefficient is not the same in tension as it is in compression. The initial rate of creep is higher in tension, which results in greater creep for relatively short
durations of load [5-14]. At later ages, compressive creep may exceed tensile creep, as the creep function appears to stabilize sooner in tension.

5.2.8 Discussion of Creep Parameters

Strain measurements from the unrestrained specimen were compared to the measured deformation of either the restrained or constant load specimen to obtain creep deformation. The procedure for calculating creep from raw test data is illustrated in Figure 5-40 for both the restrained and constant load tests. A chart of typical deformation data from a restrained test is displayed in Figure 4-4. The difference in deformation between the unrestrained and the loaded specimen is attributed to creep. The total tensile creep strain was calculated as the difference between the accumulated restrained deformation and the free shrinkage according to

$$\varepsilon_{cr} = \varepsilon_r - \varepsilon_f,$$

(5.5)

where, $\varepsilon_{cr}$ is total creep strain, $\varepsilon_r$ is restrained deformation and $\varepsilon_f$ is unrestrained deformation.

Creep experiments are often performed using a constant load rather than simulating restraint. A constant load relates to structural conditions where external loads are applied for long periods. It is simpler to perform a constant load test, but a constant load does not simulate the creep that occurs due to restrained shrinkage stresses. The creep is different between the tests because of the aging and strength development of the material. Creep strain for the constant load test is initially higher than the restrained test and then levels off since there is no accumulation of stress. Creep strain from a restrained test will eventually surpass the constant load test as it reaches higher stress. Figure 5-41 shows the typical difference in creep strain measurements for a constant stress test and a restrained stress test. Creep at early age is highly sensitive to loading age, so as load is applied at later age during the restrained test, the additional creep due to increasing restrained load is proportionally lower. Therefore, instead of evaluating creep behavior according to strain, it is common to normalize the creep strain, $\varepsilon_{cr}$, by the elastic strain, $\varepsilon_{el}$, at the time of loading according to
The creep coefficient, $\phi$, applies to constant load creep tests where the elastic strain is measured during initial load application. The calculation has been modified for restrained creep testing [5-1, 5-22] where creep strain under restrained conditions is normalized with the measured elastic strain at each load compensation cycle. At early age, the creep coefficient evolves over time and reflects the developing microstructure stiffness, which represents the ability of concrete to relax stresses. Early age tensile creep coefficient values in the literature are limited, but are typically in the range of 0.3 to 0.5 initially and then increase to 0.5 to 1.5 after the first week. These values are comparable to compressive creep after 100 days of loading [5-14].

Specific creep is a similar parameter to creep coefficient. The creep strain, $\varepsilon_{cr}$, is divided by applied stress, $\sigma$, instead of elastic strain according to

$$c = \frac{\varepsilon_{cr}}{\sigma}. \quad (5.7)$$

When comparing concrete mixtures of the same stiffness, the creep coefficient and specific creep will give similar results. However, when comparing materials of different strength and stiffness, specific creep is more appropriate, as it does not include difference in elastic deformation.

5.2.8.1 Restrained Tensile Creep Strain

The calculated creep strain for IDOT materials is shown in Figure 5-42. Trends related to material composition, $w/cm$ ratio or material source are not apparent when comparing the mixtures. Creep is dependent on the stress level, so creep strain is not an effective measure of creep behavior between tests at different stress levels. Specific creep and creep coefficient are more appropriate parameters than strain for comparison of creep capacity.

The creep strain can be related to free shrinkage strain as the Creep-Shrinkage ratio. This value can be used as a measure of how much of the developing shrinkage stress is relaxed by creep. This is a useful estimate when calculating the impact of creep and shrinkage on concrete
structures. The creep-shrinkage ratio for IDOT concrete mixtures, shown in Figure 5-43, converges to about 0.5 after 1 week for most mixtures. Previous research has shown that the value is not very sensitive to material proportions or w/cm ratio, but it may be affected by sealed or wet curing [5-1].

5.2.8.2 Restrained Tensile Creep Coefficient and Specific Creep

The tensile creep coefficients for IDOT HPC mixtures are shown in Figure 5-44. The results do not indicate a trend relating creep coefficient to either mixture proportions or pozzolans, but the field material mixtures had lower creep coefficient than mixtures containing laboratory materials. The relationship is similar to the differences in strength. The specific creep for IDOT HPC mixtures is shown in Figure 5-48. The specific creep was similar for all mixtures when compared relative to the applied stress level.

Figure 5-45 shows the creep coefficient for remaining 0.44 w/cm mixtures with laboratory materials, all of which had similar strength. The creep coefficient is similar for these materials, indicating little dependence on mixture proportioning of aggregates and cementitious material, within the range of this study. The mixture with the highest creep coefficient in both cases also has the lowest strength. The creep coefficient seems to be unaffected by changes in mixture proportioning or cementitious materials, as long as the strength of the material does not change. Specific creep, shown in Figure 5-49 was similar for materials containing silica fume and higher for the Duncan Rd mixture IDL44R1, which did not contain silica fume, only fly ash. In Figure 5-46, the creep coefficient does not correlate with changes in w/cm ratio for Duncan Rd and Lake Springfield mixtures. This may again be due to differences in strength between the materials, changing the amount of elastic strain accumulated in each load cycle. The specific creep, shown in Figure 5-50, shows that as w/cm ratio decreases, the amount of creep decreases, as expected.

The creep coefficients for laboratory and field material sources are shown in Figure 5-47, and the specific creep is shown in Figure 5-51. Field material mixtures had lower creep than the laboratory mixtures, indicating that creep is dependent on cementitious material source. The relationship of creep to cement composition is similar to shrinkage. It is generally believed that increasing C₃A content or decreasing C₃S content produces higher creep due to changes in the
hydration product structure [5-14]. The combination of cement and pozzolans from different sources makes the prediction of shrinkage and creep potentials difficult in this study, but it is evident that the combination used in the UIUC laboratory has high shrinkage and creep relative to cement/pozzolan combinations tested from other IDOT sources.

The practice of wet curing for concrete under field conditions has important implications for creep behavior. Wet curing may help to reduce cracking by reducing shrinkage, suppressing early shrinkage stress development, and increasing tensile strength, but tensile creep is reduced. Creep is lower for mature concrete, particularly when compared to very early age concrete that is one day old. Tensile creep decreases over time once drying starts, as moisture is lost from the pore system. Østergaard performed basic creep tests concrete at different ages and the results showed that significantly higher creep is measured when load is applied at one day rather than three days [5-22]. A reduction in creep capacity will reduce the ability of concrete to relax tensile drying shrinkage stresses and resist cracking.

5.2.9 Conclusions of HPC Tests

The early age stress and creep and shrinkage interaction of IDOT concrete mixtures was investigated using a uniaxial creep and shrinkage measurement system developed at UIUC. A summary of results from this portion of the study is given in Table 5-1. The IDOT HPC mixtures of like $w/cm$ containing laboratory materials performed similarly to each other and had slightly higher shrinkage than the conventional mixture after one week of drying. The increase in shrinkage was attributed to greater autogenous shrinkage from lower $w/cm$ and/or silica fume that was most pronounced at early age. Lowering $w/cm$ ratio did not produce much change in free shrinkage. Additional benefit of reduced drying shrinkage from low $w/cm$ ratio was counteracted by an increase in autogenous shrinkage. The balance between autogenous shrinkage and drying shrinkage was optimized in this range of $w/cm$. Increasing the $w/cm$ ratio produced an increase in shrinkage, as expected. The mixtures containing field materials consistently had less shrinkage than mixtures containing laboratory materials, at early ages, but this trend did not continue to later ages, beyond 40 days. These differences were attributed to the source of cementitious materials. Trends that were apparent in the early age (8 day) measurements are not always consistent with longer-term (40 day) measurements. The
mechanisms of shrinkage change over time as the pore structure densifies and autogenous shrinkage ceases to play a role in the overall shrinkage behavior. Microstructures that may develop slowly in the first week tend to catch up later in the hydration process as the mechanisms of hydration change. A significant trend was not evident from the shrinkage vs. weight loss measurements, possibly due to self-desiccation that occurred before the first measurement.

The HPC mixtures did not have significantly higher early age tensile strength than the IDOT conventional mix, ISTD. The mixtures containing field materials sometimes had higher initial strengths than the corresponding mixtures with laboratory materials indicating the retarder may have interacted differently with the various cement sources. Lowering the \( w/cm \) ratio produced the expected result of increasing strength. Although additional tensile strength is beneficial to resist cracking, the strength does not indicate the cracking potential of the material. Strength is related to creep and the reduction in creep that occurs with an increase in strength may counteract a resistance to cracking. A strong concrete material that also has high creep capacity is ideal to resist cracking due to shrinkage stress.

Early age tensile strength plays a critical role in the long-term durability of concrete structures. Shrinkage stresses that develop at early age will cause cracking much sooner if sufficient strength has not been achieved. High strength usually means low creep, which is detrimental to crack resistance. The importance of wet curing is to delay the start of drying shrinkage and allow tensile strength to develop, which reduces the risk of cracking. Concrete will continue to gain strength over time, so if shrinkage stress development is prevented during the very early age when the concrete is one or two days old, the risk for cracking is greatly diminished.

Thermal characteristics of each concrete mixture were measured and the results indicate that HPC designs developed by IDOT do not have higher heat evolution or faster setting time when compared to standard IDOT concrete mixtures. The HPC mixtures in this study are unlikely to display an increase in cracking potential from early age thermal stresses.
All mixtures developed similar magnitudes of stress when compared at the same water cement ratio and with the same materials source, leading to the conclusion that changing mixture proportions and mineral admixtures within in the range of this study did not have much effect on stress development. Stress development did not follow the same trends as free shrinkage, emphasizing the importance of creep when considering stress. Average shrinkage stress levels were on average approximately 80% of the concrete tensile strength at the time of failure. The difference may be attributed to the split tensile method for tensile strength measurement or the presence of a drying stress gradient in the specimen. The stiffness of each concrete mixture, indicated by the elastic modulus, did not follow any observed trends.

The creep-shrinkage ratio for IDOT concrete mixtures converges to about 0.5 after 1 week for most mixtures. The results do not indicate a trend relating creep coefficient to either mixture proportions or pozzolans, but the field material mixtures had lower creep coefficient than mixtures containing laboratory materials. The relationship is similar to the differences in strength. The specific creep was similar for mixtures of the same w/cm ratio and material source when compared relative to the applied stress level. The creep coefficient was similar for constant w/cm and materials source, indicating little dependence on mixture proportions, within the range investigated in this study. Field material mixtures had lower creep than the laboratory mixtures, indicating that creep is dependent on cementitious material source. The combination of cement and pozzolans from different sources makes the prediction of shrinkage and creep potentials difficult in this study, but it is evident that the combination used in the UIUC laboratory has high shrinkage and creep relative to cement/pozzolan combinations tested from other IDOT sources. Results also demonstrated that pozzolans have a more substantial benefit for decreasing long term shrinkage than for early age.

The practice of wet curing for concrete under field conditions has important implications for creep behavior. Wet curing may help to reduce cracking by reducing shrinkage, suppressing early shrinkage stress development, and increasing tensile strength, but tensile creep was reduced. Creep is lower for mature concrete, particularly when compared to very early age concrete that is one day old. Tensile creep decreases over time once drying starts, as moisture is lost from the pore system. A reduction in creep capacity will reduce the ability of concrete to relax tensile drying shrinkage stresses and resist cracking.
Figure 5-40. Schematic for determination of creep for restrained (a) and constant load test (b)

Figure 5-41. Creep strain measurements for constant and restrained tests
Figure 5-42. Tensile creep strain due to restrained drying shrinkage stress, all materials

Figure 5-43. Tensile creep-shrinkage ratio at early age, all materials
Figure 5-44. Tensile creep coefficient evolution, $w/cm = 0.44$

Figure 5-45. Tensile creep coefficient evolution, $w/cm = 0.44$, laboratory materials only
Figure 5-46. Tensile creep coefficient evolution, variation in w/cm

Figure 5-47. Tensile creep coefficient evolution, laboratory vs. field materials
Figure 5-48. Tensile specific creep, $w/cm = 0.44$

Figure 5-49. Tensile specific creep, $w/cm = 0.44$, laboratory materials only
Figure 5-50. Tensile specific creep, variations in w/cm

Figure 5-51. Tensile specific creep, laboratory vs. field materials
Table 5-1. Uniaxial Test Results

<table>
<thead>
<tr>
<th>Designation</th>
<th>w/c</th>
<th>Cement Content</th>
<th>Cementitous Content</th>
<th>Failure Age</th>
<th>Maximum Stress</th>
<th>Tensile Strength at Failure</th>
<th>Tensile Strength (8-day)</th>
<th>Stress-Strength Ratio</th>
<th>Creep Coefficient</th>
<th>Specific Creep</th>
<th>Free Shrinkage after 1 week of drying</th>
<th>Creep-Shrinkage Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>HPC1</td>
<td>0.44</td>
<td>465</td>
<td>612</td>
<td>5.2</td>
<td>208</td>
<td>250</td>
<td>312</td>
<td>0.83</td>
<td>1.52</td>
<td>0.57</td>
<td>215</td>
<td>0.60</td>
</tr>
<tr>
<td>HPC1F</td>
<td>0.44</td>
<td>465</td>
<td>612</td>
<td>8.0</td>
<td>325</td>
<td>400</td>
<td>0.81</td>
<td>0.81</td>
<td>0.32</td>
<td>217</td>
<td>0.46</td>
<td>0.47</td>
</tr>
<tr>
<td>HPC2</td>
<td>0.44</td>
<td>465</td>
<td>610</td>
<td>3.1</td>
<td>218</td>
<td>165</td>
<td>272</td>
<td>1.32</td>
<td>0.85</td>
<td>0.46</td>
<td>306</td>
<td>0.46</td>
</tr>
<tr>
<td>HPC2F</td>
<td>0.44</td>
<td>465</td>
<td>610</td>
<td>8.0</td>
<td>262</td>
<td>428</td>
<td>0.61</td>
<td>0.94</td>
<td>0.42</td>
<td>223</td>
<td>0.49</td>
<td>0.49</td>
</tr>
<tr>
<td>HPC4</td>
<td>0.44</td>
<td>565</td>
<td>590</td>
<td>7.8</td>
<td>327</td>
<td>409</td>
<td>409</td>
<td>0.80</td>
<td>1.18</td>
<td>0.49</td>
<td>294</td>
<td>0.54</td>
</tr>
<tr>
<td>ISTD</td>
<td>0.44</td>
<td>605</td>
<td>605</td>
<td>8.0</td>
<td>397</td>
<td>400</td>
<td>0.99</td>
<td>1.43</td>
<td>0.35</td>
<td>240</td>
<td>0.45</td>
<td>0.45</td>
</tr>
<tr>
<td>IDL41R1</td>
<td>0.41</td>
<td>515</td>
<td>655</td>
<td>3.3</td>
<td>245</td>
<td>310</td>
<td>340</td>
<td>0.79</td>
<td>1.02</td>
<td>0.40</td>
<td>309</td>
<td>0.51</td>
</tr>
<tr>
<td>IDL44R1</td>
<td>0.44</td>
<td>515</td>
<td>655</td>
<td>8.0</td>
<td>254</td>
<td>321</td>
<td>0.79</td>
<td>1.43</td>
<td>0.65</td>
<td>303</td>
<td>0.56</td>
<td>0.56</td>
</tr>
<tr>
<td>ISL39R1</td>
<td>0.39</td>
<td>465</td>
<td>635</td>
<td>4.3</td>
<td>345</td>
<td>400</td>
<td>455</td>
<td>0.86</td>
<td>1.12</td>
<td>0.35</td>
<td>262</td>
<td>0.54</td>
</tr>
<tr>
<td>ISF39R1</td>
<td>0.39</td>
<td>465</td>
<td>635</td>
<td>4.8</td>
<td>327</td>
<td>440</td>
<td>440</td>
<td>0.74</td>
<td>0.70</td>
<td>0.22</td>
<td>181</td>
<td>0.42</td>
</tr>
<tr>
<td>ISL44R1</td>
<td>0.44</td>
<td>465</td>
<td>635</td>
<td>7.9</td>
<td>335</td>
<td>390</td>
<td>390</td>
<td>0.86</td>
<td>0.94</td>
<td>0.36</td>
<td>261</td>
<td>0.48</td>
</tr>
<tr>
<td>ISL50R1</td>
<td>0.50</td>
<td>465</td>
<td>635</td>
<td>8.0</td>
<td>208</td>
<td>281</td>
<td>281</td>
<td>0.74</td>
<td>1.60</td>
<td>0.80</td>
<td>348</td>
<td>0.60</td>
</tr>
<tr>
<td>ISF44R1-2</td>
<td>0.44</td>
<td>465</td>
<td>635</td>
<td>8.0</td>
<td>352</td>
<td>347</td>
<td>1.01</td>
<td>0.60</td>
<td>0.23</td>
<td>200</td>
<td>0.35</td>
<td>0.35</td>
</tr>
<tr>
<td>IKL44R1</td>
<td>0.44</td>
<td>545</td>
<td>560</td>
<td>6.7</td>
<td>315</td>
<td>360</td>
<td>380</td>
<td>0.88</td>
<td>1.00</td>
<td>0.60</td>
<td>269</td>
<td>0.35</td>
</tr>
<tr>
<td>IKF44R1</td>
<td>0.44</td>
<td>545</td>
<td>560</td>
<td>8.0</td>
<td>266</td>
<td>411</td>
<td>0.65</td>
<td>0.84</td>
<td>0.39</td>
<td>225</td>
<td>0.42</td>
<td>0.42</td>
</tr>
<tr>
<td>IBL44R1</td>
<td>0.44</td>
<td>445</td>
<td>570</td>
<td>3.5</td>
<td>237</td>
<td>470</td>
<td>470</td>
<td>0.64</td>
<td>0.87</td>
<td>0.47</td>
<td>309</td>
<td>0.46</td>
</tr>
<tr>
<td>IBF44R1</td>
<td>0.44</td>
<td>445</td>
<td>570</td>
<td>8.0</td>
<td>321</td>
<td>374</td>
<td>374</td>
<td>0.86</td>
<td>0.99</td>
<td>0.45</td>
<td>275</td>
<td>0.58</td>
</tr>
</tbody>
</table>

Average: 492, 610.8, 6.5, 290.8, 343.8, 378.3, 0.83, 1.05, 0.44, 261, 0.49
Standard Dev: 47, 31, 2, 57, 88, 58, 0.17, 0.29, 0.15, 47, 0.08

(stress and strength in psi, cement content in lbs/yd³, shrinkage in με)
5.3 SRA

5.3.1 Shrinkage and Stress Development

The addition of SRA 1 to the 0.35 and 0.45 w/cm ratio mixtures reduced free shrinkage by 30 to 60% after seven days of drying. The measured free shrinkage strains are shown in Figure 5-52 and Figure 5-53. The dosage of 0.75 gal/yd$^3$ was at the lower limit of the manufacturer’s recommendation. This demonstrates that lower dosages are effective at early ages. A greater reduction in shrinkage was observed for the 0.35 w/cm mixture. This mixture contained silica fume in addition to having a low w/cm ratio. This suggests that SRA reduces autogenous shrinkage as well as drying shrinkage, since this mixture would be expected to have a greater amount of autogenous shrinkage. Early expansion due to the addition of SRA was observed for the 0.35 w/cm mixture.

The addition of SRA reduced the free shrinkage of the 0.40 and 0.50 mixtures by 20–60% after 7 days drying. The magnitude of shrinkage reduction was proportional to the SRA dosages of 1 and 2 gal/yd$^2$. Free shrinkage results for these mixtures are shown in Figure 5-54 and Figure 5-55. In addition to the observation of reduced shrinkage, some specimens with SRA exhibited a slight expansion during the first day of testing. When compared to the 0.35 and 0.45 w/cm ratio materials, these mixtures have higher paste content, which may be associated with a higher rate and magnitude of expansion. However, the 1% SRA 1 mixture did not undergo the same expansion as the 1% SRA 2 mixture. This difference may be explained with a direct measurement of the extracted fresh concrete pore solution to determine the effectiveness of each chemical in reducing surface tension. Such a measurement has not yet been performed. The mechanism of expansion was discussed in Section 7.2.

Unrestrained shrinkage measurements of concrete prisms are shown in Figure 5-56 through Figure 5-59. These measurements correlate well with the reduction in shrinkage found for the uniaxial free shrinkage specimen. Early expansion of these materials is difficult to resolve with a conventional length change comparator, due to the low resolution of the measurement device (± 20 με). The magnitude of shrinkage reduction is proportional to SRA dosage for the 0.40 and 0.50 concretes, confirming the results of the uniaxial test.
Unrestrained shrinkage measurements were made of the mortar bars for one week using a standard length change comparator. The results shown in Figure 5-60 and Figure 5-61 confirm early expansion behavior with SRA that is proportional to the admixture dosage. The magnitude of shrinkage reduction increases with dosage as expected and the percent shrinkage reduction is comparable to measured values in concrete.

5.3.2 Discussion of Early Expansion

Early expansion was observed in most concrete mixtures containing SRA. The magnitude of expansion increased with admixture dosage. For concrete mixtures containing a dosage of 1% SRA, expansion amounts of 10 to 15 με were observed and the onset of tensile stress was delayed by up to 12 hours. In the 2% SRA mixtures, expansion of up to 70 με was observed and tensile stress was delayed by 24 to 48 hours. This expansive behavior is beneficial for the prevention of cracking since it delays the tensile stress that develops due to shrinkage, but an explanation of this behavior is needed in order to account for it in the analysis of creep and stress relaxation. Expansion is common in cases of wet cured or sometimes in sealed concrete specimens due to swelling pressure. However, in the case of expansion during drying, the explanation is not clear.

Several experimental issues were carefully studied and rejected as viable explanations for early expansion. The re-absorption of bleed water was eliminated as an explanation, since condensed liquid water was not observed on the specimens at the time of testing. Furthermore, an anomaly would not explain the occurrence of expansion in only specimens containing SRA. Environmental conditions were considered as a reason for early expansion, but the testing conditions were identical for every test and the climate control chamber used for this study had a variation of ± 0.5°C and ± 5% relative humidity. Other issues, such as incorrect strain readings due to a power surge or a disturbance of the test specimen, happen infrequently and are not seen in any of the test data presented in this study.

Two hypotheses are proposed to explain the phenomena of early expansion under drying conditions of concrete containing SRA. First, thermal behavior is considered as a reason for early expansion. Internal temperature measurements showed that the SRA did not significantly
affect hydration kinetics or temperatures. However, internal concrete temperature was carefully observed during each test and the temperature dropped 2 to 3°C after the initial removal of side formwork. This temperature change results in 20 to 30με of contraction if a typical value for the coefficient of thermal dilation in concrete is assumed. The experiment was started consistently one hour after the removal of the side forms from the specimen to allow the specimen to equilibrate to room temperature. The intent was to prevent premature load application during the restrained test, which occurs when a threshold strain of 8με is reached. During this time, internal concrete temperatures indicated that one hour was sufficient to allow for any evaporative cooling to occur. The temperature then proceeded to increase back to room temperature. The findings of this study support the view that during this temperature rise, drying shrinkage in specimens without SRA masked the thermal expansion that occurred. In concrete mixtures containing SRA, the suppression of drying shrinkage revealed the expansion attributed to thermal rise. This theory implies that the amount of expansion is dependent on the time of initial deformation measurement (“time-zero”) and the magnitude of temperature change before the initial measurement. Future improvements to the uniaxial device will include a new measurement system that is capable of measuring deformation from the time of casting, thus eliminating the dependence on the deformation measurement at time-zero. The limitation of the current measurement system is that the concrete must have sufficient strength to support the weight of an extensometer bar anchored into the concrete. An embedded-type or non-contact measuring device was used for very early measurements, including thermal deformation before and during the demolding process.

A second hypothesis for early expansion in concrete containing SRA is a reduction in surface tension reveals expansion from crystallization pressure associated with early hydration and microstructure development. Some hydration products are expansive and may cause sufficient pressure for volumetric expansion [5-14]. This expansion is probably always present, but in this case, the role of the SRA is to reveal the expansion by suppressing drying shrinkage. Realistically, the early expansion mechanism should combine both hypotheses, which would explain a case where the overall magnitude is greater than 20 to 30με (such as 2% SRA 1). Sealed shrinkage tests are recommended for further research in this area to validate these hypotheses.
5.3.3 Weight Loss and Drying Rate

Weight measurements of 0.35 and 0.45 concrete prisms in Figure 5-62 and Figure 5-63 show a reduction in weight loss with SRA. This suggests that the SRA is acting to reduce the drying rate of concrete. Shrinkage reduction has often been attributed to a reduction in the stress generated by capillary tension during the drying process. It was expected that a reduction in surface free energy would also increase the drying rate. Therefore, weight loss due to the addition of SRA requires further investigation when considering the dosage used in this study. Bentz has recently confirmed this result by demonstrating that the free liquid solution does exhibit a drying rate increase, but that in concrete samples it is difficult to determine the rate [5-23].

SRA did not significantly affect weight loss of concrete prisms for the 0.40 and 0.50 w/cm mixtures, as shown in Figure 5-64 and Figure 5-65. A slight increase was observed for the 1% dosage of SRA 1, indicating the lack of a consistent trend. Weight loss measurements of mortar bars in Figure 5-66 and Figure 5-67 also showed inconsistent behavior, indicating that SRA probably has little effect on this property. Weight loss specimens were not soaked prior to beginning the test, and this may have contributed to the differences in weight loss. It is recommended that weight loss and shrinkage measurements always be soaked in limewater before initiating the first measurement, as this allows for a consistent starting point for the measurement.

The dependence of unrestrained shrinkage measurements on weight loss indicates the mechanism of shrinkage reduction with SRA. Figure 5-68 and Figure 5-69 show weight loss measurements vs. free shrinkage measurements for 0.40 and 0.50 w/cm respectively. The results show that in most cases, SRA reduced the amount of free shrinkage per unit of weight loss. The mechanism of shrinkage reduction suggests this result, since the reduction of capillary stress should reduce shrinkage regardless of the drying rate. The SRA mixtures do not follow the same curve as the control, which indicates that shrinkage reduction will continue through long-term measurements. The 0.40 mixtures had a greater amount of shrinkage for the same amount of weight loss, due to a greater amount of autogenous shrinkage.
5.3.4 Hydration Kinetics

SRA did not significantly affect the peak internal temperature of concrete and the time at which the peak temperature occurred, as shown in Figure 5-70 and Figure 5-71. The initial fresh concrete temperature decreased as SRA dosage increased for the 0.40 \( w/cm \) ratio mixtures. A possible explanation is an increased evaporation rate of the fresh concrete with SRA, which reduces the initial temperature by increasing evaporative cooling. This effect was also observed by Bentz et al. when the properties of bulk solution containing SRA were measured [5-23]. The evaporation rate on hardened cement paste containing SRA decreased. Further investigation is required to study the effect of SRA on the drying rate, as also indicated by the weight loss measurements. Regardless, a reduction in initial temperature should not have a significant impact on shrinkage and creep behavior in the current study.

5.3.5 Tensile Strength Development

SRA did not significantly affect the early age tensile strength of concrete with \( w/cm = 0.35 \) and 0.45 as shown in Figure 5-72 and Figure 5-73. The tensile strength at 7 days was only slightly reduced for the 0.40 and 0.50 \( w/cm \) mixtures as shown in Figure 5-74 and Figure 5-75. Many researchers have reported a loss of strength of up to 10% with the addition of SRA [5-24, 5-25]. Manufacturer’s recommendations call for reduction of the \( w/cm \) ratio to offset the liquid content of SRA to achieve consistent workability (although it reportedly does not contain water), which offsets loss of strength. In this study, a direct comparison of \( w/cm \) ratio was desired to study shrinkage behavior. Therefore, a slight loss of strength was expected.

5.3.6 Restrained Stress Development

SRA significantly reduced the stress development under restrained drying conditions. Stress development over time is shown for the restrained tests in Figure 5-76 and Figure 5-77. Stress levels are 50 to 60% lower in concrete containing SRA, which reduces the probability for early age cracking. Time to failure due to restrained stress was delayed in mixtures containing SRA, indicating that early age cracking is delayed or prevented in concrete pavements and structures. Concrete will continue to gain strength over time, so if stress is reduced during the very early age when the concrete is one or two days old, the risk for cracking is greatly diminished.
Stress levels were approximately 50% of the concrete tensile strength at the time of failure. Previous work by Altoubat also showed that specimens failed well below their measured tensile strength. In his work, the rate of tensile strength gain was documented by split tensile strength measurements. Part of the difference may be attributed to the method for tensile strength measurement. The split cylinder test is an indirect measurement of tensile strength that is dependent on test conditions and may be as much as 5 to 12% higher than direct tensile strength [5-14]. Another factor in the stress at failure is the presence of a drying stress gradient in the specimen. As drying occurs, stress develops due to capillary tension in the pore microstructure. The stress, which is assumed constant in this study, is actually much greater at the surface where drying has occurred. The surface stress results in microcracking and causes damage to the specimen, which results in failure when the cracks propagate. The proper analysis for failure of concrete in direct tension should probably be based on a fracture energy approach. Comparison of the stress with the tensile strength at the macro level does not account for micro-mechanical processes that govern failure and the quasi-brittle nature of concrete [5-21].

5.3.7 Evolution of Elastic Modulus at Early Age

The elastic modulus was measured for concrete mixtures tested with the restrained test method each time the applied load was incremented. Each loading increment results in a finite amount of elastic strain as the specimen returns to original length from the threshold deformation of 0.005 mm. The elastic modulus was calculated as from the summation of accumulated stress divided by the summation of elastic strain according to Equation 5.3. Figure 5-78 and Figure 5-79 show the elastic modulus results for 0.40 and 0.50 materials respectively. The elastic modulus measurement decreases after the initial measurement for most mixtures. Then the measurements either level out or begin to increase slightly over the test period of one week. These trends do not represent the expected evolution of elastic modulus for concrete. The ACI equation was used to predict the elastic modulus, also shown in the Figures 33 and 34, according to Equation 5.4. The measured values for elastic modulus do not correlate with the magnitude or evolution of the ACI equation. To explain this contradiction, the nonlinear stress-strain behavior of concrete in tension was explored. The initial measurement of the elastic modulus from the first load application during the test, which is at relatively low stress and strain, corresponds to the initial tangent modulus, $E_1$ shown in Figure 5-37. At higher stress levels, the measurement
corresponds to the chord modulus, shown as $E_2$ in Figure 5-37 since it represents a linear slope of the current stress-strain measurements. Slope $E_3$ is a chord modulus at higher stress-strain levels, where the concrete exhibits highly nonlinear behavior. The initial tangent modulus is expected to be greater than the chord modulus. At higher levels of stress, the chord modulus decreases due to the non-linear behavior of concrete. In this testing program, stress levels frequently exceeded 40% of the measured tensile strength, which is cited by many researchers as the point at which nonlinear behavior typically begins [5-26]. Some elastic modulus measurements showed a gradual increase in stiffness after an initial drop. This behavior is specific to early age concrete in which the microstructure is continuously developing stiffness and strength, causing the stress-strain curve to increase over time. To illustrate this point, two hypothetical cases of stress-strain curves for early age concrete are presented. In case I, shown in Figure 5-38, the measured stiffness decreases with time, despite the increase in the stress strain curve with evolving microstructure. In case II, shown in Figure 5-39, the opposite effect is measured. By changing the rate of stiffness evolution over time, the elastic modulus appears to increase despite the nonlinear behavior at later measurements. Therefore, measurements depend on the rate of stiffness evolution, stress level at measurement, and the shape of the stress strain curve. More appropriately, the measurements of elastic modulus conducted in this study should be considered stiffness measurements, which account for nonlinearity at high stress levels in addition to the stiffness evolution over time. Measurement of the stress-strain curve in tension at various ages would provide a complete answer to the question of stiffness evolution.

5.3.8 Early Age Tensile Creep

Creep of concrete has been studied extensively for mature concrete under compressive loading. The importance of creep in concrete was first established from structural concerns about long-term deformation and prestress loss. Creep coefficient and specific creep are parameters that were developed to describe creep under sustained loading. They were originally single values that to be used in structural design to account for long-term deformation. The typical range for long-term creep coefficient of mature concrete in compression is 1.5 to 3.0 [5-14].

Recent research has focused on tensile creep of concrete, since it relates to the ability to resist cracking by relaxation of stress. Early age concrete in particular is sensitive to volumetric changes such as drying and autogenous shrinkage or thermal deformation. Stress relaxation is
important, since the material has not achieved full strength and is more susceptible to cracking. At early ages, the creep coefficient changes from a single value to a time curve. The shape of the curve depends on the evolution of microstructure and environmental conditions. Furthermore, the creep coefficient is not the same in tension as it is in compression. The initial rate of creep is higher in tension, which results in greater creep for relatively short durations of load [5-14]. At longer times compressive creep may exceed tensile creep.

Analytical models are used to predict the creep and shrinkage behavior of concrete. Some models are included in design codes such as ACI 209, RILEM B3, and CEB-FIP Model Code 90 [5-27, 5-28, 5-29]. The experimental data used to construct and validate these models was based primarily on compressive creep results from constant load tests on mature concrete. However, to predict early age cracking in concrete, we should consider tensile creep at early age and under restrained conditions. Recent research to modify the B3 model has resulted in the addition of a parameter to describe very early creep in tension [5-22].

5.3.9 Constant Load Tensile Creep Strain
Constant load tensile creep strain calculations are shown in Figure 5-80 and Figure 5-81. SRA reduced tensile creep strain after seven days of drying under constant applied load. The magnitude of creep reduction was much greater for the 0.35 w/cm mixture, which contained silica fume and superplasticizer. The reduction in free shrinkage was also much greater for the 0.35 mixture. Berke et al. [5-30] reported a synergistic relationship between silica fume and SRA. The admixture probably reduces autogenous shrinkage in addition to reducing drying shrinkage since both shrinkage mechanisms originate with capillary stress. Further testing of sealed specimens to measure autogenous shrinkage and creep are recommended to confirm this hypothesis.

5.3.10 Constant Load Tensile Creep Coefficient
Tensile creep coefficients were calculated for the constant load test by dividing creep deformation by the measured elastic strain at age of one day according to Equation 3. The constant load test results are shown in Figure 5-82 and Figure 5-83. After seven days of drying,
SRA reduced tensile creep coefficient under constant applied load. At constant stress the creep coefficient is directly proportional to creep strain. The reduction in creep coefficient is similar to the reduction in shrinkage, indicating that the admixture affects both creep and shrinkage in the same manner. The reduction in creep coefficient was greater for the 0.35 mixture.

### 5.3.11 Restrained Tensile Creep Strain

SRA reduced the tensile creep strain of concrete for the restrained load test. Creep strain measurements for restrained load tests are shown in Figure 5-84 and Figure 5-85. SRA reduced the early age stress development by reducing the magnitude of drying and autogenous shrinkage. Lower stress levels and a delay in stress development due to early expansion result in lower creep strain. This result does not imply that creep capacity of the material was reduced due to the presence of SRA. Creep is dependent on the stress level, so creep strain is not an effective measure of creep behavior between tests at the different stress level. The creep coefficient or compliance are more appropriate for relative comparison for creep capacity.

### 5.3.12 Restrained Tensile Creep Coefficient

SRA slightly reduced the restrained tensile creep coefficient for the 0.40 \( \frac{w}{cm} \) ratio materials, but had very little impact on the 0.50 \( \frac{w}{cm} \) creep coefficients. The results of tensile creep coefficient, calculated according to Equation 3, are shown in Figure 5-86 and Figure 5-87. A slight reduction in creep coefficient may be explained by the age at which the concrete develops stress due to drying shrinkage. Creep is lower for mature concrete, particularly when compared to very early age concrete that is one day old. Østergaard performed basic creep tests concrete at different ages and the results show that significantly higher creep is measured when load is applied at one day rather than three days [5-22]. It may then be inferred that SRA did not reduce the tensile creep in either case, but only delayed stress development. Since the concrete was slightly more mature when the initial load was applied, lower tensile creep was measured.

### 5.3.13 Conclusions

Ordinary and high performance concrete mixtures containing SRA have been evaluated using uniaxial creep-shrinkage test that is capable of applying constant load or simulating restrained
end conditions. The results of this study indicate that the addition of an SRA was effective in reducing the early age unrestrained shrinkage of concrete. Shrinkage reduction of up to 60% was measured in the first week after casting with the manufacturers recommended dosage of SRA. Shrinkage reduction was not proportional to weight loss measurements and SRA did not have a significant effect of the rate of weight loss of concrete. This is consistent with the theory of the mechanism of shrinkage reduction with SRA, which contends that SRA causes a reduction in shrinkage by reducing the surface tension of the pore solution. Reduction in unrestrained shrinkage was greater for mixtures with a lower \( \frac{w}{cm} \) ratio, indicating that SRA reduces autogenous shrinkage in addition to external drying shrinkage. The suppression of drying and autogenous shrinkage revealed expansion that persisted for up to 24 hours. The expansion could have been due to thermal changes, pressure from the formation of hydration products, or both.

Stress development due to drying under restrained conditions was reduced when SRA was added to the concrete mixture. Initial stress development was delayed in some cases due to early expansion of concrete containing SRA. Split tensile strength of concrete containing SRA was only slightly reduced and hydration kinetics were not affected. A reduction in stress without loss of strength delayed or prevented cracking of concrete containing SRA.

SRA reduced the tensile creep strain of concrete, but did not appreciably reduce the creep coefficient. Concrete containing SRA developed lower stress, and as a result had less creep strain. The results of this study indicate that creep coefficient is independent of stress level, but does depend on loading age for concrete of the roughly the same stiffness. A slight reduction in creep coefficient with SRA in the 0.40 \( \frac{w}{cm} \) mixtures may be attributed to a delay in initial loading from early expansion and the reduction of drying shrinkage stress.

With the addition of SRA to concrete, a reduction in stress due to suppression of drying shrinkage combined with only slight impact on strength and creep coefficient, delayed or even prevented cracking of early age concrete. A reduction in cracking produces concrete that is more durable and increases the service life of concrete structures.
Figure 5-52. Unrestrained uniaxial shrinkage for $w/cm = 0.35$

Figure 5-53. Unrestrained uniaxial shrinkage for $w/cm = 0.45$
Figure 5-54. Unrestrained uniaxial shrinkage for w/cm = 0.40

Figure 5-55. Unrestrained uniaxial shrinkage for w/cm = 0.50
Figure 5-56. Unrestrained shrinkage measurements for concrete prisms, \( w/cm = 0.35 \)

Figure 5-57. Unrestrained shrinkage measurements for concrete prisms, \( w/cm = 0.45 \)
Figure 5-58. Unrestrained shrinkage measurements for concrete prisms, w/cm = 0.40

Figure 5-59. Unrestrained shrinkage measurements for concrete prisms, w/cm = 0.50
Figure 5-60. Unrestrained shrinkage of mortar bars, $w/cm = 0.50$, SRA 1

Figure 5-61. Unrestrained shrinkage of mortar bars, $w/cm = 0.50$, SRA 2
Figure 5-62. Weight loss measurements of concrete prisms, $w/cm = 0.35$

Figure 5-63. Weight loss measurements of concrete prisms, $w/cm = 0.45$
Figure 5-64. Weight loss measurements of concrete prisms, $w/cm = 0.40$

Figure 5-65. Weight loss measurements of concrete prisms, $w/cm = 0.50$
Figure 5-66. Weight loss measurements of mortar bars, w/cm = 0.50

Figure 5-67. Weight loss measurements of mortar bars, w/cm = 0.50
Figure 5-68. Drying shrinkage vs. weight loss, \( w/cm = 0.40 \)

Figure 5-69. Drying shrinkage vs. weight loss, \( w/cm = 0.50 \)
Figure 5-70. Internal temperature measurements of concrete prisms, w/cm = 0.40

Figure 5-71. Internal temperature measurements of concrete prisms, w/cm = 0.50
Figure 5-72. Tensile strength, $w/cm = 0.35$

Figure 5-73. Tensile strength, $w/cm = 0.45$
Figure 5-74. Split tensile strength, \( w/cm = 0.40 \)

Figure 5-75. Tensile strength, \( w/cm = 0.50 \)
Figure 5-76. Restrained stress development, $w/cm = 0.40$

Figure 5-77. Restrained stress development, $w/cm = 0.50$
Figure 5-78. Elastic modulus, $w/cm = 0.40$

Figure 5-79. Elastic modulus, $w/cm = 0.50$
Figure 5-80. Tensile creep strain, \( w/cm = 0.35 \)

Figure 5-81. Tensile creep strain, \( w/cm = 0.45 \)
Figure 5-82. Tensile Creep Coefficient, w/cm = 0.35

Figure 5-83. Tensile Creep Coefficient, w/cm = 0.45
Figure 5-84. Restrained tensile creep strain, w/cm = 0.40

Figure 5-85. Restrained tensile creep strain, w/cm = 0.50
Figure 5-86. Restrained tensile creep coefficient, $w/cm = 0.40$

Figure 5-87. Restrained tensile creep coefficient, $w/cm = 0.50$
5.4 Development of Mechanical Properties at Early Age

5.4.1 Strength Development

The development of mechanical properties at early age is relevant to determining the susceptibility of concrete to cracking. Strength development was measured in accordance with ASTM tests, as discussed in Section 4.3. The purpose of these tests, other than to establish common material parameters, is to study the effect of proportioning of SCC and other materials on strength and stiffness. Using the results, it will be demonstrated if current model equations can be used effectively or modified to fit the observed behavior.

5.4.2 Stiffness Development

Sealed and drying tests were performed at early age in three prescribed stress programs. The zero stress program shown in Figure 5-88a consisted of a load application and unloading at 1, 3, 5, and 7 days. The loading rate was fixed and the entire stress-strain response at each time was recorded. In between the load applications, shrinkage was measured on the specimen in the drying test condition. Figure 5-88b shows the constant stress program, which consists of an applied stress at 40% of the ultimate tensile strength, as defined by the split tensile test results from an age of 1d. Creep was calculated in between each load application. The constant stress case was used to study the effects of stress history on aging and stiffness development. In the incremental stress program shown in Figure 5-88c, the stress level increased at each loading application. This program was used to study the effects of damage at higher stress level on stiffness development.

Figure 5-88. Stress programs for (a) zero stress (b) constant stress and (c) incremental stress
Results demonstrate that different stiffness is measured when concrete undergoes long term sustained load versus an incremental load. Figure 5-89 shows that concrete under sustained load exhibits a lower modulus when measured according to Figure 5-88b versus concrete exposed to incremental applied stress. It appears that a change in modulus occurs after concrete passes a threshold stress to which it has already been exposed. Once past this threshold the concrete behaves in a stiffer manner (higher elastic modulus) then for lower stress levels. Generally good agreement was found between tension and compression tests for elastic modulus, as demonstrated by Figure 5-89 and Figure 5-90. The level of damage that occurs in a uniaxial tension specimen under fully restrained conditions was observed through elastic modulus testing, as shown in Figure 5-91. Compressive elastic modulus measurements were performed at relatively low stress levels for uniaxial test specimens to avoid buckling of the test specimen and testing rig.

Figure 5-89. Elastic modulus measurements for concrete exposed to different stress histories
5.4.3 Compression vs. Tension

Although elastic modulus measurements in tension and compression seem to agree, other mechanical properties of concrete are different in tension and compression, especially when considering strength. A common rule of thumb is that tensile strength is 10 to 15% of compressive strength for mature concrete [5-14]. In the case of elastic modulus and creep, it is often assumed that the properties are similar. When considering mature concrete, several researchers have studied the differences between tensile and compressive creep. Neville et al.
found equal total creep in compression and tension at the same stress level [5-26]. However Illston [5-31] and Brooks [5-32] showed that the initial rate of creep was higher in tension than in compression. Over longer time periods, tensile creep was reportedly smaller than compressive creep.

At early ages, it is less understood how mechanical properties compare when considering tension and compression. Factors such as early stress development due to shrinkage and the relatively low stiffness of the hardened cement paste to the aggregate make it difficult to assume that elastic and viscoelastic properties are the same. Figure 5-92 demonstrates creep test results conducted on identical concrete materials at the same age (1 day), constant stress ratio (33%), and under the same environmental conditions (50% RH, 23°C). The result shows that concrete tensile creep is three times greater than compressive creep at early age. Figure 5-93 shows a similar result for 10% applied stress to strength ratio and the difference is even greater, with tensile creep exhibiting 10 times the deformation of compressive creep at early age. The presence of microcracking from skin stresses due to the drying gradient is likely a major contributor of measured tensile creep strain, as the other known creep mechanisms operating on the nanostructural scale would seem to function similarly in either tension or compression (sliding of C-S-H gel, seepage of interlayer water). Another contributing factor is that specimens in compression experience confinement due to friction at the boundary conditions from load contact points. Triaxial stresses are produced in the end regions in the specimen which is another explanation for lower specific creep under compression [5-21].
5.4.4 Non-Linearity of Creep at Early Age and Under Variable Stress

The linearity of creep in proportion to stress is proposed in literature to be valid up to 40-50% of the ultimate strength for mature concrete in compression [5-14]. Illston reported tensile creep to be linearly proportional to applied stress up to a stress/strength ratio of approximately 0.5 [5-31].
whereas Domone reported linearity to stress/strength ratios of 0.4 and 0.6 for saturated and sealed concrete respectively [5-33]. For early age concrete subjected to tensile stress from restrained shrinkage, stress-strength ratios can exceed 40% on the first day of drying. The tensile creep reported may also include microcracking, which contributes to nonlinear behavior. Microcracking and non-uniformity of stress in the cross-section may also be caused by the drying shrinkage gradient. The gradient tends to be most severe during the first few days of the drying period [5-17]. Generally microcracking is thought to occur only at high levels of applied stress, but it has been shown that due to the presence of a drying gradient, microcracking can occur at the surface even in unrestrained shrinkage specimens [5-18, 5-19, 5-34, 5-35].

The RSTM was performed for different stress levels to identify the age at which the concrete creep-stress relationship can be defined as linear and the stress levels where the linearity assumption is appropriate. The importance of this portion of the study is for modeling tensile creep at early age, where practical limits to current models must be properly defined. Structural creep calculations often depend on the principle of superposition, which depends on the linearity of the creep function. The original authors of popular models such as B3 and ACI did not intend for the application of these models to early age tension and therefore did not recommend guidelines for their use. The lack of experimental data for early age tensile creep also prohibited developing such guidelines.

![Figure 5-94. Specific creep at early age under constant stress showing the dependence on loading age for SCC1](image-url)
Figure 5-95. Specific creep of SCC 1 at different applied constant stress levels

Figure 5-96. Specific creep of SCC 4 at different applied constant stress levels
5.4.5 Re-visiting Superposition for Drying Creep Analysis

The superposition analysis for shrinkage and creep strain has been utilized to study restrained concrete by many researchers and is common in structural design [5-16]. Pickett observed that the creep deformation of a specimen loaded in compression during drying was greater than creep measured in a sealed specimen and shrinkage added together from separate specimens [5-36]. This led to the term Pickett effect, also referred to as drying creep or stress-induced shrinkage, to describe the additional creep that was not explained with superposition. The mechanism of drying creep has not been well established, although several theories exist. L’ Hermite [5-37] and Neville [5-26] both recognized that creep and shrinkage are related, and observed that factors affect shrinkage and creep in a similar manner and thus the drying creep effect was based on the interaction of the two processes.

Shrinkage and creep strain superposition is a necessary assumption for modeling time-dependent deformations in concrete. Shrinkage and creep are caused by the same mechanism, the viscoelastic response of concrete to stress, and that response has a non-linear component (particularly at early age). An analysis based on the driving source for stresses from drying and self-desiccation can improve our understanding of the mechanism of drying creep.
One theory of drying creep focuses on the mechanism of microcracking [5-36, 5-38]. During drying, a stress gradient develops due to the non-uniformity of relative humidity in a specimen. The outer surface has been estimated to be at very high tensile stress [5-18, 5-19, 5-34], beyond the point where damage is thought to occur. In a free shrinkage specimen, the tensile stress causes microcracking, but in a compressive creep specimen, the entire cross section is under net compression. Due to irrecoverable creep and damage caused by the stress gradient, the measured shrinkage is thought to be less than the potential shrinkage. Shrinkage was greater under compressive loads and less under tensile loads, leading Wittmann to the landmark conclusion that shrinkage was not a material property, but dependent on the state of stress from drying [5-38].

Another theory proposes that when under stress, the amount of shrinkage is altered due to the existence of two moisture diffusion processes; Macro-diffusion and Micro-diffusion [5-39]. The macro-diffusion process exists in large capillary pores without affecting deformation, whereas the micro-diffusion process transports water from capillary pores to gel pores, where the deformation was affected by molecular interaction of water with the hydration products. Bažant
and Xi compared the curvature creep of beams subjected to the same bending moment but very different axial forces [5-40]. Microcracking and stress-induced shrinkage were identified as the sources of drying creep. Kovler examined drying creep in tension. His suggestion was to analyze drying creep as the sum of shrinkage-induced creep and creep-induced shrinkage, which explained both the compressive and tensile drying creep behavior [5-41]. However, he concluded in a later study that drying creep cannot be due to stress induced shrinkage because it results in additional creep strain in both compression and tension [5-42]. He then postulated only one mechanism is responsible for drying creep and it can be explained by the change in volume of capillary pores during loading, which changes the radii of curvature of the meniscus resulting in greater stress, and thus greater creep.

In an effort to further understand drying creep, Altoubat utilized the RSTM to separate the mechanisms in tension [5-1, 5-5]. In his procedure, a restrained creep test was performed to measure the stress development and total creep during drying. Then, a basic creep test was performed under the same stress history as the drying test for both sealed and moist cured conditions. This allowed for the separation of drying creep, and a damage based approach was used to model microcracking. The microcracking was attributed to the softening behavior of concrete at relatively high stress due to the non-uniformity of stress caused by the drying gradient.

A relative humidity system has been used to measure drying profiles in concrete to determine the stress distribution during drying [4-21]. This system can be incorporated into the RSTM analysis in order to quantify drying stress and examine creep according to the applied stress. It is proposed that an explanation for drying creep lies in properly accounting for the magnitude of stress during drying with creep being proportional to the sum of internal residual stress and externally applied stress.

To perform this analysis, a finite element software program developed at UIUC called ICON was selected to construct a layered analysis of the RSTM [5-43]. This model allows the superposition of stress, instead of strain to analyze the experiment. By variation of the specimen cross-section, an apparent size effect may be used to identify the drying creep mechanism in terms of high creep or damage in microcracked zones. Wider formwork was adapted to the
linkage of the RSTM to enable larger cross-sections to be utilized in the test as shown in Figure 5-99. The results are shown in Figure 5-99 and demonstrate that a narrow cross section undergoes more creep than a wider cross section under that same applied stress. This result reveals that drying creep plays a larger role in smaller concrete sections, possibly causing more non-linearity and more microcracking damage due to the drying stress gradient.

Figure 5-99. Drying creep specimens for size effect approach (top section view)

Figure 5-100. Investigation of size effect in uniaxial tensile creep of concrete

5.4.6 Autogenous Shrinkage and its Role in Creep Analysis

Autogenous shrinkage measurements should be incorporated into practical models that can be used in conjunction with existing prediction models such as ACI 209 model and the B3 model [5-27, 5-28]. Currently, the models do not treat autogenous deformation directly. The ACI 209 model was not intended for low w/cm ratio concrete and when it was developed in the 1960s
autogenous shrinkage was not a significant concern in concrete structures [5-44, 5-45]. The B3 model, while it was proposed to work at ratios as low as 0.25, only includes autogenous shrinkage on an empirical basis, and it only appears to correlate with later age test results. Early autogenous shrinkage is neglected due to the lack of test results in the database that was used to calibrate the model. It can be seen in Figure 5-101 that the total shrinkage prediction at early age decreases proportionally with the $w/cm$. Experimental results in Figure 5-102 show that the opposite is true at early age. The total shrinkage is dominated by autogenous shrinkage, driven by self-desiccation, and can be greater for low $w/cm$ ratio materials in the first days after casting. To incorporate this behavior into the B3 model, first the experimental results were modeled as a function of $w/cm$ ratio and paste content. Models in literature were evaluated for their usefulness at early age and for the materials considered in this study, but ultimately a new approach was sought for its dependence on internal RH.

The effect of autogenous shrinkage on creep measurements was evaluated. Previous research has seldom considered the effect of autogenous shrinkage on the true magnitude of stress at early age; however it may be significant when studying materials at low $w/cm$ ratio and high paste content. It is also a worthy of discussion when considering the measurement of basic creep, whether or not a specimen should be moist cured. When autogenous shrinkage occurs, it has been shown that basic creep measurements are affected [5-1]. In an effort to suppress autogenous shrinkage in creep testing, Altoubat used a wet cloth to cure specimens during testing. The presence of water has the potential to affect temperatures during the test, so the evaporation rate must be carefully controlled for the test results to be consistent. In addition, in cases of very low $w/cm$ concrete the wet curing may not be capable of suppressing autogenous shrinkage entirely, as the porosity of the concrete will be so low that the additional curing water cannot reach beyond the outer few millimeters. So a conclusion of this work is that basic creep of concrete with very low $w/cm$ ratio cannot be measured experimentally at early age, due to the presence of residual stresses from autogenous shrinkage or the presence of curing water to suppress autogenous shrinkage.
Figure 5-101. B3 shrinkage prediction at early age does not account for autogenous shrinkage

Figure 5-102. The effect of autogenous shrinkage at low w/cm ratio on total shrinkage rate at early age
5.5 References

5-1 Altoubat, S.A., Early age stresses and creep-shrinkage interaction of restrained concrete, Ph.D. Thesis, Department of Civil and Environmental Engineering, University of Illinois at Urbana-Champaign (2000).


D’Ambrosia, M. D., Lange, D. A., Modeling Early Age Tensile Creep and Shrinkage of Concrete, ACI-SP 227 (2004).


5-31 Illston, J. M., The creep of concrete under uniaxial tension, Mag Con Res 17 (51) (1965) 77-84.
5-34 Cady, P., Clear, K., Marshall, L., Tensile strength reduction of mortar and concrete due to moisture gradients, ACI J Nov (1972) 700-705.
5-37 L’Hermite, R., What do we know about the plastic deformation and creep of concrete?, RILEM Bull., No. 1, Paris, France, 21-51.


6. ANALYSIS

6.1 Introduction and Problem Statement

A technique for modeling concrete shrinkage as a response to internal stress in an aging viscoelastic porous material is presented. The technique uses measured internal relative humidity (RH) and the theoretical relationship between RH and capillary pressure to predict shrinkage stress, which is then applied to an aging viscoelastic continuum using the finite element method. An application of the material model for concrete accounts for aging material properties using the solidification theory. Predictions of unrestrained shrinkage for drying and sealed conditions are shown to be in good agreement with laboratory measurements. This chapter presents basic research on stresses in partially saturated porous materials, modeling approaches for gradients that exist in drying concrete, and how the model is applicable to structures.

Many similarities between shrinkage and creep behavior of concrete have been observed in previous research [6-1, 6-2]. These studies have shown that both phenomena responded similarly to changes in the environment, w/cm ratio, paste content, etc. In this study, the similarities are attributed to the fact that creep and shrinkage of concrete are both a viscoelastic response to stress, where creep is due to external applied loads and shrinkage is due to internal stress. Furthermore, it was shown that shrinkage deformation can be predicted if the magnitude of the internal stress and the constitutive properties of the materials are known. The advantages of this approach are that size effects due to the nonlinear gradient of drying stresses, age of drying effects and ambient environment (RH) effects are taken into account.

A traditional modeling approach for incorporation of hygrothermal effects is shown in Figure 6-1, where the strains are superposed in structural models using simulated temperature and changes to the elastic modulus. A new approach is proposed, shown in Figure 6-2, where the material model is dependent on the internal RH.
Figure 6-1. Traditional modeling approach for implementation of creep and shrinkage in concrete structures

Figure 6-2. New modeling approach, with solidifying material element with dependence on internal RH

The mechanism of volume change in porous materials is widely attributed to changes in capillary pressure due to reduction of internal relative humidity [6-3]. This relationship is expressed by the Kelvin-Laplace equation. A reduction in capillary pressure causes the solid microstructure of cement paste to be compressed, generating an internal compressive stress in the material. Shrinkage of cement paste is a time-dependent viscoelastic response to this compressive stress. Furthermore, just as concrete response to externally applied loads is affected by the aging (hydration) of cement paste, shrinkage is dependent on the initiation of drying because of aging. By implementing a modeling approach that treats shrinkage as a response to internal stress, the changes in size and geometry (stress gradient), drying age, and ambient RH are treated by the applied internal stress condition. Therefore, they are no longer empirically related to the material and a model can be applied generally to any conditions. The model concept has been implemented numerically in a finite element code and is capable of full scale simulations of concrete elements, creating a versatile tool that can predict drying stresses.
6.2 Modeling Approach

The relationship between capillary pressure reduction and relative humidity in a porous solid is defined by the Kelvin-Laplace equation given by

\[ p = \frac{2\gamma}{r} = -\frac{\ln(RH)RT}{v'}, \quad (6.1) \]

where \( p \) is the pore pressure, \( \gamma \) is the surface tension, \( r \) is the mean pore radius, \( R \) is the universal gas constant, \( T \) is the temperature in kelvins, and \( v' \) is the molar volume of water. \( RH \) is the relative humidity, which must be adjusted for the initial equilibrium \( RH \) due to salt concentration [6-4]. It is assumed that the relative concentration of salts in the pore solution does not change greatly with time due to drying or continued hydration. The concentration levels are likely close to saturation, which would indicate little further change is possible. Typically, the initial \( RH \) in concrete is between 94% and 96%. This initial equilibrium point can be predicted using Rault’s law based on the pore solution chemistry.

The reduction in internal pressure accompanied by a drop in \( RH \) generates a compressive stress on the solid skeleton of cement hydration product. By treating the internal pressure as a hydrostatic force, an applied stress on the solid skeleton is determined. Because the pore structure is not completely saturated, a reduction factor was used for the hydrostatic load, assumed to be approximately equivalent to the measured internal \( RH \). Over time, a volumetric contraction of the solid occurs which is a viscoelastic response of the hydration product to compressive stress.

Researchers have shown this relationship to be applicable to materials such as glass and cement paste [6-4, 6-5, 6-6]. The cement hydration product is viscoelastic, so an elastic solution is only approximate as shown by Lura [6-4] and Grasley et al. [6-5]. In the case of cement paste, analytical viscoelastic solutions have been developed that show a time dependent solution is necessary to predict the deformation [6-5]. Furthermore, an aging material function must be used to capture the changing material properties due to continued hydration, as well as other aging effects such as increasing polymerization of the cement paste.
The aging material properties of hydrating cement paste are treated using the solidification theory proposed by Bažant [6-7]. The theory uses a volumetric growth function to represent an increasing amount of solid hydration product. The hydration product itself is considered as a non-aging material. The aging effect on concrete mechanical properties is caused by the increasing growth of new hydration product, which forms in a stress free state.

### 6.3 Material Model for Aging Concrete

The material models account for aging viscoelastic behavior and response to changes in internal RH and temperature. This is accomplished by using two sets of material parameters. One set can be considered the internal material response, which implies that the material properties are representative of the solid skeleton. The external set, on the other hand, has material properties derived from the porous body and primarily constitutes the response due to external loads. A diagram of this model concept is shown in Figure 6-3. The internal material model responds to the pressure calculated from the RH change as well as any external applied stress. The other material model responds only to an applied stress. The combination of the models allows for a coupled analysis of creep and shrinkage for an aging viscoelastic material.

![Diagram of material model](image)

**Figure 6-3.** Model concept diagram showing internal and external material models
The total deformation in the solid body for a 1D representative element is given by

$$\varepsilon_{\text{total}} = \varepsilon_{e_1} + \varepsilon_{e_2} + \varepsilon_{r_1} + \varepsilon_{r_2},$$  \hspace{1cm} (6.2)

where \( \varepsilon_{e_1} \) and \( \varepsilon_{e_2} \) are the instantaneous elastic deformation of the internal and external components respectively, \( \varepsilon_{r_1} \) and \( \varepsilon_{r_2} \) are the time-dependent viscoelastic deformation of the internal and external components respectively, and \( \varepsilon_{\text{total}} \) is the total deformation in the body.

The solidification theory was used to account for aging of the material properties [6-8]. The model consists of three components, creep magnification factor for a high stress to strength ratio, effective load-bearing volume growth of the product, and non-aging viscoelastic response of the hydrated product. The creep rate of each viscoelastic component is given by

$$\dot{\varepsilon}_{r_1} = F(s) \frac{\dot{\gamma}_{r_1}}{v(t)}$$ \hspace{1cm} (6.3)

and

$$\dot{\varepsilon}_{r_2} = F(s) \frac{\dot{\gamma}_{r_2}}{v(t)},$$ \hspace{1cm} (6.4)

where \( \dot{\gamma}_{r_{1,2}} \) is the non-aging viscoelastic response and the effective load bearing volume \( v(t) \) is given by

$$\frac{1}{v(t)} = \left( \frac{\lambda_0}{t} \right)^m + \alpha,$$ \hspace{1cm} (6.5)

and \( \lambda_0, m \) and \( \alpha \) are material constants that depend on the rate of hydration. The creep magnification factor \( F(s) \) is a dimensionless scalar function given by

$$F(s) = \frac{1 + s^2}{1 - s^{10}},$$ \hspace{1cm} (6.6)

where \( s \) is the stress to strength ratio. The ratio \( s \) can be in compression or tension depending on the application. For a three-dimensional extension, one may define \( s \) as a function of stress.

134
invariants and the strengths. For the current work, $s$ was defined based on the Mohr-Coulomb failure criterion given by

$$s = \frac{\sigma_1 - \sigma_3}{f_i - f_c}, \quad (6.7)$$

where $\sigma_1, \sigma_3, f_i$ and $f_c$ are maximum principle stress, minimum principle stress, uniaxial tensile strength and uniaxial compressive strength respectively.

The non-aging viscoelastic response of the model was implemented using separate chains of Generalized Kelvin Models (GKMs) for both the internal and the external material components as shown in Figure 6-4. For an incremental time step $i$ of time $\Delta t$, the total strain in the body can be solved according to

$$\Delta \varepsilon_{total} = \Delta \varepsilon_{el_1} + \Delta \varepsilon_{cr_1} + \Delta \varepsilon_{el_2} + \Delta \varepsilon_{cr_2}, \quad (6.8)$$

where

$$\Delta \varepsilon_{el_1} = \frac{\Delta \sigma + \Delta p}{E_1} \quad (6.9)$$

and

$$\Delta \varepsilon_{el_2} = \frac{\Delta \sigma}{E_2} \quad (6.10)$$

are the instantaneous strain increments for the internal and external material components, $E_1$ and $E_2$ are the elastic moduli of the internal and external material functions, and

$$\Delta \varepsilon_{cr_1} = \frac{F(\sigma_o + p_o)}{v(t_o)} \sum_{i=1}^{n} \left[ \frac{\Delta \sigma + \Delta p}{K_i} \left(1 - \frac{\tau_i}{\Delta t} \left(1 - e^{-\gamma_i} / \mu_i \right) \right) + \left(\frac{\sigma + p}{K_1} - \gamma_i \right) \left(1 - e^{-\gamma_i} / \mu_i \right) \right], \quad (6.11)$$

and

$$\Delta \varepsilon_{cr_2} = \frac{F(\sigma_o)}{v(t_o)} \sum_{i=1}^{n} \left[ \frac{\Delta \sigma}{m_i} \left(1 - \frac{\tau_i}{\Delta t} \left(1 - e^{-\gamma_i} / \mu_i \right) \right) + \left(\frac{\sigma}{m_i} - \beta_i \right) \left(1 - e^{-\gamma_i} / \mu_i \right) \right]. \quad (6.12)$$
are the viscoelastic strain increments, following the work of Bažant [6-8].

The compliance function at each step is then computed according to

\[
J = \left[ \frac{1}{E_1} + \frac{1}{E_2} + \frac{F(\sigma_o + p_o)}{v(t_o)} \sum_{i=1}^{n} \frac{1}{k_i} \left\{ 1 - \frac{\tau_i}{\Delta t} \left( 1 - e^{-\Delta t/\gamma_i} \right) \right\} + \frac{F(\sigma_o)}{v(t_o)} \sum_{i=1}^{n} \frac{1}{m_i} \left\{ 1 - \frac{\mu_i}{\Delta t} \left( 1 - e^{-\Delta t/\mu_i} \right) \right\} \right].
\] (6.13)

If we let

\[
R = \frac{\Delta p}{E_1} + \frac{F(\sigma_o + p_o)}{v(t_o)} \sum_{i=1}^{n} \left( \frac{\Delta p}{k_i} \left\{ 1 - \frac{\tau_i}{\Delta t} \left( 1 - e^{-\Delta t/\gamma_i} \right) \right\} + \left( \frac{\sigma + p}{k_i} - \gamma_i \right) \left( 1 - e^{-\Delta t/\gamma_i} \right) \right) + \frac{F(\sigma_o)}{v(t_o)} \sum_{i=1}^{n} \left( \frac{\sigma}{m_i} - \beta_i \right) \left( 1 - e^{-\Delta t/\mu_i} \right)
\] (6.14)

then \( \Delta \sigma \cdot J = \Delta \varepsilon_{\text{total}} - R \),

(6.15)

and the incremental stress can be determined according to

\[
\Delta \sigma = J^{-1} \cdot \Delta \varepsilon_{\text{total}} - J^{-1} R.
\] (6.16)


6.4 Finite Element Implementation

The three-dimensional finite element analysis code ICON was developed at University of Illinois for the analysis of time-dependent deformation and stress analysis of concrete structures [6-9]. The model set developed for aging concrete was implemented in the code. With the assumption that the viscoelastic Poisson’s ratio is the same as the elastic case, the extension can be made to 3-D. This assumption has been shown to be adequate for concrete materials, but may be considerably less accurate for cement paste [6-5].

6.5 Model Calibration

Embedded strain gages were used to obtain deformation measurements from the time of casting. The main advantages over conventional length change comparators are (1) continuous measurements are possible with a strain recorder or data acquisition system and (2) for autogenous (sealed) deformation, demolding is not necessary, allowing for permanently sealed specimens. Demolding of early age specimens can cause significant errors in shrinkage measurements due to evaporative cooling or brief periods of drying [6-10]. In addition to providing a long term continuous measurement of strain, the initial measurable response of the embedded gages roughly correlates to the time of set. As the material sets and immediately undergoes volume change, the gage responds to the stress exerted on it by the surrounding cement paste, thereby resulting in measured strain. Prior to set the volume change that occurs does not cause stress, and therefore is neglected. This indication can be useful when trying to assess the development of early age stresses.

The deformations of concrete specimens were measured with an embedment-type strain gage starting from the time of casting. The sensors were mounted in a 100x200mm (4x8-in.) cylinder mold using a thin wire suspended through the center of the mold as shown in Figure 4-7. The holes left by the wire were then filled using a putty sealant and the sides of the mold were coated with a debonding agent to reduce friction. Temperature and relative humidity (RH) were measured in the center of the specimen using a digital RH sensor shown in Figure 4-9. The development and validation of this system for measuring RH in concrete has been published elsewhere [6-5]. The drying profile was measured using digital RH sensors inserted to different
depths in a 75x75mm (3x3-in.) concrete prim specimen as shown in Figure 4-10. Drying began at an age of one day and was carried out to an age of at least 90 days.

6.6 Model Validation

Relative humidity measurements for sealed specimens are shown in Figure 6-5. The RH shown in the figure is relative to the initial RH at time of set. The initial RH is typically lower than 100% due to salt concentrations, but this initial RH reduction does not cause stress in the concrete before set, so shrinkage stress prediction should be based on the change in RH that occurs after set. The change in relative humidity for sealed concrete is driven by self-desiccation and the overall magnitude of RH change is generally comparable to other studies in literature [6-4, 6-5]. As expected, the material with the lowest w/cm ratio had the greatest drop in RH, indicating more self-desiccation had occurred, thus increasing the driving force for shrinkage.

![Figure 6-5. Internal relative humidity reduction measurements in sealed concrete specimens](image)

The autogenous shrinkage data shown in Figure 6-6 was used to calibrate the creep functions in the model. A series of four Kelvin chains was used in each creep function (internal and external) with retardation times $\tau$ and $\mu$ of 0.1, 1, 10, and 100 days. The coefficients $k_i$ and $m_i$ were selected for each material to fit the autogenous shrinkage of concrete as shown in Figure 6-6.
Figure 6-6. Autogenous shrinkage measurements and calibrated shrinkage model

Figure 6-7. Model calibrated to autogenous shrinkage and basic creep tests
For the drying case, measured RH profiles were used as input to a 3D finite element implementation of the model for three different materials. Figure 6-8 through Figure 6-10 show the drying behavior over time up to 120 days. The material functions determined by fitting autogenous shrinkage data were used without any modification. The results are shown in Figure 6-11. It can be observed that the model is in sufficient agreement with the data for the drying case without any change to the material function. This indicates that by using RH as a driving force, drying shrinkage can be modeled by using a single autogenous shrinkage test for calibration of the creep function. The resulting model accounts for gradients of stress and changes in geometry and size provided that the internal RH can be determined.

Figure 6-8. Relative humidity for drying prism specimen, $w/cm = 0.32$
Figure 6-9. Relative humidity for drying prism specimen, $w/cm = 0.38$

Figure 6-10. Relative humidity for drying prism specimen, $w/cm = 0.41$
Figure 6-11. Prediction of drying shrinkage for three different concrete materials

A prediction of the deformations in the RSTM test are shown in Figure 6-12 and the 3-D stress distribution determined using the model is shown in Figure 6-13. The model predicts the maximum tensile stress in the longitudinal direction for an unrestrained drying concrete prism will reach 1.4 MPa after 120 days. Cracking would not be expected for the magnitude of stresses generated in the unrestrained case. The model predicts that failure in the test will occur at 11 days, and the actual failure occurred at 9 days, which is a reasonable prediction. The deformations near the end of the model do not appear to correlate well with the experiment, possibly due to damage that occurs later in the test, which contributes to non-linearity of the material. In Figure 6-14, the maximum stress distribution is shown for a restrained concrete wall. The maximum tensile stress after 120 days is over 9 MPa, indicating that cracking is likely to occur. The model does not currently include modules for softening or post-cracking behavior, but this may be a useful addition for future applications.
Figure 6-12. Model prediction of the RSTM behavior using SCC1

Figure 6-13. Stress distribution due to drying in an unrestrained concrete prism after 120 days, 75mm x 75mm x 1m (3x3x39.5-in.)
A shrinkage deflection test shown in Figure 6-15 was developed specifically for the purpose of validating the 3D finite element model. In this test, one side of the uniaxial specimen is exposed to drying while the other side is sealed, promoting a differential drying gradient to develop. As the gradient develops, differential stresses cause the specimen to deflect laterally. Deflection was measured using two dial gages mounted on the end of the sample, as shown in the figure. This simple test was useful for calibration purposes, although it was very sensitive to disturbances and vibrations. An interesting aspect of the test result, shown in Figure 6-16, demonstrate the ability of creep to eventually relax the curling stress that occurs in the sample. The ICON model using RH as an input parameter predicts this behavior, although it slightly over predicts the deflection that occurs.

A final validation experiment was conducted using the restrained ring test apparatus. The test was performed in accordance with AASHTO T334, which consists of a 3-in. thick concrete ring surrounding a $\frac{1}{2}$-in thick steel ring as shown in Figure 6-17. The test was modeled in ICON as a
one-eighth of a symmetric ring, as shown in Figure 6-19. The model prediction of the measured steel strain due to restrained concrete shrinkage is shown in Figure 6-18. The model under predicts the strain result of the ring test slightly. The maximum stresses versus time at various depths into the concrete from the surface were examined and are shown in Figure 6-20, demonstrating the ability of creep to relax tensile stresses at the surface. The figure shows that initially the maximum stress is at the surface, but over time the maximum tensile stress progresses inward both due to progression of the drying gradient inward and due to the relaxation of the high surface stresses over time. Eventually cracking occurs in the ring when the tensile stress at a depth 1-in. into the concrete ring is equal to the stress near the surface of the ring. The predicted stress does not equal the tensile strength at failure, but is approximately 80% of the measured strength, which is consistent with uniaxial test results. This discrepancy is likely explained by the presence of microcracking in both the uniaxial and ring test geometries.

Figure 6-15. Differential shrinkage test for validation of ICON 3-D finite element model
Figure 6-16. Model prediction of differential shrinkage deflection test result, SCC1

Figure 6-17. Restrained ring testing for SCC1
Figure 6-18. ICON 3D model representation of the restrained ring test specimen SCC1, illustrating the peak stress occurrence beneath the concrete surface at the time of cracking.

Figure 6-19. Model prediction of restrained ring result for SCC1
6.7 Modeling Conclusions and Extensions

A model was developed for stress analysis of concrete during drying that treats concrete shrinkage as a response to internal stress in an aging viscoelastic porous material. The technique uses measured internal relative humidity (RH) to predict internal shrinkage stress, which is then applied to an aging viscoelastic continuum using the finite element method. Predictions of unrestrained shrinkage are in good agreement with laboratory measurements and the following conclusions can be drawn from this work:

- RH is a powerful parameter for modeling drying shrinkage and stress development. Using the RH-capillary stress relationship as driving force, a combined autogenous/drying shrinkage model was obtained.
- The fundamental concept for this model uses intuitive representations of the microstructural properties of concrete. The inner model component can represent C-S-H hydration product properties, meaning that mechanisms such as sliding of polymer sheets can be physically represented, while the second (outer) component of the model can represent pore fluid based or microcracking mechanisms on a larger scale. This
fundamental separation of scale allows for the use of two aging functions also, which could be used to represent aging polymerization (inner) or solidification (outer).

- After calibrating the autogenous shrinkage model by choosing an appropriate creep function, a measured drying profile was imposed using RH data and the model predicted drying shrinkage and stress distribution.
- An example application of the model to a full scale concrete structure is shown where the maximum stresses were likely high enough to cause cracking. This example demonstrates the capability of the model to investigate cracking risk, or to determine residual stresses that could be added to design stresses.
- The model was validated using several different experimental geometries, including the uniaxial RSTM and the popular restrained ring test. The results demonstrate the ability of the model to predict behavior of the drying gradient in various cases, allowing for the simulation of stress gradients in real structures.

6.8 References


6-10. D'Ambrosia, M. D., D.A. Lange, Modeling Early Age Tensile Creep and Shrinkage of Concrete, Shrinkage and Creep of Concrete, ACI SP-227, New York, NY, USA, 2005, pp 349-366.
7. PRACTICAL IMPLEMENTATION

7.1 Updating Predictive Models of Concrete Creep and Shrinkage for Early Age

The current models available in literature for structural calculation of creep and shrinkage were developed primarily to predict drying shrinkage and compressive creep in mature concrete. The prediction of tensile creep at early age due to restrained drying and autogenous shrinkage requires careful reexamination of these equations. As the database of early age tensile creep and shrinkage increases, the modification of these models to account for early age behavior becomes possible. Guidelines for usage are needed to improve the usefulness of the models in both research and practice. The experimental results were modeled to identify the strengths and deficiencies of the most popular models. The work considers constant load and incremental restrained load cases for measurement and modeling of early age tensile creep and shrinkage of concrete. An experimental program will measure early age tensile creep and shrinkage and the results will be used to develop suggestions for improving existing models.

7.1.1 The ACI 209 Model

The ACI model recommended by committee 209 is based on the work of Branson et al. [7-1]. It uses empirical creep correction factors for curing, relative humidity, load duration, slump, aggregate, and air content to modify the ultimate creep coefficient, \( v_u \). The compliance function \( J(t,t') \) at time \( t \) can then be calculated from the ultimate creep coefficient \( v_u \) according to

\[
J(t,t') = \frac{1}{E(t') \left( 1 + \frac{(t-t')^\psi}{d + (t-t')^\psi} v_u \right)}
\]

where \( E(t') \) is Young’s modulus of elasticity at the loading time \( t' \), and \( d \) and \( \psi \) are constants.

The shrinkage strain \( \varepsilon_s(t,t_u) \) is modified by correction factors for relative humidity, duration of drying, slump, cement content, aggregate, and air content and is given by the equation

\[
\varepsilon_s(t,t_u) = \frac{(t-t_u)^\psi}{f_c + (t-t_u)^\psi} \varepsilon_u^s
\]
where $t_o$ is the time at which drying begins, $f_c$ and $\alpha$ are constants, and $\varepsilon_u^s$ is the ultimate shrinkage. Recommendations are given for each constant, based on standard test conditions and equations for each parameter account for deviations from the standard set of conditions.

It can be seen from the preliminary results shown in Figure 7-1 that the ACI equation can be fit to experimental data at early age, demonstrating that the form of the function is reasonable. However, the fit of the model is no longer predictive, requiring a parameter to be modified outside of the recommended range.

![Figure 7-1. Prediction of creep strain with ACI 209 equation](image)

### 7.1.2 The B3 model

The B3 model developed by Bažant et al. [7-2] is based on the solidification theory for concrete creep [7-3]. It represents the third version of the solidification models and was refined from previous versions to improve accuracy and usefulness. Previous attempts to model concrete as a viscoelastic material had difficulty with the aging process which involves the dependence of material properties on time. Solidification theory states that at the microscale, solidifying cement hydration products are non-aging and linear viscoelastic and as hydration continues, the volume fraction of load bearing matter increases, as shown in Figure 7-2.
Figure 7-2. Solidification model for concrete creep

In the B3 model, total strain is calculated according to

\[ \varepsilon(t) = J(t,t')\sigma + \varepsilon_{sh}(t) + c\Delta T(t) \]  \hspace{1cm} (7.3)

where \( J(t,t') \) is the compliance function, \( t \) is the age of concrete, and \( t' \) is the age at loading. \( J(t,t') \) can be subdivided further into

\[ J(t,t') = q_1 + C_o(t,t') + C_d(t,t') \]  \hspace{1cm} (7.4)

where \( q_1 \) is the instantaneous compliance, \( C_o(t,t') \) is the basic creep component, and \( C_d(t,t') \) is the drying creep component. \( C_o(t,t') \) and \( C_d(t,t') \) are given by

\[ C_o(t,t') = q_2 Q(t,t') + q_3 \ln[1 + (t-t')^\alpha] + q_4 \ln(t/t') \]  \hspace{1cm} (7.5)

\[ C_d(t,t') = q_5 \{ \exp\{ -8H(t) \} - \exp\{ -8H(t') \} \}^{1/2} \]  \hspace{1cm} (7.6)

The parameters \( q_1 \) through \( q_5 \) are material dependent constants, \( \tau_{sh} \) is the shrinkage half-time, and \( H(t) \) represents the average relative humidity of a cross section as a function of time, given by
A detailed explanation of the model, including the definitions and equations for constants is given in reference [7-2].

### 7.2 Model Improvement and Updating

The B3 model is attractive for describing early age tensile creep because is it based on a fundamental approach for aging of concrete. However, application of the basic creep portion of this model to experimental results by Østergaard et al. has shown that the B3 model in its native form does not give accurate prediction at loading ages of one day or less [7-4]. To account for this discrepancy, an additional parameter was proposed in his work to capture very early creep. The additional term was incorporated into parameter $q_2$ according to

$$q_2 = q_2 \left( \frac{t'}{t' - q_6} \right).$$  \hspace{1cm} (7.8)

[Note: The new coefficient was called $q_5$ in the original reference but has been renamed $q_6$ here to avoid confusion with the drying creep parameter $q_3$. Østergaard demonstrated the improvement of fit in his study, where he considered wet-cured samples only, and altered the basic creep component of the prediction model to produce successful results.

The ACI equations were applied to a concrete mixture with a $w/cm$ of 0.50. A comparison between creep strain measurements from a constant load test and the ACI model prediction is shown in Figure 7-1 using two different values for $v_u$, the ultimate creep coefficient. The lower curve reflects the ACI recommended constants modified for test conditions, and the other uses an ultimate creep coefficient $v_u$ of 13.5, which is beyond the recommended range of the parameter. The prediction fits the experimental data quite well – demonstrating that even early age creep can be modeled with the ACI equation – but only after the $v_u$ parameter has been modified beyond a realistic range. This finding confirms the limitation on the ACI model of a loading age of 7 days, which is reasonable for structural loads. For earlier loading ages from deformation...}
due to drying and autogenous shrinkage or temperature change, modifications of some kind are necessary to apply this prediction.

The following modifications are suggested, in the form of additional creep and shrinkage correction factors for early age. For creep, the modification \( K_{c EA} \) is given by

\[
K_{c EA} = \left[ 1 + \frac{r}{t'} \left( \frac{1}{t - s} \right) \right],
\]

where \( t \) is time, \( t' \) is the loading time, and \( s \) is the setting time, in days. The parameter \( r \) was obtained from fitting experimental data and was determined to be 1.4 for normal concrete. The parameter depends on the rate of early strength gain and should be reduced for high early strength concrete. The values of this correction factor equation are shown over time and for different loading ages in Figure 7-3. It can be observed that the function approaches one as the loading time exceeds 7 days, thereby reducing the model to its original form.

A shrinkage correction factor for early age was also determined and is given by the equation

\[
K_{z EA} = \left[ 1 + \frac{z}{t_o} \left( \frac{1}{t - s} \right) \right],
\]

\[ (7.10) \]

where \( t \) is time, \( t_o \) is the length of curing, and \( s \) is the setting time, all given in days. The parameter \( z \) was obtained from fitting experimental data and was determined to be 5.6 for normal concrete. The parameter depends on the diffusion rate, which is dependent on the degree of hydration and should probably be reduced for high early strength or steam cured concrete. This equation approaches one as the curing time exceeds 7 days, reducing the model to its original form, as shown in Figure 7-4.

The shrinkage constant \( f_c \) in the original ACI model (equation 7.2) needs to be reduced for early age concrete. It should be noted that this constant is not the design compressive strength of the concrete, but it may have a correlation to the strength divided by 100. The recommended values according to ACI are 35 for normal concrete and 55 for steam cured concrete, suggesting that the
parameter is dependent on degree of hydration or rate of strength gain. For early age, this parameter should be adjusted to account for young concrete. In this study, a value of 25 was used for normal strength concrete at early age. It may be important to correlate this parameter more directly to compressive strength in future version of the model.

The modified B3 (MB3) and modified ACI (MACI) models were used to predict creep and shrinkage for concrete mixtures with 0.40, 0.44 and 0.50 w/cm ratios under restrained drying conditions. The elastic modulus for this material was approximated using the ACI equation [7-5]. The model predictions for shrinkage at early age closely fit the experimental data after the proposed modifications are made, as shown in Figure 7-5 through Figure 7-7. No modifications were needed for the original B3 model to account for early age drying or autogenous shrinkage in this study. However, it is reasonable to be cautious about applicability of the model to materials with lower w/cm ratio beyond the range of the study. Lower w/cm ratio materials with high autogenous shrinkage were not considered in the current study. The model predicted shrinkage decreases in proportion to the w/cm ratio, as it is known that the drying rate decreases with diffusion rate for smaller pores. Autogenous shrinkage at early age will increase for lower w/cm ratios in the first few days. Figure 5-102 shows the effect of autogenous shrinkage at early age as w/cm decreases. It can be seen that the early shrinkage of the 0.25 w/cm material is 5 times greater than the 0.50 material in the first hours of drying. The timing of early age shrinkage measurements is important because after one week the low w/cm ratio materials are no longer shrinking rapidly. The current model does not account for cases where autogenous shrinkage dominates behavior.

The model predictions of tensile creep agree with experimental data, as shown in Figure 7-9 through Figure 7-11. The original models without modifications are also shown for comparison. The MB3 model incorporates equations from [7-4] and early age parameter values used by Østergaard et al. No additional terms were used to account for drying creep at early age. After several days of drying, the restrained stress will exceed 40% of the material strength, causing microcracking damage to occur. After 4-6 days, the creep prediction diverges from experimental data and the assumption of a linear relationship between creep and stress is not valid in this region. To further understand the amount of damage that occurs in fully restrained drying conditions, the stress strain relationships were examined at each load increment during the test.
It can be seen in Figure 7-12 that damage is observed as the stress-strain curve begins to exhibit softening in the higher stress regions. A comparison of the stress-strain ratio with the elastic Young’s modulus $E$ predicted by ACI equations is shown in Figure 7-13. The ACI equations were intended for mature concrete in compression, and are shown to demonstrate the evolving nature of the stress-strain relationship. The measurement of stress-strain behavior from this experiment is not equivalent to the elastic modulus, since the stress increments are not equal for each step and concrete at early age does not typically behave as a linear elastic solid. However, the comparison does reveal both the influence of aging (i.e. increase in stiffness) and softening (i.e. microcracking).

Creep behavior under restrained conditions was analyzed using an incremental application of each model where the stress was increased at each load step. Figure 7-14 through Figure 7-16 show the model results and experimental data for the 0.40, 0.44, and 0.50 mixtures respectively. The results show good agreement for restrained creep deformation, indicating that after the proposed modifications, the models can be applied incrementally for structural predictions, and that they are valid for early age tensile creep in addition to mature compressive creep. The modified B3 and ACI models were applied to SCC, as shown in Figure 7-17 and Figure 7-18. It should be noted that ACI model contains a parameter based on slump, and whenever water reducer or superplasticizer are used, the input into the model should be the slump before admixture additions. The effect of admixtures is not treated by the models, but the change in paste and water contents in the mixture will be judged based on slump, which was the case before admixture usage was prevalent. In general, predictions do not match well for very early measurements, however the overall prediction can be considered adequate for structural calculations. The modified B3 model was applied to SRAC as shown in Figure 7-19 and it was able to predict restrained tensile creep only if the difference in loading age was accounted for in the model prediction. Early expansion occurs when SRA is used, and the result on tensile stress development is to delay the onset of shrinkage stress. The resulting creep occurs at slightly later age and is relatively lower in magnitude due to aging of the viscoelastic properties.

7.3 Practical Modeling Recommendations

Constant load and incremental restrained load cases for measurement and modeling of early age tensile creep and shrinkage of concrete were considered. An experimental program measured
early age tensile creep and shrinkage and the results were used to develop suggestions for improving existing models. The experimental results were compared to the ACI 209 and B3 prediction models. The following conclusions were drawn:

- Suggested changes to the ACI 209 model enable the prediction of early age creep and shrinkage. The changes are in the form of correction factors that can be employed in the same manner as other factors that are already in the model.

- The B3 model predicted early age shrinkage with reasonable accuracy without any changes in its original formulation. However, to account for autogenous shrinkage, lower w/cm ratios should be investigated at early age.

- The B3 model, modified by Østergaard for basic creep, was successfully used for early age tensile creep under both constant load and restrained drying conditions. No further modifications were made to account for drying creep at early age.

- A stepwise application of the modified B3 and ACI models effectively predicted creep strain under restrained drying conditions at early age. It is an approximation that neglects some of the material aging that occurs, but the results suggest that for structural calculations, the assumptions are acceptable.
Figure 7-3. Creep correction factor values versus time for different loading times at early age

Figure 7-4. Shrinkage correction factor values for different curing times at early age
Figure 7-5. Shrinkage measurements compared with ACI and B3 models for $w/cm = 0.40$

Figure 7-6. Shrinkage measurements compared with ACI and B3 models for $w/cm = 0.44$
Figure 7-7. Shrinkage measurements compared with ACI and B3 models for w/cm =0.50

Figure 7-8. The effect of autogenous shrinkage at low w/cm ratio on early age shrinkage rate
Figure 7-9. Modified ACI and B3 model prediction for tensile specific creep, w/cm = 0.40

Figure 7-10. Modified ACI and B3 model prediction for tensile specific creep, w/cm = 0.44
Figure 7-11. Modified ACI and B3 model prediction for tensile specific creep, w/cm = 0.50

Figure 7-12. Stress-strain relationship during the restrained drying test
Figure 7-13. Stress-strain ratio over time under restrained drying conditions

Figure 7-14. Modified ACI and B3 model prediction for early age tensile creep strain under fully restrained conditions, w/cm = 0.40
Figure 7-15. Modified ACI and B3 model prediction for early age tensile creep strain under fully restrained conditions, \( \frac{w}{c}m = 0.44 \)

Figure 7-16. Modified ACI and B3 model prediction for early age tensile creep strain under fully restrained conditions, \( \frac{w}{c}m = 0.50 \)
Figure 7-17. Modified B3 predictions for restrained self-consolidating concrete at early age

Figure 7-18. Modified B3 (MB3) and modified ACI (MACI) models applied to SCC materials with correction made to slump requirements for the ACI model
7.4 Tools for Predicting Stress Development

7.4.1 Calibrated Model for Internal RH prediction

Parrot’s model was used to develop mathematical relationships for the drying gradient that were then used as input for the three dimensional model and for graphical representation of the drying profiles shown in Figure 7-20. The humidity at any depth $d$ and time $t$ is given by

$$h = h_a + \frac{(a - h_a)}{a + bt / d^n}$$  \hspace{1cm} (7.11)

where

- $t$ - time
- $d$ - distance from surface
- $h_a$ - ambient humidity
- $a, b, n$ - fitting parameters
7.4.2 Using shrinkage models to identify practical limits for paste content

Hansen’s model was identified as a potential tool to make adjustments for paste content within a given concrete mixture. The application of this tool is illustrated when considering the modeling of segregated concrete. The model application in this case consists of known properties of the cement paste and the design concrete mixture. Using the model we can approximately determine how much additional shrinkage deformation would be expected from the increase in paste content as segregation occurs along the length of the wall, as shown in Figure 6-14.

The model relationships are given by

\[
\frac{\varepsilon_c}{\varepsilon_p} = 0.5 (1 - V_a) \left( \frac{1}{1-V_a + km V_a} + (1 - V_a) + \frac{V_a}{km} \right) \quad (7.12)
\]

\[
k = \frac{(1 - 2 V_p)}{(1 - 2 V_a)} \quad (7.13)
\]

and

\[
m = \frac{E_u}{E_p} \quad (7.14)
\]

where

\( \varepsilon_c \) – concrete shrinkage
7.4.3 Using rheological model chains

A MATLAB coupled rheological model for stress prediction was developed based on the equations presented in Chapter 6 for solving one-dimensional RH-induced stress problems. The source code for this program is given in Appendix E. A screenshot of the program is shown in Figure 7-21.
Figure 7-21. Screenshot of MATLAB program for solving RH induced stress creep-shrinkage problems

7.5 Proportioning for Minimum Shrinkage Stress

Limits for proportioning can be suggested based on this body of experimental work. The practical use of new concrete materials requires an understanding of proportioning limits to obtain properties such as strength in the case of HPC, or flowability in the case of SCC, while minimizing the impact on other material properties, such as shrinkage and creep. The degree of importance should be weighted according to the application. For example, in flatwork such as concrete slabs on grade or bridge decks, the minimization of shrinkage is critical to limit cracking, and the importance of creep as a shrinkage relaxation mechanism should be recognized. Proportioning strategies should reduce shrinkage as much as possible without reducing creep. However, when long term compressive loads are significant, such as in prestressed concrete or tall columns, both shrinkage and creep are unfavorable, since they lead loss of prestressing force or serviceability problems in structures.
The proportioning parameters that are most influential ($w/cm$ and paste content) should be scrutinized closely with regard to limitations that should be imposed for certain applications. A minimum $w/cm$ ratio should be imposed in many cases to limit autogenous shrinkage. The experimental and modeling work was used to identify the correct limits and to establish guidelines for optimizing proportions to achieve the necessary properties (strength, flowability) without sacrificing long term durability. It is possible to develop low shrinkage concrete mixtures using these guidelines.

The possibility of extrapolating design parameters for shrinkage and creep out of one single free shrinkage test was explored as an intriguing way to simplify prediction methods. Shrinkage is in fact viscoelastic behavior, although it is a response to internal stress. Therefore, it could be possible to relate free shrinkage behavior to creep in a simplified manner if the proper analysis is applied to one experiment.

7.5.1 Minimization of Cement Paste Content

In SCC, the desire to achieve flowable concrete can lead a designer to increase the cement paste content. In HPC, the desire to obtain higher strength or lower permeability drives the requirement for higher paste content in order to maintain workability. When the available constituent materials are not suitable for making SCC or HPC, such poorly graded aggregates, the tendency is to correct the mixture with higher cement paste. If possible, any increase in cement content over the recommended range should be prohibited, as the impact on mechanical properties will be dramatic. Increasing the cement paste content outside of the normal design range will cause in the elastic modulus, creep, and shrinkage properties, as shown in . It has been shown through research worldwide, as well as through local implementation, that it is possible to achieve the necessary flowability in SCC without a dramatic increase in cement paste content. The blending of multiple coarse aggregates has been shown to be especially effective at producing cohesive SCC that resists segregation. The top size of the aggregate should be selected such that the passing ability is appropriate for a given structure, based on ACI 318 recommendations. The remainder of the aggregate should be uniformly graded to minimize the necessary paste volume required to fill the void volume. Specifications may be proposed that resemble Figure 7-23, where the size fractions should be held within an acceptable range for the production of SCC. These limits can be achieved with most aggregate materials available in
practice. If multiple aggregate sources are available, tighter limits may be imposed, such as the “8-18” criterion, which refers to the percent retained curve falling between 8% and 18%. Based on this study, limitations of cement paste content for minimization of risk associated with shrinkage stresses should be categorized according to the level of structural restraint according to Table 7-1. It is important to note that these limits are based on restrained shrinkage stresses only. In the case of mass concrete, these limits will not prevent excessive thermal stresses or cracking, and it is assumed that other means will be provided to reduce in-place temperatures, such as cooling pipes.

Figure 7-22. Effect of cement paste content on shrinkage
Figure 7-23. Aggregate gradation suggestions based on uniformity

Table 7-1. Maximum cement paste content specification

<table>
<thead>
<tr>
<th>Category</th>
<th>Examples</th>
<th>Cement Paste Limit by Volume</th>
<th>Approximate cement content at 0.44 w/c ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>High Restraint</td>
<td>Bridge decks, pavement, slabs, high volume-to-surface area</td>
<td>30%</td>
<td>605 lb/yd³</td>
</tr>
<tr>
<td>Current IDOT Limit, Medium Restraint</td>
<td>Beams, pipe, precast, substructures, mass concrete</td>
<td>34%</td>
<td>705 lb/yd³</td>
</tr>
<tr>
<td>Low Restraint</td>
<td>only cases where restraint is minimal and creep and shrinkage are not a concern</td>
<td>36%</td>
<td>750 lb/yd³</td>
</tr>
</tbody>
</table>

7.5.2 Aggregate Optimization

Recent research has shown that aggregate optimization can effectively reduce paste content without a degradation of concrete properties. Proportions are based on minimizing the volume of voids in a given mixture based on the particle size distribution. The proportions are determined based on optimum packing models [7-1, 7-8, 7-9]. The resulting mixture requires superplasticizer to achieve the same workability as OPC without adding additional water, however this strategy is expected to reduce overall cost and result in a more workable concrete.
mixture when a superplasticizing admixture is added. Other benefits may include better finishability, faster finishing times, and reduced bleeding than with OPC. Figure 7-24 shows that a reduction in shrinkage is possible through the use of an optimized mixture when compared to OPC at the same w/cm ratio. The optimized mixture also attained similar strength to the OPC mixture despite having lower paste content. It is a common misunderstanding that using more portland cement in concrete can generate higher strength. This apparent relationship is a holdover from the days before superplasticizer existed and is derived from the need for more water to achieve the same level of workability in a concrete mixture with less cement. The perceived loss of strength was caused by adding water, not removing cement. It is well understood that the dominant factor determining strength is the w/cm ratio, which controls the porosity of cement paste. Therefore if optimized concrete is produced without additional water, but by using superplasticizer to achieve the same workability as OPC, the results indicate that optimization strategies will not have a negative impact on strength.

![Figure 7-24. Effect of aggregate optimization on shrinkage](image-url)

**Figure 7-24. Effect of aggregate optimization on shrinkage**
7.5.3 Optimization of w/cm ratio for minimum shrinkage

A clear result of this study is that autogenous shrinkage may cause significant stress at early age. SCC strategies may include a low w/cm ratio to improve cohesiveness of the mixture and avoid segregation. Unfortunately, this may cause an increase in shrinkage as shown in Figure 7-25 and as a result, many low w/cm ratio systems are highly susceptible to early age cracking. This is opposite to the common relationship between drying shrinkage and w/cm ratio. Drying shrinkage is driven by external drying and controlled by the diffusion characteristics of the material, which decrease with concrete porosity. Autogenous shrinkage is typically negligible at a w/cm ratio of 0.42. Below that, it increases and eventually surpasses drying shrinkage at early age. It is recommended that specifications be written that impose a minimum w/cm ratio to avoid significant autogenous shrinkage and early age cracking. The limits would also be based on restraint conditions as with paste content, as shown in Table 7-2. It is important to note that these limits are based on restrained shrinkage stresses only. In the case of mass concrete, these limits will not prevent excessive thermal stresses or cracking, and it is assumed that other means will be provided to reduce in-place temperatures, such as cooling pipes.

![Figure 7-25. At early age, shrinkage increases as w/cm ratio decreases](image-url)
Table 7-2. Recommended limitations for w/cm ratio to avoid early age cracking

<table>
<thead>
<tr>
<th>Category</th>
<th>Examples</th>
<th>Minimum w/cm Ratio</th>
<th>Approximate autogenous shrinkage strain at 28d (με)</th>
</tr>
</thead>
<tbody>
<tr>
<td>High Restraint</td>
<td>Bridge decks, pavement, slabs, high volume-to-surface area</td>
<td>0.42</td>
<td>0</td>
</tr>
<tr>
<td>Current IDOT Limit, Medium Restraint</td>
<td>Beams, pipe, precast, substructures, mass concrete only cases where restraint is minimal and creep and shrinkage are not a concern</td>
<td>0.38</td>
<td>80</td>
</tr>
<tr>
<td>Low Restraint</td>
<td></td>
<td>0.32</td>
<td>200</td>
</tr>
</tbody>
</table>

7.5.4 Other mitigation strategies

If it is not possible to abide by limitations shown in Tables 5 and 6, then it is recommended based on this body of research, that shrinkage mitigation methods be employed. Mitigation methods based on SRA have been shown in this study to be effective at reducing the driving force for shrinkage and reducing the probability for cracking. Another mitigation strategy that should be considered is the use of saturated lightweight aggregates, which have been shown to be particularly effective for mitigation of both drying and autogenous shrinkage [7-10, 7-11, 7-12]. Although beyond the scope of this work, there are guidelines available in the literature for selection of lightweight materials for this purpose. Potential limitations that have been reported are the pumpability and stability of the entrained air void system, similar to traditional lightweight concretes, although substantially less lightweight material should be needed to provide shrinkage mitigation and internal curing enhancement than for lightweight concrete.

7.6 References


8. CONCLUSIONS

8.1 Summary

The goal of this study was to improve the general understanding of concrete mechanical behavior at early ages, particularly in new materials such as SCC and HPC and SRC. Practitioners designing HPC and SCC, as part of their strategies to achieve certain properties such as strength, rapid strength gain, or flowability, may inadvertently reduce long term durability by increasing the risk of cracking at early ages. The development of shrinkage stress that causes cracking was observed to correlate primarily with the cementitious paste content and the w/cm ratio. The behavior of concrete with high paste content and low w/cm fall outside the boundary of most prediction models used in practice, which may in turn lead to improper structural design and long term durability problems. SRA as a mitigation strategy has the potential to reduce shrinkage stress and reduce cracking tendency.

This study has yielded new insight into early stress development. The experiments and analytical work have also established guidelines for implementation of new material models that are particularly useful for practice. Finally, a primary goal of this study was to provide acceptance criteria and guidance for the development of emerging concrete materials that are resistant to early shrinkage cracking.

The primary outcomes of this study are as follows:

- Characterized the early age mechanical properties for emerging types of concrete materials: SCC, HPC, and SRC. Demonstrated that modifications may be needed to model prediction equations for creep, shrinkage, strength, and modulus.

- Demonstrated that concrete containing supplementary cementitious materials such as fly ash and slag, or inert filler such as limestone powder develops lower shrinkage stress at early age. Results indicate that specifications should encourage SCMs when shrinkage mitigation is needed. Silica fume increased early shrinkage stress development and therefore should be used cautiously when early shrinkage cracking is a concern. Other mitigation strategies such as SRA or saturated lightweight aggregate could be used to offset the early shrinkage stress caused by silica fume.
• Demonstrated that SRAs are effective both at reducing free shrinkage and shrinkage stress development, and a significant reduction in shrinkage cracking can be obtained.

• Verified that SCC mixture proportioning strategies typically produce concrete with high cement paste content and relatively low w/cm, which can be detrimental to long term durability if restrained shrinkage cracking is not mitigated. Careful proportioning using aggregate optimization to obtain a uniform gradation and use of SCMs is helpful for reducing the cracking potential in SCC. Maximum cementitious paste contents and minimum w/cm ratios should be implemented for shrinkage or creep critical structures. Suggestions were made to guide practitioners in establishing these limits.

• Confirmed that autogenous shrinkage of both HPC and SCC must be mitigated in highly restrained structures to prevent shrinkage cracking. Autogenous shrinkage is typically measurable below w/cm of 0.42 but generally is not problematic unless the w/cm is below 0.36. Typical project specifications do not include testing for autogenous shrinkage and an ASTM standard test method for concrete is not yet available. The test method developed during this study provides a simple yet effective procedure to measure autogenous shrinkage in the laboratory or in the field. The method can be coupled with RH sensors to develop calibration inputs for the model presented as part of this research.

• Suggested restrained ring shrinkage testing (ASTM C 1581 or AASHTO T334) be utilized in specifications to provide performance criteria for cracking. This is becoming increasingly common in practice. Though not a focus of this study, the ring test is a simple yet effective tool for specification purposes. However, careful interpretation may be needed to decide what level of cracking in the ring test should correlate to good field performance for different levels of mechanical restraint.

• Examined the relationship of early age concrete mechanical behavior in tension vs. compression. Results indicate up to a 10 fold increase in tensile creep when measured at the same age, applied stress ratio, and environmental conditions. This difference is mainly attributed to the drying stress gradient, which results in tensile stresses at the outer surface that cause microcracking. Surface tensile stresses reached a maximum around 3 days according to model results.

• Established the role of tensile creep in relaxing shrinkage stresses of SCC, HPC, and SRC at early age. Higher tensile creep capacity translates to more durable, crack resistant concrete, as demonstrated by restrained cracking test results. It is therefore considered imperative that
a tensile creep test be promoted to ASTM based on the dead load tensile apparatus used in this study.

- Observed non-linearity of creep at early age and under variable stress, leading to new insights for superposition in drying creep analysis. Drying creep in concrete is creep non-linearity due to stress gradients that cause additional creep in higher stress regions. Drying creep is an artifact of predicting total creep using superposition of drying shrinkage and basic creep.

- Provided guidelines and modeling tools for the prediction of autogenous shrinkage and its role in creep analysis in low w/cm systems. Developed a new experimental technique for characterization of autogenous shrinkage in concrete.

- Established a fundamentally based, intuitive, predictive creep and shrinkage modeling approach at early age for SCC, HPC, and SRC.

- Developed a new experiment to characterize differential drying using a curling beam technique. The beam was used to validate drying shrinkage stress predictions, but could have other applications for floor slab concrete mixtures or floor covering evaluations to examine curling.

- Recommended guidelines for proportioning for minimum shrinkage stress and acceptance criteria for practice.

**8.2 Future Extensions**

Additional studies would make logical extensions of this research. The investigation of curing effectiveness and the role of curing compounds and sealers for mitigation of cracking, prevention of drying stress gradients, and promoting continued hydration may be studied through the utilization of RH sensors and the RSTM. Continuous moist curing of concrete structure is an expensive practice for contractors. It has often been debated in recent years that the requirement for 7 to 14 days of continuous moist curing is ineffective for HPC due to low porosity and relatively rapid strength gain. It would be valuable to the industry to verify curing effectiveness for HPC, or modify suggested practice, such as curing compound.

Internal curing shows promise as a mitigation strategy in concrete and the modeling approach established in this research, based on internal RH, should work well for concrete with internal curing agents, since their role is to keep the RH high, and this can be measured with sensors. The
additional benefit of saturated lightweight aggregates for internal curing is the reduction of the elastic modulus, which reduces restrained shrinkage stress.

Sensor technology continues to improve and the availability of nanoscale RH sensors in the near future is possible. Applications for nano or molecular scale sensors in concrete are numerous, and it could be envisioned that a network of nano-sensors the size of cement grains could be distributed in fresh concrete as a powder, allowing for full-field measurement of RH and wireless transmission to a datalogging system. If such a system were available, field implementation of the techniques described in this research for modeling shrinkage could be drastically simplified. Furthermore, a smaller RH sensor would be expected to produce more accurate results, due to less anticipated volumetric interference of the sensor itself with the drying process.

This research establishes the role of tensile creep for relaxation of shrinkage stress in new materials that are outside the scope of many current code models and design guidelines. The experimental work to characterize early age behavior was used to develop a fundamentally sound, intuitive modeling approach utilizing relative humidity measurements for prediction of drying shrinkage stress. Guidelines for proportioning and acceptance criteria were proposed that are intended to help practitioners utilize innovative concrete materials while minimizing cracking risk and its negative impact on long term durability.
## APPENDIX A - Cementitious Materials Characterization

### Table A-I. Chemical Composition of Cementitious Materials

<table>
<thead>
<tr>
<th>Cement</th>
<th>Essroc</th>
<th>Saylor's Type I</th>
<th>Macon Ashgrove Type III</th>
<th>Highland</th>
<th>Duncan Rd</th>
<th>Kaskaskia</th>
<th>Big Creek L.</th>
<th>Springfield Lab 1</th>
<th>Lab 2</th>
<th>Peoria</th>
<th>Peoria (dup)</th>
<th>RILEM</th>
<th>Norcem Strong Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO₂</td>
<td>22.01</td>
<td>21.30</td>
<td>22.48</td>
<td>19.15</td>
<td>19.80</td>
<td>19.83</td>
<td>20.70</td>
<td>20.93</td>
<td>20.43</td>
<td>20.11</td>
<td>20.98</td>
<td>22.03</td>
<td></td>
</tr>
<tr>
<td>Al₂O₃</td>
<td>4.60</td>
<td>4.72</td>
<td>4.32</td>
<td>4.62</td>
<td>4.50</td>
<td>4.26</td>
<td>5.01</td>
<td>4.22</td>
<td>5.02</td>
<td>6.16</td>
<td>5.22</td>
<td>5.08</td>
<td></td>
</tr>
<tr>
<td>Fe₂O₃</td>
<td>1.64</td>
<td>2.30</td>
<td>1.94</td>
<td>2.47</td>
<td>1.47</td>
<td>2.53</td>
<td>1.98</td>
<td>2.31</td>
<td>1.89</td>
<td>1.89</td>
<td>2.41</td>
<td>2.37</td>
<td></td>
</tr>
<tr>
<td>CaO</td>
<td>62.59</td>
<td>61.83</td>
<td>61.67</td>
<td>61.64</td>
<td>59.01</td>
<td>60.68</td>
<td>63.29</td>
<td>65.86</td>
<td>63.28</td>
<td>63.73</td>
<td>63.54</td>
<td>65.55</td>
<td></td>
</tr>
<tr>
<td>MgO</td>
<td>2.45</td>
<td>2.66</td>
<td>1.95</td>
<td>2.87</td>
<td>3.28</td>
<td>2.46</td>
<td>1.72</td>
<td>2.44</td>
<td>2.15</td>
<td>3.63</td>
<td>3.59</td>
<td>1.65</td>
<td></td>
</tr>
<tr>
<td>K₂O</td>
<td>0.07</td>
<td>0.62</td>
<td>0.56</td>
<td>0.62</td>
<td>1.31</td>
<td>0.72</td>
<td>0.47</td>
<td>0.16</td>
<td>0.76</td>
<td>0.82</td>
<td>0.79</td>
<td>1.08</td>
<td></td>
</tr>
<tr>
<td>Na₂O</td>
<td>0.05</td>
<td>0.14</td>
<td>0.09</td>
<td>0.14</td>
<td>0.14</td>
<td>0.09</td>
<td>0.09</td>
<td>0.19</td>
<td>0.05</td>
<td>0.22</td>
<td>0.22</td>
<td>0.28</td>
<td></td>
</tr>
<tr>
<td>Total Alkali (Na eq)</td>
<td>0.10</td>
<td>0.56</td>
<td>0.47</td>
<td>0.57</td>
<td>1.04</td>
<td>0.59</td>
<td>0.52</td>
<td>0.16</td>
<td>0.59</td>
<td>0.78</td>
<td>0.76</td>
<td>1.02</td>
<td></td>
</tr>
<tr>
<td>TiO₂</td>
<td>0.31</td>
<td>0.17</td>
<td>0.16</td>
<td>0.18</td>
<td>0.24</td>
<td>0.22</td>
<td>0.17</td>
<td>0.30</td>
<td>0.25</td>
<td>0.29</td>
<td>0.24</td>
<td>0.23</td>
<td></td>
</tr>
<tr>
<td>P₂O₅</td>
<td>0.10</td>
<td>0.10</td>
<td>0.10</td>
<td>0.07</td>
<td>0.08</td>
<td>0.10</td>
<td>0.24</td>
<td>0.08</td>
<td>0.20</td>
<td>0.16</td>
<td>0.17</td>
<td>0.07</td>
<td></td>
</tr>
<tr>
<td>Sr(ppm)</td>
<td>414</td>
<td>487</td>
<td>515</td>
<td>461</td>
<td>336</td>
<td>187</td>
<td>366</td>
<td>513</td>
<td>188</td>
<td>150</td>
<td>248</td>
<td>335</td>
<td></td>
</tr>
<tr>
<td>Ba (ppm)</td>
<td>207</td>
<td>349</td>
<td>100</td>
<td>215</td>
<td>232</td>
<td>460</td>
<td>403</td>
<td>734</td>
<td>747</td>
<td>611</td>
<td>661</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Zr(ppm)</td>
<td>81</td>
<td>74</td>
<td>90</td>
<td>77</td>
<td>56</td>
<td>84</td>
<td>87</td>
<td>84</td>
<td>124</td>
<td>124</td>
<td>94</td>
<td></td>
<td></td>
</tr>
<tr>
<td>LOI (1000°C)*</td>
<td>1.02</td>
<td>0.68</td>
<td>1.27</td>
<td>0.63</td>
<td>1.53</td>
<td>1.31</td>
<td>1.42</td>
<td>0.18</td>
<td>1.02</td>
<td>1.79</td>
<td>1.29</td>
<td>1.28</td>
<td>2.43</td>
</tr>
<tr>
<td>H₂O (110°C)</td>
<td>0.75</td>
<td>0.25</td>
<td>0.69</td>
<td>0.37</td>
<td>0.51</td>
<td>0.66</td>
<td>0.50</td>
<td>0.13</td>
<td>0.50</td>
<td>0.51</td>
<td>0.75</td>
<td>0.76</td>
<td>0.41</td>
</tr>
<tr>
<td>TOTAL</td>
<td>94.4</td>
<td>99.8</td>
<td>98.9</td>
<td>99.4</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>99.0</td>
<td>98.2</td>
<td>98.7</td>
<td>98.8</td>
<td>97.6</td>
<td>99.0</td>
</tr>
</tbody>
</table>

### Bogue Calculation

| A/F | 2.81 | 2.06 | 2.22 | 1.87 | 1.94 | 2.90 | 3.15 | 1.67 | 2.94 | 2.17 | 3.26 | 3.25 | 2.17 | 2.14 |
| C₂S | 53.08 | 39.91 | 34.14 | 61.46 | 55.84 | 64.35 | 50.22 | 67.32 | 67.67 | 68.77 | 56.23 | 60.76 | 59.21 | 60.46 |
| C₃S | 23.07 | 30.95 | 38.70 | 8.54 | 10.23 | 8.73 | 18.87 | 6.08 | 8.30 | 8.13 | 16.16 | 11.82 | 15.49 | 17.56 |
| C₄AF | 4.98 | 6.99 | 5.91 | 7.51 | 7.08 | 4.48 | 4.84 | 7.69 | 6.03 | 7.04 | 5.75 | 5.74 | 7.32 | 7.21 |
### Table A-2. Chemical Composition of Fly Ash

<table>
<thead>
<tr>
<th>Flyash</th>
<th>Min Solns</th>
<th>Macon</th>
<th>Lab Class</th>
<th>Duncan</th>
<th>Duncan (rep)</th>
<th>Kaskaskia</th>
<th>L. Springfield</th>
<th>Paiewa flyash</th>
<th>Highand flyash</th>
<th>MBT Proash</th>
<th>Wali FA</th>
<th>Min Solns</th>
<th>Lafarge</th>
<th>Peoria Class C</th>
<th>Peoria Class C</th>
<th>MBT Proash</th>
</tr>
</thead>
<tbody>
<tr>
<td>Date</td>
<td>Date</td>
<td>Date</td>
<td>Date</td>
<td>Date</td>
<td>Date</td>
<td>Date</td>
<td>Date</td>
<td>Date</td>
<td>Date</td>
<td>Date</td>
<td>Date</td>
<td>Date</td>
<td>Date</td>
<td>Date</td>
<td>Date</td>
<td>Date</td>
</tr>
<tr>
<td>SiO2</td>
<td>32.29</td>
<td>33.04</td>
<td>49.10</td>
<td>36.63</td>
<td>36.91</td>
<td>32.89</td>
<td>33.14</td>
<td>31.00</td>
<td>33.08</td>
<td>30.39</td>
<td>34.57</td>
<td>25.69</td>
<td>36.50</td>
<td>57.28</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fe2O3</td>
<td>5.11</td>
<td>4.41</td>
<td>12.51</td>
<td>5.22</td>
<td>5.14</td>
<td>4.59</td>
<td>4.99</td>
<td>5.10</td>
<td>5.23</td>
<td>5.07</td>
<td>3.13</td>
<td>5.17</td>
<td>6.59</td>
<td>5.25</td>
<td>2.86</td>
<td>5.39</td>
</tr>
<tr>
<td>CaO</td>
<td>28.59</td>
<td>31.84</td>
<td>8.47</td>
<td>22.44</td>
<td>22.48</td>
<td>27.72</td>
<td>27.82</td>
<td>26.09</td>
<td>27.67</td>
<td>26.12</td>
<td>0.86</td>
<td>23.02</td>
<td>28.38</td>
<td>29.38</td>
<td>51.04</td>
<td>25.63</td>
</tr>
<tr>
<td>MgO</td>
<td>6.24</td>
<td>5.59</td>
<td>1.81</td>
<td>4.06</td>
<td>4.13</td>
<td>5.42</td>
<td>5.33</td>
<td>5.53</td>
<td>5.28</td>
<td>5.06</td>
<td>1.06</td>
<td>5.23</td>
<td>4.65</td>
<td>5.43</td>
<td>4.12</td>
<td>5.21</td>
</tr>
<tr>
<td>K2O</td>
<td>0.40</td>
<td>0.52</td>
<td>2.19</td>
<td>0.60</td>
<td>0.62</td>
<td>0.56</td>
<td>0.56</td>
<td>0.50</td>
<td>0.45</td>
<td>0.49</td>
<td>0.22</td>
<td>0.80</td>
<td>0.44</td>
<td>0.47</td>
<td>0.72</td>
<td>0.50</td>
</tr>
<tr>
<td>Na2O</td>
<td>2.89</td>
<td>1.73</td>
<td>1.23</td>
<td>1.50</td>
<td>1.59</td>
<td>1.55</td>
<td>1.52</td>
<td>1.80</td>
<td>1.76</td>
<td>1.26</td>
<td>0.26</td>
<td>1.36</td>
<td>1.25</td>
<td>1.25</td>
<td>0.53</td>
<td>1.21</td>
</tr>
<tr>
<td>Total Alkali (Na eq)</td>
<td>3.16</td>
<td>2.09</td>
<td>1.91</td>
<td>2.01</td>
<td>1.94</td>
<td>1.90</td>
<td>2.14</td>
<td>2.07</td>
<td>2.07</td>
<td>1.78</td>
<td>2.00</td>
<td>3.57</td>
<td>2.08</td>
<td>2.10</td>
<td>1.20</td>
<td>2.00</td>
</tr>
<tr>
<td>TiO2</td>
<td>1.25</td>
<td>1.00</td>
<td>0.96</td>
<td>1.45</td>
<td>1.47</td>
<td>1.23</td>
<td>1.22</td>
<td>1.23</td>
<td>1.28</td>
<td>1.22</td>
<td>1.17</td>
<td>1.27</td>
<td>1.25</td>
<td>1.25</td>
<td>0.53</td>
<td>1.21</td>
</tr>
<tr>
<td>P2O5</td>
<td>1.44</td>
<td>1.46</td>
<td>0.44</td>
<td>1.22</td>
<td>1.25</td>
<td>1.49</td>
<td>1.50</td>
<td>1.82</td>
<td>1.58</td>
<td>1.76</td>
<td>0.12</td>
<td>1.04</td>
<td>1.28</td>
<td>0.64</td>
<td>1.43</td>
<td>1.01</td>
</tr>
<tr>
<td>MnO</td>
<td>0.11</td>
<td>0.08</td>
<td>0.34</td>
<td>0.15</td>
<td>0.12</td>
<td>0.17</td>
<td>0.17</td>
<td>0.25</td>
<td>0.15</td>
<td>0.22</td>
<td>0.06</td>
<td>0.29</td>
<td>0.15</td>
<td>0.01</td>
<td>0.05</td>
<td>0.10</td>
</tr>
<tr>
<td>SO3</td>
<td>0.01</td>
<td>0.01</td>
<td>1.58</td>
<td>0.97</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sr(ppm)</td>
<td>1.23</td>
<td>1.37</td>
<td>1.55</td>
<td>5.22</td>
<td>5.14</td>
<td>4.59</td>
<td>4.99</td>
<td>4.49</td>
<td>4.86</td>
<td>4.81</td>
<td>3.65</td>
<td>4.15</td>
<td>4.32</td>
<td>4.32</td>
<td>2.65</td>
<td>4.32</td>
</tr>
<tr>
<td>Ba (ppm)</td>
<td>0.08</td>
<td>0.27</td>
<td>0.34</td>
<td>0.15</td>
<td>0.12</td>
<td>0.17</td>
<td>0.17</td>
<td>0.25</td>
<td>0.15</td>
<td>0.22</td>
<td>0.06</td>
<td>0.29</td>
<td>0.15</td>
<td>0.01</td>
<td>0.05</td>
<td>0.10</td>
</tr>
<tr>
<td>Zr (ppm)</td>
<td>98.32</td>
<td>88.71</td>
<td>98.11</td>
<td>96.97</td>
<td>96.51</td>
<td>95.50</td>
<td>95.81</td>
<td>95.61</td>
<td>95.81</td>
<td>95.3</td>
<td>98.8</td>
<td>99.2</td>
<td>98.1</td>
<td>98.3</td>
<td>97.2</td>
<td>98.0</td>
</tr>
</tbody>
</table>

### Table A-3. Chemical Composition of Supplementary Cementitious Materials

<table>
<thead>
<tr>
<th>Material</th>
<th>LSP</th>
<th>Silica Fume</th>
<th>Metakaolin</th>
<th>Slag</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Date</td>
<td>Date</td>
<td>Date</td>
<td>Date</td>
<td>Date</td>
</tr>
<tr>
<td>SiO2</td>
<td>3.54</td>
<td>94.13</td>
<td>50.41</td>
<td>37.39</td>
</tr>
<tr>
<td>Al2O3</td>
<td>0.14</td>
<td>0.37</td>
<td>46.52</td>
<td>9.21</td>
</tr>
<tr>
<td>Fe2O3</td>
<td>0.22</td>
<td>0.01</td>
<td>0.27</td>
<td>0.68</td>
</tr>
<tr>
<td>CaO</td>
<td>52.39</td>
<td>1.14</td>
<td>0.13</td>
<td>40.94</td>
</tr>
<tr>
<td>MgO</td>
<td>0.55</td>
<td>0.36</td>
<td>0.06</td>
<td>9.69</td>
</tr>
<tr>
<td>K2O</td>
<td>0.01</td>
<td>0.51</td>
<td>0.13</td>
<td>0.45</td>
</tr>
<tr>
<td>Na2O</td>
<td>0.10</td>
<td>0.10</td>
<td>0.10</td>
<td>0.25</td>
</tr>
<tr>
<td>Total Alkali (Na eq)</td>
<td>0.11</td>
<td>0.45</td>
<td>0.19</td>
<td>0.55</td>
</tr>
<tr>
<td>TiO2</td>
<td>0.01</td>
<td>0.02</td>
<td>1.58</td>
<td>0.97</td>
</tr>
<tr>
<td>P2O5</td>
<td>0.10</td>
<td>0.13</td>
<td>0.06</td>
<td>0.02</td>
</tr>
<tr>
<td>MnO</td>
<td>0.01</td>
<td>0.03</td>
<td>0.01</td>
<td>0.44</td>
</tr>
<tr>
<td>SO3</td>
<td>1.23</td>
<td>0.13</td>
<td>0.01</td>
<td>0.39</td>
</tr>
<tr>
<td>Sr(ppm)</td>
<td>324</td>
<td>112</td>
<td>59</td>
<td>431</td>
</tr>
<tr>
<td>Ba (ppm)</td>
<td>100</td>
<td>121</td>
<td>177</td>
<td>402</td>
</tr>
<tr>
<td>Zr (ppm)</td>
<td>50</td>
<td>50</td>
<td>118</td>
<td>450</td>
</tr>
<tr>
<td>LOI (10000C)</td>
<td>41.62</td>
<td>3.05</td>
<td>0.49</td>
<td>-1.07</td>
</tr>
<tr>
<td>H2O (1100°C)</td>
<td>58.38</td>
<td>0.6</td>
<td>0.34</td>
<td>0.30</td>
</tr>
<tr>
<td>TOTAL</td>
<td>58.40</td>
<td>97.27</td>
<td>99.46</td>
<td>101.1</td>
</tr>
</tbody>
</table>

---

**Chemical Oxide Composition Analysis by X-ray Fluorescence (XRF) on Dry Sample Basis**
Figure A-1. Relative Bogue calculations for portland cements used in this study
## APPENDIX B - Aggregate Characterization

### Table B-1. Laboratory aggregate gradation

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Weight Retained (lb)</th>
<th>Amount Retained (%)</th>
<th>Cumulative Amount Retained (%)</th>
<th>Cumulative Amount Passing (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Coarse aggregate (crushed limestone)</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3/4-in.</td>
<td>3.5</td>
<td>9%</td>
<td>9%</td>
<td>91%</td>
</tr>
<tr>
<td>1/2-in.</td>
<td>19.1</td>
<td>47%</td>
<td>55%</td>
<td>45%</td>
</tr>
<tr>
<td>3/8-in.</td>
<td>9.5</td>
<td>23%</td>
<td>79%</td>
<td>21%</td>
</tr>
<tr>
<td>#4</td>
<td>7.6</td>
<td>19%</td>
<td>97%</td>
<td>3%</td>
</tr>
<tr>
<td>pan</td>
<td>1.1</td>
<td>3%</td>
<td>100%</td>
<td>0%</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>40.8</td>
<td>100%</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Fine aggregate (natural torpedo sand)</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>#4</td>
<td>0.02</td>
<td>2%</td>
<td>2%</td>
<td>98%</td>
</tr>
<tr>
<td>#8</td>
<td>0.10</td>
<td>9%</td>
<td>11%</td>
<td>89%</td>
</tr>
<tr>
<td>#16</td>
<td>0.14</td>
<td>12%</td>
<td>23%</td>
<td>77%</td>
</tr>
<tr>
<td>#30</td>
<td>0.19</td>
<td>17%</td>
<td>40%</td>
<td>60%</td>
</tr>
<tr>
<td>#50</td>
<td>0.49</td>
<td>44%</td>
<td>84%</td>
<td>16%</td>
</tr>
<tr>
<td>#100</td>
<td>0.15</td>
<td>13%</td>
<td>97%</td>
<td>3%</td>
</tr>
<tr>
<td>#200</td>
<td>0.02</td>
<td>2%</td>
<td>99%</td>
<td>1%</td>
</tr>
<tr>
<td>pan</td>
<td>0.01</td>
<td>1%</td>
<td>100%</td>
<td>0%</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>1.1</td>
<td>100%</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table B-2. Field fine aggregate gradations

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Weight Retained (lb)</th>
<th>Amount Retained (%)</th>
<th>Cumulative Amount Retained (%)</th>
<th>Cumulative Amount Passing (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Kaskaskia</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.25</td>
<td>0.1</td>
<td>2%</td>
<td>2%</td>
<td>98%</td>
</tr>
<tr>
<td>0.125</td>
<td>0.5</td>
<td>10%</td>
<td>12%</td>
<td>88%</td>
</tr>
<tr>
<td>0.0625</td>
<td>0.72</td>
<td>15%</td>
<td>27%</td>
<td>73%</td>
</tr>
<tr>
<td>0.033</td>
<td>0.84</td>
<td>17%</td>
<td>44%</td>
<td>56%</td>
</tr>
<tr>
<td>0.02</td>
<td>1.66</td>
<td>34%</td>
<td>78%</td>
<td>22%</td>
</tr>
<tr>
<td>0.01</td>
<td>0.96</td>
<td>20%</td>
<td>98%</td>
<td>2%</td>
</tr>
<tr>
<td>0.005</td>
<td>0.06</td>
<td>1%</td>
<td>99%</td>
<td>1%</td>
</tr>
<tr>
<td>0.001</td>
<td>0.04</td>
<td>1%</td>
<td>100%</td>
<td>0%</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>4.88</td>
<td>100%</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Lake Springfield</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.25</td>
<td>0.12</td>
<td>2%</td>
<td>2%</td>
<td>98%</td>
</tr>
<tr>
<td>0.125</td>
<td>0.58</td>
<td>11%</td>
<td>14%</td>
<td>86%</td>
</tr>
<tr>
<td>0.0625</td>
<td>0.84</td>
<td>16%</td>
<td>30%</td>
<td>70%</td>
</tr>
<tr>
<td>0.033</td>
<td>0.92</td>
<td>18%</td>
<td>48%</td>
<td>52%</td>
</tr>
<tr>
<td>0.02</td>
<td>1.92</td>
<td>37%</td>
<td>85%</td>
<td>15%</td>
</tr>
<tr>
<td>0.01</td>
<td>0.68</td>
<td>13%</td>
<td>98%</td>
<td>2%</td>
</tr>
<tr>
<td>0.005</td>
<td>0.06</td>
<td>1%</td>
<td>100%</td>
<td>0%</td>
</tr>
<tr>
<td>0.001</td>
<td>0.02</td>
<td>0%</td>
<td>100%</td>
<td>0%</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>5.14</td>
<td>100%</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Big Creek</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.25</td>
<td>0</td>
<td>0%</td>
<td>0%</td>
<td>100%</td>
</tr>
<tr>
<td>0.125</td>
<td>0.72</td>
<td>15%</td>
<td>15%</td>
<td>85%</td>
</tr>
<tr>
<td>0.0625</td>
<td>1.06</td>
<td>22%</td>
<td>37%</td>
<td>63%</td>
</tr>
<tr>
<td>0.033</td>
<td>1.5</td>
<td>31%</td>
<td>67%</td>
<td>33%</td>
</tr>
<tr>
<td>0.02</td>
<td>1.3</td>
<td>27%</td>
<td>94%</td>
<td>6%</td>
</tr>
<tr>
<td>0.01</td>
<td>0.26</td>
<td>5%</td>
<td>100%</td>
<td>0%</td>
</tr>
<tr>
<td>0.005</td>
<td>0.02</td>
<td>0%</td>
<td>100%</td>
<td>0%</td>
</tr>
<tr>
<td>0.001</td>
<td>0</td>
<td>0%</td>
<td>100%</td>
<td>0%</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>4.86</td>
<td>100%</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### Table B-3. Field coarse aggregate gradations

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Weight Retained (lb)</th>
<th>Amount Retained (%)</th>
<th>Cumulative Amount Retained (%)</th>
<th>Cumulative Amount Passing (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1&quot;</td>
<td>0</td>
<td>0%</td>
<td>0%</td>
<td>100%</td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>0</td>
<td>0%</td>
<td>0%</td>
<td>100%</td>
</tr>
<tr>
<td>1/2&quot;</td>
<td>0</td>
<td>0%</td>
<td>0%</td>
<td>100%</td>
</tr>
<tr>
<td>3/8&quot;</td>
<td>1.98</td>
<td>3.5%</td>
<td>3.5%</td>
<td>96%</td>
</tr>
<tr>
<td>#4</td>
<td>35.68</td>
<td>63.9%</td>
<td>67.5%</td>
<td>33%</td>
</tr>
<tr>
<td>pan</td>
<td>18.16</td>
<td>32.5%</td>
<td>100.0%</td>
<td>0%</td>
</tr>
<tr>
<td>Total</td>
<td>55.82</td>
<td>100%</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Small coarse aggregate (crushed limestone)**

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Weight Retained (lb)</th>
<th>Amount Retained (%)</th>
<th>Cumulative Amount Retained (%)</th>
<th>Cumulative Amount Passing (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1&quot;</td>
<td>0</td>
<td>0%</td>
<td>0%</td>
<td>100%</td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>5.12</td>
<td>9%</td>
<td>9%</td>
<td>91%</td>
</tr>
<tr>
<td>1/2&quot;</td>
<td>29.88</td>
<td>50%</td>
<td>59%</td>
<td>41%</td>
</tr>
<tr>
<td>3/8&quot;</td>
<td>15.26</td>
<td>26%</td>
<td>84%</td>
<td>16%</td>
</tr>
<tr>
<td>#4</td>
<td>6.22</td>
<td>10%</td>
<td>95%</td>
<td>5%</td>
</tr>
<tr>
<td>pan</td>
<td>3.2</td>
<td>5%</td>
<td>100%</td>
<td>0%</td>
</tr>
<tr>
<td>Total</td>
<td>59.68</td>
<td>100%</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Large coarse aggregate (crushed limestone)**

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Weight Retained (lb)</th>
<th>Amount Retained (%)</th>
<th>Cumulative Amount Retained (%)</th>
<th>Cumulative Amount Passing (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1&quot;</td>
<td>0</td>
<td>0%</td>
<td>0%</td>
<td>100%</td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>4.096</td>
<td>7%</td>
<td>7%</td>
<td>93%</td>
</tr>
<tr>
<td>1/2&quot;</td>
<td>23.904</td>
<td>41%</td>
<td>48%</td>
<td>52%</td>
</tr>
<tr>
<td>3/8&quot;</td>
<td>12.604</td>
<td>21%</td>
<td>69%</td>
<td>31%</td>
</tr>
<tr>
<td>#4</td>
<td>12.112</td>
<td>21%</td>
<td>89%</td>
<td>11%</td>
</tr>
<tr>
<td>1/8</td>
<td>6.192</td>
<td>11%</td>
<td>100%</td>
<td>0%</td>
</tr>
<tr>
<td>Total</td>
<td>58.908</td>
<td>100%</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Kaskaskia - COMBINED**

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Weight Retained (lb)</th>
<th>Amount Retained (%)</th>
<th>Cumulative Amount Retained (%)</th>
<th>Cumulative Amount Passing (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1&quot;</td>
<td>0</td>
<td>0%</td>
<td>0%</td>
<td>100%</td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>1.1</td>
<td>7%</td>
<td>7%</td>
<td>93%</td>
</tr>
<tr>
<td>1/2&quot;</td>
<td>2.62</td>
<td>50%</td>
<td>57%</td>
<td>43%</td>
</tr>
<tr>
<td>3/8&quot;</td>
<td>1.2</td>
<td>22%</td>
<td>80%</td>
<td>20%</td>
</tr>
<tr>
<td>#4</td>
<td>2.52</td>
<td>17%</td>
<td>96%</td>
<td>4%</td>
</tr>
<tr>
<td>1/8</td>
<td>0.54</td>
<td>4%</td>
<td>100%</td>
<td>0%</td>
</tr>
<tr>
<td>Total</td>
<td>15.18</td>
<td>100%</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Lake Springfield**

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Weight Retained (lb)</th>
<th>Amount Retained (%)</th>
<th>Cumulative Amount Retained (%)</th>
<th>Cumulative Amount Passing (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1&quot;</td>
<td>0</td>
<td>0%</td>
<td>0%</td>
<td>100%</td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>1.1</td>
<td>7%</td>
<td>7%</td>
<td>93%</td>
</tr>
<tr>
<td>1/2&quot;</td>
<td>2.62</td>
<td>50%</td>
<td>57%</td>
<td>43%</td>
</tr>
<tr>
<td>3/8&quot;</td>
<td>1.2</td>
<td>22%</td>
<td>80%</td>
<td>20%</td>
</tr>
<tr>
<td>#4</td>
<td>2.52</td>
<td>17%</td>
<td>96%</td>
<td>4%</td>
</tr>
<tr>
<td>1/8</td>
<td>0.54</td>
<td>4%</td>
<td>100%</td>
<td>0%</td>
</tr>
<tr>
<td>Total</td>
<td>15.18</td>
<td>100%</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Figure B-1. Coarse aggregate gradation

Figure B-2. Fine aggregate gradation
APPENDIX C - Restrained Stress Test Machine (RSTM) Operation Manual

Written by Matthew D’Ambrosia

Test Setup and Instrumentation Instructions

Specimen Preparation

- Clean and oil all surfaces of forms and bolts
- Reassemble forms
- Cast specimens
- After ~12 hours, release restraint (one set of bolts from free, both sets from restrained)
- Remove side molds
- Clean LVDT supports
- Apply foil on the top of the straight section of specimen for symmetric drying conditions.
- Install LVDTs
- Align rod between LVDT supports and bolt in place.
- Install LVDT bracket. Align with rod and bolt in place
- Spacers may be required to align height of LVDT and rod

Pump, Controller, and Computer Setup

- Strain 1 = arbitrary initial value, range =2.5mm
- Set initial reading to within 1mm (~0.6 to 0.8 mm) by adjusting the bracket manually
- Start hydraulic pump
- Open cooling water valves
- Hit “low pressure start”
- Hit “high pressure start”
- Pump should warm up for ~10 min. Check temperature and pressure periodically
- T [ 150° F, p [ 3000 psi, oil level below redline
- Set Instron 8500 Controller “Load Protect” to [ 0.3 kN
- Instron Display Set → Ch 1 to Load, Ch 2 to Position
- Turn load protect off when controlled by computer.
- Set load limits to Max = 2.0, Min = -2.0, Light is “on” when limit is active.
- LabView → follow the red folders, run “Concrete Test S2”
- Position Mode light “on”
- Hydraulics “on”
- Actuator “low”
- Actuator “high”
- pos/N ~ 10 to 13 → 8.656
• Adjust position with arrow keys until pin holes are aligned and insert pin
• Highest channel “0”
• Load protect “off”
• Hit “remote” when prompted
• Set limits to Pos max = 20 mm → “arm”, min =”-10” → “arm”, actuator “off”
• Load max = “1800” → “arm”, min = “-5” → ”arm”, actuator “off”
• LVDT1 max = “2.50” → “arm”, min = “-2.3” → ”arm”, actuator “off”
• Threshold = “1.000” → (to avoid load compensation during early expansion)
• New setpoint → “0.03 kN” → Proceed
• Set user parameters and output file name
• Set interval to read every 10 minutes (600 s) or every minute for basic creep test
• Threshold and load can be changed at anytime
• Change threshold to 0.005 mm when it is observed that specimen is shrinking (~0to6hrs)
• When testing is complete (~6 days) press “halt” arrow
• PID loop parameters Prop → ~ 4 to 6 (“5.0”) Autotune depends on load, use 5.0

**Actuator Orientation Instructions**

Reset Orientation of Actuator
- Function
- Instron Service
- continue
- Unlock
- Return
- more
- more
- Actuator mounting → base
- Set up arrow = north, down arrow = south

**Polarity Check Instructions**

Check polarity
- Function
- Instron Service
- Continue
- Calibration lock
- Unlocked (must be unlocked to change polarity)
- Return
- more
- valve polarity
• Set to inverted for my RSTM setup

Check Redefine Transducer

• L.STR/N
• 0.05 in

Closed Loop (PID) Parameters for the Instron 8500 Controller

A proportional-integral-derivative controller (PID controller) is a common feedback loop component in experimental control systems. The controller takes a measured value from the experiment and compares it with a reference value. The difference (or "error" signal) is then used to adjust some input to the process in order to bring the process' measured value to its desired set point. Unlike simpler controllers, the PID can adjust process outputs based on the history and rate of change of the error signal, which gives more accurate and stable control.

"PID" is named after its three correcting terms, whose sum constitutes the output of the PID controller.

1. **Proportional Gain** - To handle the immediate error, the error is multiplied by a constant $P$ (for "proportional"). Note that when the error is zero, a proportional controller's output is zero. A higher gain typically means faster response since the larger the error, the larger the feedback to compensate

2. **Integral Time** - To learn from the past, the error is integrated and multiplied by a constant $I$. Without integral term, a PID controller cannot eliminate error if the process requires a non-null input to produce the desired set-point (e.g. heater when controlling a temperature or electrical motor when controlling a speed). Smaller integral time implies steady state errors are eliminated quicker. The tradeoff is larger overshoot: any negative error integrated during transient response must be integrated away by positive error before we reach steady state

3. **Derivative Time** - To anticipate the future, the first derivative (the slope of the error) over time is calculated and multiplied by another constant $D$. Larger derivative time decreases overshoot, but slows down transient response

Adjustment of PID Settings

• Choose parameter
  \[
  \begin{align*}
  &\text{Strain 1} \\
  &\text{Load} \\
  &\text{Position}
  \end{align*}
  \]

• Setup
• Loop
<table>
<thead>
<tr>
<th>Strain 1</th>
<th>UA1 (old)</th>
<th>25</th>
<th>1</th>
<th>0</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>UA2 (new)</td>
<td>28</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>Load</td>
<td>UA1 (old)</td>
<td>13</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>UA2 (new)</td>
<td>2.66</td>
<td>2</td>
<td>0.343</td>
</tr>
<tr>
<td>Position</td>
<td>UA1 (old)</td>
<td>37</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>UA2 (new)</td>
<td>30</td>
<td>1</td>
<td>0</td>
</tr>
</tbody>
</table>

**Load Cell Calibration Instructions**

Load Cell Calibration

- Setup
- Cal.
- Cal.
- Manual
- Cors. bal (led blinks)
- Relay → on
- Span → 5000 με → go
- Relay off
- Fine bal →

**LVDT Calibration Instructions**

LVDT Calibration

- Mount LVDT in micrometer using bracket (Pratt & Whitney on yellow cart)
- Zero barrel on micrometer
- Zero LVDT measurement on controller using micrometer coarse balance
- Setup (select Strain1 or Strain2)
- Cal.
- Cal.
- Manual
- Coarse balance (led blinks) → press GO
- Turn micrometer barrel to desired span. Typically 0.05” is good for normal concrete in creep-shrinkage test and gives a range of 2000 με
- Each increment on the micrometer barrel = 0.001” and there are 50 increments on the barrel, so 1 full rotation of micrometer = 0.05”
- Enter selected Span then → press GO
- Turn back to zero measurement on micrometer barrel, then use coarse balance wheel on micrometer to get zero on the controller measurement* This is required because there is slack in the micrometer barrel and it will not return to zero exactly by itself
- Fine balance → GO
- Check linearity at intermediate points in span by moving the barrel one half rotation 0.0250”
- Done
APPENDIX D - Restrained Test Analysis (ReTA) Visual Basic Code

The following program was written in Microsoft Visual Basic for the purpose of analyzing the test data for the Restrained Stress Test Machine (RSTM) at the University of Illinois Urbana-Champaign. Two such machines were built for the purpose of studying tensile creep and shrinkage of concrete. The first device was constructed by Salah Altoubat starting in 1996 and the second device was built by Matthew D’Ambrosia starting in 2001. This program calculates stress, strain, shrinkage, elastic modulus, as well as creep, from the raw data output of the LabView program that is used to control either device. To resolve the incremental load application into steps in order to facilitate the calculation of creep for concrete specimens under restrained tension at early age, the program responds to incremental load application using an IF statement to detect when the loading step has been accomplished. Failure of the test equipment to complete clean and discrete loading steps will render this program and, most likely, analysis of the test result impossible. Therefore, it is advised that when performing early age restrained tests using the RSTM that the utmost care be taken to prevent electrical noise, fluctuation of temperature and relative humidity in the testing environment.

Sub RestrainedTestAnalysis()
' RestrainedTestAnalysis (ReTA)
' Written 10/28/2002 by Matthew D'Ambrosia
' This program allow the user to reduce and analyze the output of a restrained tensile creep test
   Rows("1:1").Select
   Selection.Delete Shift:=xlUp
   Columns("F:F").Select
   Selection.Cut
   Columns("A:A").Select
   Selection.Insert Shift:=xlToLeft
   Columns("B:B").Select
   Selection.Insert Shift:=xlToLeft
   Columns("E:E").Select
   Selection.Insert Shift:=xlToLeft
   Selection.Insert Shift:=xlToLeft
   Selection.Insert Shift:=xlToLeft
Columns("I:I").Select
Selection.Insert Shift:=xlToRight
Columns("K:K").Select
Selection.Insert Shift:=xlToRight
Range("B1").Select
ActiveCell.FormulaR1C1 = "Age (days)"
Range("E1").Select
ActiveCell.FormulaR1C1 = "Stress (MPa)"
Range("F1").Select
ActiveCell.FormulaR1C1 = "Load (lb)"
Range("G1").Select
ActiveCell.FormulaR1C1 = "Stress (psi)"
Range("H1").Select
ActiveCell.FormulaR1C1 = "Strain 1 (me)"
Range("I1").Select
ActiveCell.FormulaR1C1 = "Strain 1 (me)"
With ActiveCell.Characters(Start:=1, Length:=10).Font
  .Name = "Arial"
  .FontStyle = "Regular"
  .Size = 10
  .Strikethrough = False
  .Superscript = False
  .Subscript = False
  .OutlineFont = False
  .Shadow = False
  .Underline = xlUnderlineStyleNone
  .ColorIndex = xlAutomatic
End With
With ActiveCell.Characters(Start:=11, Length:=2).Font
  .Name = "Symbol"
  .FontStyle = "Regular"
  .Size = 10
  .Strikethrough = False
  .Superscript = False
  .Subscript = False
  .OutlineFont = False
  .Shadow = False
  .Underline = xlUnderlineStyleNone
  .ColorIndex = xlAutomatic
End With
.ColorIndex = xlAutomatic

End With

With ActiveCell.Characters(Start:=13, Length:=1).Font
    .Name = "Arial"
    .FontStyle = "Regular"
    .Size = 10
    .Strikethrough = False
    .Superscript = False
    .Subscript = False
    .OutlineFont = False
    .Shadow = False
    .Underline = xlUnderlineStyleNone
    .ColorIndex = xlAutomatic
End With

Range("I1").Select
Selection.Copy Destination:=Range("K1")
Range("K1").Select
ActiveCell.FormulaR1C1 = "Strain 2 (me)"

With ActiveCell.Characters(Start:=1, Length:=10).Font
    .Name = "Arial"
    .FontStyle = "Regular"
    .Size = 10
    .Strikethrough = False
    .Superscript = False
    .Subscript = False
    .OutlineFont = False
    .Shadow = False
    .Underline = xlUnderlineStyleNone
    .ColorIndex = xlAutomatic
End With

With ActiveCell.Characters(Start:=11, Length:=2).Font
    .Name = "Symbol"
    .FontStyle = "Regular"
    .Size = 10
    .Strikethrough = False
    .Superscript = False
    .Subscript = False
    .OutlineFont = False
    .Shadow = False
    .Underline = xlUnderlineStyleNone
    .ColorIndex = xlAutomatic
End With
With ActiveCell.Characters(Start:=13, Length:=1).Font
    .Name = "Arial"
    .FontStyle = "Regular"
    .Size = 10
    .Strikethrough = False
    .Superscript = False
    .Subscript = False
    .OutlineFont = False
    .Shadow = False
    .Underline = xlUnderlineStyleNone
    .ColorIndex = xlAutomatic
End With
Range("B2").Select
ActiveCell.FormulaR1C1 = "={RC[-1]}/60/60/24+1"
Range("B2").Select
Range("B2:B975").Select
Selection.FillDown
Range("E2").Select
ActiveCell.FormulaR1C1 = "={RC[-1]}*1000/(76.2^2)"
Range("F2").Select
ActiveCell.FormulaR1C1 = "={RC[-2]}*224.8089431"
Range("F2").Select
Range("G2").Select
ActiveCell.FormulaR1C1 = "={RC[-1]}/9"
Range("I2").Select
ActiveCell.FormulaR1C1 = "={RC[-1]}-R2C8)/622.3*1000000"
Range("I2").Select
Selection.Copy Destination:=Range("K2")
Range("K2").Select
ActiveCell.FormulaR1C1 = "={RC[-1]}-R2C10)/622.3*1000000"
Range("E2:G2").Select
Range("E2:G975").Select
Selection.FillDown
Range("I2").Select
Range("I2:I975").Select
Selection.FillDown
Range("K2").Select
Range("K2:K975").Select
Selection.FillDown
Columns("J:J").Select
Selection.Insert Shift:=xlToLeft
Range("J1").Select
ActiveCell.FormulaR1C1 = "Counter"
Columns("M:M").Select
Selection.Insert Shift:=xlToLeft
Range("M1").Select
ActiveCell.FormulaR1C1 = "Creep"
Columns("N:N").Select
Selection.Insert Shift:=xlToLeft
Range("N1").Select
ActiveCell.FormulaR1C1 = "Elastic Strain"
Columns("O:O").Select
Selection.Insert Shift:=xlToLeft
Range("O1").Select
ActiveCell.FormulaR1C1 = "Creep Coefficient"
Columns("P:P").Select
Selection.Insert Shift:=xlToLeft
Range("P1").Select
ActiveCell.FormulaR1C1 = "Specific Creep (106"
Range("P1").Select
ActiveCell.FormulaR1C1 = "Specific Creep (106/MPa)"
With ActiveCell.Characters(Start:=1, Length:=18).Font
.Name = "Arial"
.FontStyle = "Regular"
.Size = 10
.Strikethrough = False
.Superscript = False
.Subscript = False
.OutlineFont = False
.Shadow = False
.Underline = xlUnderlineStyleNone
.ColorIndex = xlAutomatic
End With

With ActiveCell.Characters(Start:=19, Length:=1).Font
    .Name = "Arial"
    .FontStyle = "Regular"
    .Size = 10
    .Strikethrough = False
    .Superscript = True
    .Subscript = False
    .OutlineFont = False
    .Shadow = False
    .Underline = xlUnderlineStyleNone
    .ColorIndex = xlAutomatic
End With

With ActiveCell.Characters(Start:=20, Length:=5).Font
    .Name = "Arial"
    .FontStyle = "Regular"
    .Size = 10
    .Strikethrough = False
    .Superscript = False
    .Subscript = False
    .OutlineFont = False
    .Shadow = False
    .Underline = xlUnderlineStyleNone
    .ColorIndex = xlAutomatic
End With

Range("P9").Select
ActiveWindow.SmallScroll ToRight:=2
Columns("Q:Q").Select
Selection.Insert Shift:=xlToLeft
Range("P1").Select
Selection.Copy Destination:=Range("Q1")
Range("Q1").Select
ActiveCell.FormulaR1C1 = "Specific Creep (10^6/psi)"

With ActiveCell.Characters(Start:=1, Length:=18).Font
    .Name = "Arial"
    .FontStyle = "Regular"
    .Size = 10
    .Strikethrough = False
.Superscript = False
.Subscript = False
.OutlineFont = False
.Shadow = False
.Underline = xlUnderlineStyleNone
.ColorIndex = xlAutomatic
End With

With ActiveCell.Characters(Start:=19, Length:=1).Font
  .Name = "Arial"
  .FontStyle = "Regular"
  .Size = 10
  .Strikethrough = False
  .Superscript = True
  .Subscript = False
  .OutlineFont = False
  .Shadow = False
  .Underline = xlUnderlineStyleNone
  .ColorIndex = xlAutomatic
End With

With ActiveCell.Characters(Start:=20, Length:=5).Font
  .Name = "Arial"
  .FontStyle = "Regular"
  .Size = 10
  .Strikethrough = False
  .Superscript = False
  .Subscript = False
  .OutlineFont = False
  .Shadow = False
  .Underline = xlUnderlineStyleNone
  .ColorIndex = xlAutomatic
End With

Range("M2").Select
ActiveCell.FormulaR1C1 = "=RC[-4]-RC[-1]"
Range("M2").Select
Range("M2:M975").Select
Selection.FillDown
Range("N2").Select
Range("J3").Select
Range("J3").Select
ActiveCell.FormulaR1C1 = 
  
  
  
Range("J3").Select
Range("J3:J975").Select
Selection.FillDown
Columns("K:K").Select
Selection.Insert Shift:=xlToRight
Range("K1").Select
ActiveCell.FormulaR1C1 = "Cumulative Strain"
Range("K2").Select
ActiveCell.FormulaR1C1 = 
  
  
  "=-RC[-2]+RC[-1]"
Range("K3").Select
Range("K2").Select
Range("N2").Select
Range("O3").Select
Columns("K:K").EntireColumn.AutoFit
Range("N2").Select
ActiveCell.FormulaR1C1 = 
  
  
  "=RC[-3]-RC[-1]"
Range("K2").Select
Range("K2:K975").Select
Selection.FillDown
Range("N2").Select
Range("N2:N975").Select
Selection.FillDown
Range("O2").Select
Range("O3").Select
ActiveCell.FormulaR1C1 = 
  
  
Range("O3").Select
Range("O3:O975").Select
Selection.FillDown
Range("P2").Select
ActiveCell.FormulaR1C1 = 
  
  
  "=RC[-2]/RC[-1]"
Range("P2").Select
Range("P2:P975").Select
Selection.FillDown
Range("O2").Select
Range("O2").Select
ActiveCell.FormulaR1C1 = "=0.00001"
Range("O2").Select
ActiveCell.FormulaR1C1 = "0"
Range("P2").Select
ActiveCell.FormulaR1C1 = "0"
Range("P3").Select
ActiveCell.FormulaR1C1 = "0"
Range("P4").Select
ActiveCell.FormulaR1C1 = "0"
Range("P5").Select
ActiveCell.FormulaR1C1 = "0"
Range("Q8").Select
ActiveWindow.SmallScroll ToRight:=6
Range("P9").Select
Selection.Copy Destination:=Range("P2")
Range("P2:P8").Select
Selection.FillDown
Range("T1").Select
ActiveCell.FormulaR1C1 = "Elastic Modulus"
Range("T1").Select
ActiveCell.FormulaR1C1 = "Elastic Modulus (MPa)"
Range("T1").Select
Selection.Copy Destination:=Range("U1")
Range("U1").Select
ActiveCell.FormulaR1C1 = "Elastic Modulus (Psi)"
Range("U1").Select
ActiveCell.FormulaR1C1 = "Elastic Modulus (psi)"
Range("T2").Select
ActiveCell.FormulaR1C1 = "'=RC[-15]/RC[-5]'"
Range("T2").Select
Range("T2:T975").Select
Selection.FillDown
Range("U2").Select
ActiveCell.FormulaR1C1 = "'=RC[-14]/RC[-6]'"
Range("U2").Select
Range("U2:U975").Select
Selection.FillDown
Range("T2").Select
ActiveCell.FormulaR1C1 = "=RC[-15]/(RC[-5]/100000)
Range("U2").Select
ActiveCell.FormulaR1C1 = "=RC[-14]/(RC[-6]/100000)
Range("T2:U2").Select
Range("T2:U975").Select
Selection.FillDown
Range("V4").Select
Cells.Select
Cells.EntireColumn.AutoFit
Range("Q2").Select
ActiveCell.FormulaR1C1 = "=RC[-3]/RC[-12]
Range("R2").Select
ActiveCell.FormulaR1C1 = "=RC[-4]/RC[-11]
Range("Q2:R2").Select
Range("Q2:R1173").Select
Selection.FillDown
Range("Q4").Select
Range("P2").Select
ActiveCell.FormulaR1C1 = "=IF(RC[-1]>0,RC[-2]/RC[-1],0)"
Range("P2").Select
Range("P2:P1173").Select
Selection.FillDown
Range("T2").Select
ActiveCell.FormulaR1C1 = "=IF(RC[-5]>0,RC[-15]/(RC[-5]/100000),0)"
Range("U2").Select
ActiveCell.FormulaR1C1 = "=IF(RC[-6]>0,RC[-14]/(RC[-6]/100000),0)"
Range("T2:U2").Select
Range("T2:U1173").Select
Selection.FillDown
Range("R7").Select
Cells.Select
Cells.EntireColumn.AutoFit
With Selection
  .HorizontalAlignment = xlCenter
  .VerticalAlignment = xlBottom
  .WrapText = False
  .Orientation = 0
.AddIndent = False
.IndentLevel = 0
.ShrinkToFit = False
.ReadingOrder = xlContext
.MergeCells = False
End With
Range("C5").Select
ActiveWindow.SmallScroll Down:=12
Rows("1:1").Select
Selection.Font.Bold = True
Cells.Select
Cells.EntireColumn.AutoFit
Range("A1").Select
End Sub
clear;
%%% A material model for concrete shrinkage and creep
%%% originally written by Chang Joon Lee
%%% modified by Matthew D’Ambrosia

%%% test for 1-dof problems:
%%% 1. Unrestrained autogenous shrinkage (scaled)
%%% 2. Autogenous shrinkage + loading (compressive or tensile)
%%% 3. fully restrained autogenous shrinkage

%%%---> INPUT DATA BEGIN
%% Material Properties

E28 = 4200000;  % E of concrete at age of 28
%E28 = 4084750;  % SCC6
%E28 = 4168339;  % MAT1
%E28 = 4130006;  % MAT2
%E28 = 4093724;  % MAT3
%E28 = 4052337;  % MAT4
%E28 = 4016073;  % MAT5
%E28 = 3990533;  % MAT6
%E28 = 3966250;  % MAT7

nu = 0.2;        % Poisson's Ratio
aE = 2;          % parameters for Ec as a function of age
bE = 0.85;       % SEE ACI eq.

fc28 = -6000;    % uniaxial compressive strength, negative denotes compression
ft28 = 600;     % uniaxial tensile strength
aF = 4.3;        % parameters for strength gain as a function of age
bF = 0.92;       % SEE ACI eq.

Va = 0.64;       % aggregate volume fraction
%Va = 0.61;      % SCC6
\%Va = 0.63; \% MAT1
\%Va = 0.62; \% MAT2
\%Va = 0.61; \% MAT3
\%Va = 0.60; \% MAT4
\%Va = 0.59; \% MAT5
\%Va = 0.58; \% MAT6
\%Va = 0.57; \% MAT7

Ep0 = 2620000; \% E of cement paste at porosity ==0 (Ep(28))
Ea = 10700000; \% E of aggregate
p_cal = 0; \% additional porosity of concrete for calibration

%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%%%%%%%
% Creep Function

num_GKM_i = 4; \%# GKM for Internal creep
tau = [0.1 1 10 100]; \% retardation times
k_comp = 1e6*[2.2 5.5 1.1 1.2]; \% SCC1 spring coef. of chains
%k_comp = 1e6*[2.14 5.35 1.07 1.17]; \% SCC6
%k_comp = 1e6*[2.18 5.46 1.08 1.18]; \% MAT1
%k_comp = 1e6*[2.16 5.41 1.08 1.18]; \% MAT2
%k_comp = 1e6*[2.16 5.41 1.08 1.18]; \% MAT3
%k_comp = 1e6*[2.14 5.36 1.07 1.17]; \% MAT4
%k_comp = 1e6*[2.12 5.31 1.06 1.16]; \% MAT5
%k_comp = 1e6*[2.10 5.26 1.05 1.15]; \% MAT6
%k_comp = 1e6*[2.08 5.23 1.04 1.13]; \% MAT6

k_tens = k_comp; \% spring coef. of chains
num_GKM = 4; \% # GKM for External creep
mu = [.1 1 10 100]; \% retardation times
m_comp = 1e6*[99999 99999 99999 99999]; \% spring coef. of chains
m_tens = 1e6*[9999 9999 9999 9999]; \% spring coef. of chains

loading_age = 1; \% loading age
time = loading_age;
inc_time = 0.1; \% time interval
analysis_duration = 90;
input RH, T, applied stress

time_table = [1 1.001 1.5 2 3 7 14 21 28 56 90] + loading_age - 1;
T_table = [23 23 23 23 23 23 23 23 23 23 23];

%RH_table = [100 100 100 100 100 100 100 100];
RH_table = [100 99.9 99.8 99.7 99.55 99.15 98.7 98.36 98.27 97.92 97.5];

% Applied Stress - Positive is tension
%sigma_applied = 0.1 * compute_strength(loading_age , fc28, aF, bF);% psi
sigma_applied = 0;
sigma_table = [0 1 1 1 1 1 1 1 1 1 1] * sigma_applied;

% boundary condition 1 = fixed, 0 = free
bc = 1;

% debug flag
debug_flag = 0;

% INPUT DATA END

%.initialize variables

e_EL1 = 0;
e_EL2 = 0;
e_CR1 = 0;
e_CR2 = 0;
e_total = 0;

inc_e_EL1 = 0;
inc_e_EL2 = 0;
inc_e_CR1 = 0;
inc_e_CR2 = 0;
inc_e_total = 0;

gamma = zeros(num_GKM_i);
beta = zeros(num_GKM);

sigma = 0;
p = 0;

for i=1:1:analysis_duration/ inc_time
RH = discrete_function(time_table, RH_table, time);
T = discrete_function(time_table, T_table, time);
inc_RH = discrete_function(time_table, RH_table, time+ inc_time) - RH;
inc_T = discrete_function(time_table, T_table, time+inc_time) - T;

% Volume
v = 1/((1/ ( time+ inc_time/2))^0.5 +0.1);

% Young's modulus of concrete
E = compute_E(time, E28, aE, bE);
% Internal modulus ( k = E/(3*(1-2*nu)))
E1 = compute_E(time, 11700000, aE, bE);
% External Modulus computed from Ec
E2 = 1/((1/E-1/E1));

% porosity of concrete
p_con = compute_porosity(Va, E, Ea, Ep0) + p_cal;

% smeared pressure - OLD CALCULATION
%p = p_con * compute_pore_pressure(RH, T);
% inc_p = p_con * compute_pore_pressure(RH+inc_RH, T+inc_T) - p;

% New Calculation including saturation factor
S = RH/100;
p = (1-2*nu)*compute_pore_pressure(RH, T)*S*(1-Va);
inc_p = (1-2*nu)*compute_pore_pressure(RH+inc_RH, T+inc_T)*S*(1-Va) - p;

fc = compute_strength(time, fc28, aF, bF); % psi
ft = compute_strength(time, ft28, aF, bF); % psi
CMF_i = 1;
CMF = 1;
%CMF_i = F(sigma+p, fc, ft));
%CMF = F(sigma, fc, ft);

% Check for compression or tension
if (sigma + p >= 0)
k = k_tens;
else
k = k_comp;
end
if ( sigma >= 0)
m = m_tens;
else
m = m_comp;
end
imsi1 = 0;
for j = 1:1:num_GKM_i
    imsi1 = imsi1 + 1/k(j) * (1 - tau(j)/inc_time * (1 - exp(-inc_time/tau(j))));
end
imsi2 = 0;
for j = 1:1:num_GKM
    imsi2 = imsi2 + 1/m(j) * (1 - mu(j)/inc_time * (1 - exp(-inc_time/mu(j))));
end
J = 1/E1 + 1/E2 + CMF_i/v * imsi1 + CMF/v * imsi2;
imsi1 = 0;
for j = 1:1:num_GKM_i
    imsi1 = imsi1 + inc_p/k(j) * (1 - tau(j)/inc_time * (1 - exp(-inc_time/tau(j)))) + ((sigma + p)/k(j) - gamma(j)) * (1 - exp(-inc_time/tau(j)));
end
imsi2 = 0;
for j = 1:1:num_GKM
    imsi2 = imsi2 + (sigma/m(j) - beta(j)) * (1 - exp(-inc_time/mu(j)));
end
R = inc_p/E1 + CMF_i/v * imsi1 + CMF/v * imsi2;
E_star = 1/J;

% if free then calculate incremental strain
if bc == 0
    sigma = discrete_function(time_table, sigma_table, time);
    inc_sigma = discrete_function(time_table, sigma_table, time+inc_time) - sigma;
    inc_e_total = J * inc_sigma + R;
% if fully restrained then calculate incremental stress
else
    inc_e_total = 0;
    inc_sigma = E_star * (-R);
end
inc_e_EL1 = (inc_sigma + inc_p) / E1;
inc_e_EL2 = inc_sigma / E2;
imsi = 0;
for j = 1:1:num_GKM_i
    inc_gamma(j) = (inc_sigma + inc_p) / k(j) * (1 - tau(j)/inc_time * (1 - exp(-inc_time/tau(j)))) + ((sigma + p)/k(j) - gamma(j)) * (1 - exp(-inc_time/tau(j)));
end
for j = 1:1:num_GKM_i
    imsi = imsi + inc_gamma(j);
end
inc_e_CR1 = CMF_i/v * imsi;
imsi = 0;
for j=1:1:num_GKM
    inc_beta(j) = (inc_sigma)/ m(j) * ( 1 - mu(j)/inc_time*( 1- exp( -inc_time/mu(j) )))... 
        + ( (sigma)/m(j) - beta(j) ) * ( 1- exp( -inc_time / mu(j) ) );
end
for j=1:1:num_GKM
    imsi = imsi + inc_beta(j);
end
inc_e_CR2 = CMF / v  * imsi;
check_point = inc_e_total - inc_e_EL1 -inc_e_EL2 -inc_e_CR1 -inc_e_CR2;
if debug_flag == 1
    fprintf('CMF_i= %e, CMF = %e, check=%e
', CMF_i, CMF, check_point);
end
% update
time = time + inc_time;
sigma = sigma + inc_sigma;
for j=1:1:num_GKM_i
    gamma(j) = gamma(j) + inc_gamma(j);
end
for j=1:1:num_GKM
    beta(j) = beta(j) + inc_beta(j);
end
e_total = e_total + inc_e_total;
e_EL1 = e_EL1 + inc_e_EL1;
e_EL2 = e_EL2 + inc_e_EL2;
e_CR1 = e_CR1 + inc_e_CR1;
e_CR2 = e_CR2 + inc_e_CR2;
time_his(i) = time;
stress(i) = sigma;
strain_total(i) = e_total;
strain1(i) = e_EL1;
strain2(i) = e_EL2;
strain3(i) = e_CR1;
strain4(i) = e_CR2;
pressure(i) = p + inc_p;
porosity(i) = p_con;
fcs(i) = fc;
fts(i) = ft;
end
fprint('CMF_i= %e, CMF = %e, check=%e
', CMF_i, CMF, check_point);
e_total
function E = compute_E(ta, E28, aE, bE)
E = E28* ( ta / ( aE+ bE*ta ) )^0.5;

function p = compute_pore_pressure(RH, T)
% RH in %
% T in °C
R = 8.20578e-2 * 14.7;
% L psi k^-1 mol^-1
Vm = 0.018;
% L mol^-1
RH = RH/100;
T = T+273;
p = log(RH)* R * T / Vm;

function p_con = compute_porosity(Va, Ec, Ea, Ep0);
% compute porosity as a function of
p_cen = 1 - ( ( (1+Va)*(Ec-Ea) + ( ((1+Va)*(Ea-Ec))^2 + 4*(1-Va)^2*Ea*Ec )^0.5 ) / (2*Ep0*(1-Va)) )^ (1/3);
p_con = p_cen * (1-Va);

function fc = compute_strength(ta, fc28, aFC, bFC)
fc = fc28* ( ta / ( aFC+ bFC*ta ) );

function y = discrete_function(xdata, ydata, x)
num_data_point = length(xdata);
if( xdata(1) > x)
y = ydata(1);
return;
end
if ( xdata(num_data_point)< x)
y = ydata(num_data_point);
return;
end
for i=1:1:num_data_point
if ( xdata(i) > x)
break;
end
end
i = i-1;
y= (ydata(i+1) - ydata(i)) / ( xdata(i+1)- xdata(i) ) * (x-xdata(i)) + ydata(i);

function CMF = F(sigma, fc, ft)
% fc < 0
% ft > 0
if sigma >= 0
    s = sigma/ft;
else
    s = sigma/fc;
end

CMF = 1;     % No mag.
%CMF = 5*(s^1.5)+1;   %Other
%CMF = (1+s^2)/(1-s^10);   %Bazant

% Autogenous Shrinkage
%mult = 1.000; %SCC1
%mult = 1.092; %SCC6
%mult = 1.031; %MAT1
%mult = 1.065; %MAT2
%mult = 1.096; %MAT3
%mult = 1.127; %MAT4
%mult = 1.161; %MAT5
%mult = 1.192; %MAT6
%mult = 1.223; %MAT7

%data = mult*[1 0;1.2 -3;1.5 -5;2 -9;3 -13;5 -19;7 -24;10 -31;14 -40;21 -53;28 -59;56 -73; 90 -81]

% Basic Tensile Creep + elastic 100 psi(-autogenous)
%data = [1 0; 3 47; 5 60; 7 69; 14 90; 21 101]
%data = [1 0;1.01 42;3 89;5 101;7 111;14 131;21 143;28 164;56 200;90 227]

% Basic Tensile Creep + elastic 100 psi(+autogenous)
%data = [1 0; 3 34; 5 40; 7 45; 14 48; 21 48]
%data = [1 0;1.01 42;3 76;5 81;7 87;14 89;21 90;28 107;56 127;90 148]

% Basic Compressive Creep + elastic 300 psi(+autogenous)
%data = [1 0; 3 -52; 5 -68; 7 -78; 14 -108; 21 -124; 28 -137; 56 -183; 90 -263]
%data = [1 0;1.01 -125;3 -177;5 -193;7 -203;14 -233;21 -249;28 -262;56 -318;90 -388]

% Basic Compressive Creep + elastic 1000 psi(+autogenous)
%data = [1 0; 3 -203; 5 -249;7 -275; 14 -336; 21 -381; 28 -417; 56 -509; 90 -557]
%data = [1 0;1.01 -416;3 -619;5 -665;7 -691;14 -752;21 -798;28 -833;56 -925;90 -973]

%data(:,2) = 1e-6* data(:,2)
subplot(2,1,1);
plot(time_his, strain_total, ':g');
hold on;
plot(time_his, strain1, '-b'); % e1 - solid blue
plot(time_his, strain2, '-r'); % e2 - solid red
plot(time_his, strain3, ':b'); % creep1 - dotted blue
plot(time_his, strain4, ':r'); % creep2 - dotted red
% plot( data(:,1), data(:,2), ':k' ); % test data - black

% axis([0 20 -5E-5 0]);
hold off
title('Strains');

subplot(2,1,2);
plot(time_his, stress, '-b');
hold on;
plot(time_his, pressure, '-r');
plot(time_his, fcs, ':b');
plot(time_his, fts, ':r');
hold off
title('Stress & strength');

% subplot(3,1,3);
% plot(time_his, porosity, '-b');
% title('Porosity');