Early Volume Change of High Performance Concrete

By

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Abstract

In severe chloride/sulfate exposure conditions, there is need to provide more durable concrete, but this usually requires use of concretes with high cementitious binder contents that increase the risk of cracking due to thermal, autogenous, and drying shrinkage. Cracking is counter-productive to obtaining durability in the field. High performance concrete (HPC) can be durable in aggressive environments due to its superior mechanical, resistance to fluid transport, and durability characteristics. However, HPC mixtures typically have high binder contents (cement, pozzolanic minerals), and low w/cm, and therefore undergo high heat of hydration as well as autogenous shrinkage, resulting in rapid volume changes. The increased risk of autogenous deformation in high performance concrete can increase the risk of the cracking and durability of reinforced concrete structures such as bridge decks. For example, in Ontario the MTO had issues with increased incidence of transverse cracking in bridge decks when 50 MPa HPC was used relative to traditional 35 MPa concretes. A review of their construction records indicated that this problem was often related to concretes that set quickly and rapidly developed high temperature rise due to heat of hydration. The effects of ground granulated blast-furnace slag (GGBFS) and silica fume (SF) on the durability of HPC were investigated. In this study, HPC mixtures at 0.33 w/c were made with three sources of blended cements containing 8% SF mixed with 25, 35 and 50% GGBFS replacements by mass.
of cement. The compressive strength, drying shrinkage, and transport properties were measured. The test results have shown that increased fineness of the silica fume blended cement enhances the resistance to fluid transport and mechanical properties, but results in increased drying shrinkage, leading to increased cracking potential.

A comprehensive study was conducted to understand (a) the interrelation between different mechanisms of volume change (thermal deformation, autogenous shrinkage, drying shrinkage) under different standard and non-standard conditions and (b) the role of different shrinkage mitigation methods (e.g. by either increasing GGBFS content to overcome early-age free and restrained thermal deformation as well as autogenous shrinkage, or by reducing the cement binder content).
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Glossary

Autogenous Shrinkage

Autogenous shrinkage is defined as the external-macroscopical (bulk) dimensional reduction (volume or linear) of the cementitious system, which occurs under sealed isothermal unrestrained conditions (Kovler and Jensen 2007).

Granulated blast-furnace slag:

ACI 116R defines slag as nonmetallic product, consisting essentially of silicates and aluminosilicates of calcium and of other bases, that is developed in a molten condition simultaneously with iron in a blast furnace. It is the glassy granular material formed when molten blast-furnace slag is rapidly chilled, as by immersion in water.

Silica Fume:

Silica fume is very finely divided amorphous silica collected in precipitators at silicon metal or silicon alloy furnace in the production of silica metal at about 2000°C.

High Performance Concrete (HPC)

It is defined as a concrete meeting special combination of performance and uniformity requirements that cannot always be achieved routinely using conventional constituents and normal mixing, placing, and curing practices (ACI 318).
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List of abbreviations

Tricalcium silicate (C₃S; 3CaO.SiO₂)
Dicalcium silicate (C₂S; 2CaO.SiO₂)
Tricalcium aluminate (C₃A; 3CaO.Al₂O₃)
Ferrite (C₄AF; 4CaO.Al₂O₃.Fe₂O₃)
Calcium hydroxide (CH: Ca(OH)₂)
Calcium silicate hydrate (C-S-H)
ACI American Concrete Institute
Alumina (Al₂O₃): Aluminium Oxide
ASTM American Society for Testing and Materials
AS Autogenous Shrinkage
GGBFS Ground Granulated Blast Furnace Slag
GU8SF CSA Blended cement with 8% Silica Fume
HPC: High Performance Concrete
OPC: Ordinary Portland Cement
RC: Reinforced Concrete
RH Relative Humidity
SRA Shrinkage Reducing Admixture
MTO Ministry of Transportation Ontario
TD Thermal Deformation
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Chapter 1. Introduction and Problem Statement

1.1 Introduction

High-performance concrete (HPC) has been widely used for the last couple of decades due to its superior performance (low fluid transport properties and high compressive strength), offering substantial enhancement for concrete construction. HPC has economic benefits, significantly reducing the maintenance costs and enhancing service life (Neville and Aitcin 1998). Nowadays, the innovation trends regarding portland cement and supplementary cementing materials (i.e. silica fume (SF), ground granulated blast furnace slag (GGBFS)) helped facilitate high early-age strength concrete, yet field experience indicates that HPC may be prone to higher early-age cracking and shrinkage. The early-age volume change of concrete is a fundamental cause of cracking in concrete structures. The primary components of volume change are thermal dilation, autogenous shrinkage and drying shrinkage (Mindess et al. 2003). ACI 231R-10 reports that the two major driving forces for early-age volume change are the thermal deformation (due to cement hydration) and shrinkage deformation (due to both autogenous shrinkage and drying shrinkage). The hydration reaction causes a reduction in the total volume of the hardened paste, which causes self-desiccation of pores and associated autogenous shrinkage (ACI 231R-10, 2010). Strength development is critical at early-ages when concrete is vulnerable to cracking. Concrete durability reduces with cracking by allowing increased ingress of different aggressive substances (e.g. chloride from deicing salts in northern area, and chlorides in coastal and marine areas). An understanding of the interactions of the volume change mechanisms is crucial to prevention of cracking, thus enhancing service life and durability. Mitigating early-age cracking
will improve the durability of structures over their service life span. Autogenous shrinkage has become of significance with the use of low w/c HPC mixtures, and any additional stresses developed add to early-age stresses resulting from thermal and drying strains.

HPC typically have high binder contents (total of cement, slag and pozzolanic minerals), and low w/cm, and therefore undergo high heat of hydration as well as autogenous shrinkage, resulting in rapid volume changes. The increased risk of autogenous shrinkage in HPC concrete can increase the risk of the cracking and durability of reinforced concrete structures. Autogenous shrinkage is a result of the internal hydration chemical reactions of the concrete components. The contribution of autogenous shrinkage has previously been viewed as “insignificant” in typical concrete mixtures due to the dominant role of drying shrinkage in concretes with water-cement ratio (w/c) >0.4. In this study, to overcome the extent of early-age volume change, several mitigation strategies are implemented (increase GGBFS replacements, uses of low-fineness cement) to avoid/minimize both the thermal deformation and the early-age cracking potential. To appropriately utilize these strategies, it is important investigate the driving forces behind early-age volume change and how shrinkage mitigation methods work from a materials science perspective to reduce shrinkage under field equivalent conditions. The combination of a variety of material properties provides a basis for evaluating and quantifying the contribution of early-age autogenous shrinkage to the total performance of concrete.

1.2 Problem Statement

In severe chloride/sulfate exposure conditions, there is a need to provide more durable concrete, but this usually requires the use of concretes with high cementitious binder contents that increase the risk of cracking due to thermal, autogenous, and drying shrinkage. HPC is superior in aggressive environments due to its improved mechanical (compressive/or splitting strengths, modulus of elasticity),
transport properties (permeability). For example, the MTO has had issues with increased incidence of transverse cracking in bridge decks when 50 MPa HPC was used relative to 35 MPa concrete. A review of their construction records indicated that this problem was often related to concretes that set and developed temperature rise quickly (MTO 2012). It was noted in the MTO 2012 report that as the temperature gradient increases between the peak temperature (at mid height of the bridge deck) and the ambient temperature, the transverse cracking density increases as shown in Figure 1.1. It was reported by MTO (2012) that,

“There are two distinctly different patterns of early temperature development for low cracking density (LCD) and high cracking density (HCD) decks, respectively. The start of temperature increase of the LCD group is delayed by an average of 27 hours relative to the start of temperature increase for the HCD group. The peak temperature of the LCD is delayed by 32 hours relative to the peak temperature of the HCD group.”

Also, autogenous shrinkage has become of significance with use of low w/cm HPC mixtures, and any additional stresses developed add to early-age stresses resulting from thermal and drying strains. However, there is no standard test for measurement of autogenous shrinkage of concrete in current CSA or ASTM standards.
Previously a report by (Krauss & Rogalla, 1996), also concluded that the transverse cracking of bridge decks was a consequence of a combination of thermal deformation, drying shrinkage, and concrete with a high modulus and little creep capacity. Bridge decks exhibit a high degree of restraint because of the composite nature of the deck and supporting girders. The high degree of restraint with no free movement prevents the concrete from shrinking, resulting in buildup of tensile stresses. The major factor influencing the onset of thermal cracking (apart from thermal temperature) is the tensile strain.
capacity of the concrete measure under the very slow rates of strain which the concrete is subjected to in a real structure when cooling down. Additionally, Riding et al. (2009) concluded that the degree of restraint in the deck and bending that occurs in the deck and girders caused by temperature variations affect the stress development.

### 1.3 Thesis outline

The thesis is divided into nine chapters as follows.

Chapter 1: Introduction and problem statement.

Chapter 2: Background on the mechanisms and factors influencing volume change (autogenous, thermal deformations, restrained shrinkage as well as drying shrinkage)

Chapter 3: Experimental methodology (concrete mixtures, tests apparatus and set-up).

Chapter 4: Thermal deformation, hydration kinetics and restrained shrinkage

Chapter 5: Early-age autogenous shrinkage

Chapter 6: Mechanical properties (compressive strength, splitting strength, modules of elasticity); Durability properties (RCPT, sorptivity, salt scaling)

Chapter 7: Later age autogenous and drying shrinkage

Chapter 8: Test results analysis and discussion

Chapter 9: Conclusions and recommendations
Chapter 2. Background and Research Objectives

2.1 Cement reaction

The phases in Ordinary Portland Cement (OPC) undergo a series of exothermal hydration reactions. The main cement compounds are tricalcium silicate (C₃S), dicalcium silicate (C₂S), tricalcium aluminate (C₃A) and tetracalcium aluminoferrite (C₄AF). Tricalcium silicate and dicalcium silicate are the major compounds responsible for strength, their content in cement is around 70 to 80%. The main hydration products of C₃S and C₂S are calcium silicate hydrate (C-S-H, amorphous or micro-crystalline) and calcium hydroxide (CH, crystalline). C-S-H is an amorphous solid of variable composition, able to precipitate onto cement grains and other hydration products and grow into water-filled pores. It is the main hydration product in Portland cement, and the main phase responsible for its strength. C-S-H has a high surface area and possesses very fine pores (gel pores), which are responsible for the shrinkage and the viscoelastic properties of Portland cement paste.

Kinetics of cement hydration: The hydration heats of the cement constituents, (C₃S), (C₂S), (C₃A) and (C₄AF) are 510, 247, 1356 and 427 J/g, respectively, and the final hydration heat of the cement is directly affected by the proportion of these components (Taylor, 1997). The cement hydration process stated as follows: 1) dissolution of cement compound stage (at initial periods), and at later periods; 2) nucleation and crystal growth of hydrates; 3) diffusion of water and dissolved ions through the hydrated materials already formed. Many factors influence the kinetics of hydration process such as phase composition of cement, amount and form of gypsum in the cement (i.e. calcium sulfate can be present in the dihydrate, hemihydrate, or the anhydrite form), and Blaine fineness of cement: The higher the Blaine fineness, the higher surface area of cement particles and therefore an increased rate of reaction. Additionally, a higher water-cement ratio increases the rate of hydration until all of the
cement is consumed. The ambient relative humidity during curing can have major effects on the progress of hydration since water is needed for hydration. An increase in temperature generally causes an increase in the rate of the reaction, although the hydrated structure can be different at different temperatures. Each chemical compound in the cement shows a different hydration rate and total liberated heat. The hydration characteristics of cement compounds are listed in Table 2.1.

Table 2.1 Hydration characteristics of cement compounds (Mindess et al., 2003)

<table>
<thead>
<tr>
<th>Compounds</th>
<th>Reaction rate</th>
<th>Amount of heat liberated</th>
<th>Contribution to cement heat liberation</th>
</tr>
</thead>
<tbody>
<tr>
<td>C₃S</td>
<td>Moderate</td>
<td>Moderate</td>
<td>High</td>
</tr>
<tr>
<td>C₂S</td>
<td>Slow</td>
<td>Low</td>
<td>Low</td>
</tr>
<tr>
<td>C₃A+ C₄SH₂</td>
<td>Fast</td>
<td>Very High</td>
<td>Very High</td>
</tr>
<tr>
<td>C₄AF+ C₄SH₂</td>
<td>Moderate</td>
<td>Moderate</td>
<td>Moderate</td>
</tr>
</tbody>
</table>

2.2 The interaction between the volume change and hydrated products

As the hydration reactions progress, it causes a reduction in microstructural porosity attributed to dense hydrated products which have greater molar volume and less specific gravity compared to the original cement compounds prior mixing (Mindess et al., 2003). It should be mentioned that the chemical reactions of cement hydration generate heat of hydration and a reduction in volume change. The hydration process consumes water molecules, which induces a reduction of the liquid in the pores and therefore causes high capillary pressure (Baroghel-Bouny et al., 2004). Powers (1968) developed an empirical equation (Eq. 2.1) that’s applicable to Portland cements, the equation requires the total
amount of hydration cement products (cement gel) (C-S-H, CH and sulfoaluminates) by means of mass loss of non-evaporable water (combined with hydrated products). There are two types of water: evaporable water which is lost in a typical drying oven up to 100 °C and non-evaporable water which is lost beyond 100 °C, when the paste is heated up to 1000 °C.

\[ w_n = 0.23 \alpha, \quad \text{grams/gram of original cement} \]  
Eq. 2.1

Where \( \alpha \): degree of hydration, assuming a degree of hydration=1, 0.24g of non-evaporable water combined with each gram of cement. For blended Portland cement with 40% slag (w/c=0.5), the non-evaporable water content is 0.173 g/g cement particles based on an age of 14-months (Harrison, Winter, & Taylor, 1987). The 10% silica fume and portland cement paste with 0.45 exhibits a 15.5% non-evaporable water content at 180 days versus portland cement paste with a 20.7% non-evaporable water content (Cheng-Yi & Feldman, 1985). The CH development by means of mass loss using TGA and the chemical shrinkage exhibited a very good relationship (Baroghel-Bony, Mounanga, Loukili, & Khelidj, 2004).

Calcium hydroxide has a wide range of crystals shapes and sizes, these crystals form in irregular hexagonal plate-shaped crystals that nucleate (form initially) in the capillary pores and tend to grow to several microns. Wei (2008) believes that large CH crystals (within the cement matrix) play a main role in limiting the amount of shrinkage. Blended cement (with SF or GGBFS) typically consumes higher amounts of CH than OPC. The rate of reaction is influenced by many factors (fineness, chemical compositions of pozzolanic materials). For example, a blended cement of SF and OPC has a greater early-age rate of reaction (due to its high amorphous silica content and high fineness) than blended GGBFS and OPC (due to low heat energy and reactivity of GGBFS), and consumes CH to form additional C-S-H gel. Blended cement enhances the microstructure of the cement matrix and therefore improves the mechanical properties (early-age with SF+OPC blend/or later age for
GGBFS+OPC blend) and transport properties. However, the higher rate of consumption of the CH crystals causes a reduction of shrinkage restraints within the cement paste (Wei, 2008).

A study conducted by Lura (2003) concluded that OPC cement paste exhibited higher non-evaporable water (bound water) contents in conjunction with high early-age chemical shrinkage compared to blended OPC with slag. The results are displayed in Figure 2.1 and 2.2 for chemical shrinkage and non-evaporable water, respectively. The chemical shrinkage of the slag cement paste which develops beyond the first day was less than the OPC paste up to age of 7 days. The author reported that that the slag paste curve plateaued after 3.5 days hydration, and that could be attributed to de-percolation of the water-filled capillary porosity in the hardening cement paste (Geiker, 1983). According to Lura, another study (Roy, 1982) found similar chemical shrinkage at one day with 50% slag versus portland cement (w/c ratio 0.375). The non-evaporable water content of the slag cement paste is much lower than the one of the portland cement paste.

![Figure 2.1 Chemical shrinkage of cement pastes (Lura, 2003)](image-url)
Another investigation (Meddah & Tagnit-Hamou, 2008) was conducted to evaluate the volume change and pore volume under isothermal conditions. Three types of binder were used with w/c of 0.35 in this study: 1) ordinary portland cement (OPC); 2) binary cement with 8% of the cement (by weight) was replaced by silica fume; 3) ternary binder with 4% silica fume and 22% fly ash. Figure 2.3 presents the autogenous shrinkage development of all the three mixtures. It was interesting that binary mixtures of 8% silica fume exhibited higher length change up to 7 days compared to 0% silica fume. The shrinkage induced by self-desiccation (reduction of internal relative humidity) is the result of water consumption during the cement hydration process. The progress of hydration reactions through capillary water consumption induces the formation of new hydrate products that decrease capillary spaces and induces capillary tension that generates self-desiccation shrinkage. Figure 2.4 presents development of relative pore volume for concrete mixtures at 7 days, as shown in Figure 2.4, the finer the capillary network is, the higher autogenous shrinkage, the higher the capillary stress induced. The higher isothermal shrinkage magnitude was achieved with the binary binder containing
silica fume. It was also concluded that the amount of finer pores smaller than 50 nm was highest for the portland cement mixture compared to the other mixtures. The binary mixture with SF, however, shows the lowest amount of pores smaller than 50 nm.

Figure 2.3 Effect of binder type on isothermal shrinkage concrete mixtures (Meddah & Tagnit-Hamou, 2008)

Figure 2.4 Development of relative pore volume for concrete mixtures at 7 days (Meddah & Tagnit-Hamou, 2008).
2.3 Mechanisms of early-age volume change

Volumetric instability of concrete is the main cause of early-age cracking (ACI 224; Altoubat, 2000; ACI 209R-92, 2005, Riding et al 2008). During hydration at very early-age, there are primarily two volume change mechanisms producing autogenous shrinkage (due to drop of internal humidity) and thermal deformation (due to temperature rise and fall due to exothermic heat of hydration). After the curing stage, the concrete is exposed to a third mechanism which is drying shrinkage that produces a significant volume change in addition to autogenous shrinkage. The following sub-sections discuss particulars of these mechanisms. Tensile stress in concrete elements will develop if volume change from all of these mechanisms is restrained and that can lead to cracking. The early-age volume change of HPC is often increased due to a combination of increased thermal deformation (due high heat of hydration resulting from high binder contents) and autogenous deformation (due to the reduction of relative humidity at low w/c). The following sub-sections describe the mechanisms of early-age volume change. The autogenous shrinkage is related to chemical shrinkage. Jensen and Hansen (2001) reported that net volume reduction (chemical shrinkage) associated with the hydration reactions in a cementitious material is approximately 6 to 7 mL/100 g of fully hydrated cement. The primary compounds of cement (C3S, C2S, C3A, and C4AF) were described in Section 2.1. Each of these minerals requires water for reaction, and results in a decreased volume of the reaction products. As demonstrated by ACI 231R-10 (2010), “Chemical shrinkage is like a molecular-level volume change, and creates the underlying driving force for the occurrence of autogenous shrinkage that is the macroscopic bulk deformation of a closed, isothermal, cementitious material system not subjected to external forces”.

As demonstrated by Hauggaard-Nielsen et al. (1999), Figure 2.5 displays a neck between two solid cement gel surfaces, and the water content is subdivided into the following four distinct phases: 1)
Hindered adsorbed water; 2) Capillary water; 3) Free adsorbed water; 4) Water vapor. The cement gel is characterized by thin spaces where adsorbed water may not be accommodated completely. From thermodynamic considerations (Bažant, 1972) it appears that large local pressures can exist in these areas. Figure 2.5 corresponds to the hindered adsorbed water, and the pressure, termed the disjoining pressure, may have a magnitude of up to 300 MPa. Furthermore, due to the presence of menisci, the capillary water is subjected to pressure, termed capillary tension. The magnitude of the capillary tension in a water-filled pore with a radius of 10 mm is about 30 MPa (Powers, 1968). The disjoining pressure and the capillary tension depend on both the changes of temperature and water content (Bažant, 1972). Free adsorbed water is within the range of the attractive forces.

Figure 2.5 Water Phases between Two Cement Gel Surfaces (Hauggaard-Nielsen, Damkilde, & Hansen, 1999)
2.3.1 Capillary Tension

Changes in pore humidity, menisci and surface tension of the liquid within capillary pores maintain the equilibrium between the liquid and the vapor over the liquid (Wittmann F., 1973; Powers C., 1968). Thus, the capillary pressure develops, which is the vapor pressure minus water pressure. This capillary pressure, in turn, causes water in capillaries to be under tensile stress; it has to be balanced by compressive stress on the surrounding solid:

\[ P_c = \frac{2 \gamma_w \cos \theta_w}{r} \]  

Eq. 2.2

Where \( P_c \) (Capillary pressure); \( \theta_w \) (is the contact angle between pore solution and solid); \( \gamma_w \) (N/m) is the surface tension of the pore solution and \( r \) (m) is the radius of the menisci, also known as Kelvin radius capillary stress, can also be related to the internal relative humidity of a cementitious material using Kelvin’s equation:

\[ RH_k = \frac{RT \ln RH_k}{v_w} \]  

Eq. 2.3

where \( R \) is the universal gas constant (8.314J/mol·K), \( T \) is the temperature (K), \( RH_k \) is the internal relative humidity due to menisci formation (dimensionless) and \( v_w \) is the molar volume of pore solution. Equations 2.2 and 2.3 can be combined into the Kelvin-Laplace equation, which correlates the radius of the menisci in a circular cylindrical pore to the internal relative humidity.

2.3.2 Solid Surface Tension:

Solid surfaces develop surface tension at their interface with other materials. Changes to the surface tension of solid hydration products can cause bulk shrinkage or swelling of the cement system. The
magnitude of solid surface tension is dependent on the thickness of the adsorbed layers of water which lowers the surface tension of the cement gel particles and results in expansion. Conversely, removal of the adsorbed water increases the solid surface tension and creates a net compression of the solid, resulting in micro-scale shrinkage. Surface tension can induce compressive stresses of around 250-300 MPa in cement gel particles with large specific surface area (Wittmann F., 1973; Bažant, 1972) and will consequently produce bulk shrinkage.

2.3.3 Disjoining Pressure:

As defined by Ferraris and Wittmann (1987), the disjoining pressure is a result of the Van der Waals forces (the force holding adjacent particles together), mutual repulsion, and physical force. The Van der Waals force is an attractive interaction and the other two components are repulsive. When a dry paste is saturated with water, the mutual attraction between the water molecules and the solid particles causes water to spread over all of the surfaces of the gel particles that are available to them. When the spread is obstructed by an adjacent particle, a disjoining pressure will develop. Once the disjoining pressure exceeds the van der Waals’ force, the particles are forced apart and dilation or swelling occurs. By contrast, as a saturated material dries out, the van der Waals’ forces between particles draw them closer together and shrinkage occurs. Because cement gel is formed in the dilated state during hydration, the disjoining pressure decreases on first drying at a lowered relative humidity (RH). The decreased disjoining pressure causes the particles to be drawn closer by van der Waals’ forces, and shrinkage occurs (Youssef, 2013). Disjoining pressure is RH dependent and is only a significant factor down to about 45% RH (Mindess et al., 2003; Neville & Aitcin, 1998)
2.4 Thermal Dilation

Thermal dilation is a dimensional change caused by temperature change in concrete. Thermal deformation depends on the coefficient of thermal expansion ($\alpha_T$), which is a function of the state of internal moisture (ACI 231R-10, 2010). Researchers (Meyers, 1950; Zoldners, 1971) reached to same conclusion that ($\alpha_T$) at fully dried or saturated conditions was approximately the same, there was a dramatic increase in the $\alpha_T$ at intermediate relative humidities (RHs). Meyers suggested that in the intermediate RH range, there is additional dilation due to changes in the pore water pressure applied on the cement matrix. Another study concluded that the reduction in surface tension in the pore system was caused by the changes in temperature as water expands (Powers & Brownyard, 1946). The reduction of surface tension in water, leads to negative pressure exerted on the pore system when it goes down (the pressure pushing the solid nanostructure apart increases), causing additional expansion with increases in temperature. As hydration progresses, the exothermic heat of the cement reaction provides momentous amount of heat. In large elements, for example, the heat is harder to dissipate than with thin elements. This temperature rise typically occurs over the first few hours and days, can induce a small amount of expansion that counteracts autogenous and chemical shrinkage (Holt 2001).

The simplified expression of development in thermal stresses due to a temperature rise for concrete may be calculated as presented in Eq. (2.4). The magnitude of the thermal stress is directly proportional to the magnitude of the temperature change to which the reinforced concrete element is exposed. For an accurate estimate of the thermal stress, creep effects during early-ages and throughout the member’s life should be accounted for in Eq. 2.4 (Emborg, 1989)

\[
\text{Thermal stress} = \sigma_T = \Delta T \alpha_T E_c K_r
\]

Eq. 2.4
Where,
\[ \alpha_T = \text{coefficient of thermal expansion, strain/°C} \]
\[ Ec = \text{creep-adjusted modulus of elasticity of concrete, MPa} \]
\[ Kr = \text{degree of restraint factor} \]
\[ \Delta T = \text{measured temperature change (°C) (increase = positive; decrease = negative)} \]

Eq. 2.4 presents that the importance of \( \alpha_T \) in concrete mixtures at early-age of thermal stress. During early-ages, \( \alpha_T \) changes rapidly as the concrete gains strength. Early-age thermal coefficients is reported from different studies (Hedlund, 1996; Byfors, 1980) and is presented in Figure 2.6. The test results reported by Byfors were adopted from Weigler and Alexanderson study. The thermal dilation of concrete at very early-age shows higher values, after 24 hours, the coefficients reduces to \( 12 \times 10^{-6} \)°C (see Fig. 2.8). The \( \alpha_T \) of hardened concrete depends both on the mixture proportions and its thermal properties. There is a general agreement in the literature that the \( \alpha_T \) of a given concrete depends strongly on its water content (Meyers 1950; Neville 1981). A fully saturated pore system and an empty (dried) pore system give lower \( \alpha_T \) values, around 10 to \( 12 \times 10^{-6} \)°C. A partially saturated pore system shows maximum values in the range of \( 25 \times 10^{-6} \)°C.
Figure 2.6 Early-age thermal dilation coefficient for concrete (ACI 231R-10, 2010)

A study was carried out (Kada, Lachemi, Petrov, Bonneau, & Aitcin, 2002) to evaluate the coefficient of thermal expansions in conjunction with temperature profile of different of three different concretes mixtures having water-to-cementitious materials ratios (w/cm) of 0.45, 0.35, and 0.30. Type I Portland cement was used for the concrete with a w/c ratio of 0.45. The concrete for w/c of 0.35 and 0.30 was a Canadian Type GUb8SF (8% SF) cement. Figure 2.7a, b, and c presents the curves of both the coefficient of thermal expansion and the concrete temperature. The coefficient of thermal expansion of the concrete having a w/c of 0.45 drops from $32 \times 10^{-6}/^\circ\text{C}$ to $6.5 \times 10^{-6}/^\circ\text{C}$ in a very short period, less than three hours after its initial setting, to then remain almost constant at $6.5 \times 10^{-6}/^\circ\text{C}$ A similar behavior was observed for the 0.35 concrete, however, for the concrete having a w/c of 0.30 the coefficient of thermal expansion goes from $13 \times 10^{-6}/^\circ\text{C}$ to $7.6 \times 10^{-6}/^\circ\text{C}$ in six hours (duration of
greatest chemical reactivity), to then remain constant at $7.6 \times 10^{-6}/\degree C$. The authors concluded that the variation of the expansion can be attributed to significant differences between the concrete constituents. Water for example has an thermal expansion coefficient of $200 \times 10^{-6}/\degree C$ compared to the aggregate ($10 \times 10^{-6}/\degree C$) or clinker ($5-10 \times 10^{-6}/\degree C$) (Alexandersson, 1972).
Figure 2.7 Early-age thermal coefficients of concrete with/without silica fume and different w/c

(Kada, Lachemi, Petrov, Bonneau,, & Aitcin, 2002)
Another study was conducted by Bjøntegaard & Sellevold (2001) to evaluate the thermal dilation, the total cement binder (355 kg/m$^3$) with 5% silica fume and w/c was 0.4. The thermal dilations were ranging between 4-10.7 x 10$^{-6}$/°C. At early-age, the thermal expansion coefficient of high-strength concrete is difficult to identify as the autogenous shrinkage strain develops at the same time (Hashida & Yamazaki, 2002). It was reported that the thermal expansion coefficient of cement paste increases with age; and is highest when the relative humidity is around 70% (Neville A., 1995; Wittmann & Lukas, 1974). The thermal expansion coefficient of high-strength concrete with a high autogenous shrinkage as a result of reduction of relative humidity, increases with age as shown in Figure 2.8a, referring to the Bjøntegaard and Sellevold (1998) test results. This was also demonstrated by Hashida and Yamazaki (2002). Prior to final setting, the thermal expansion coefficient of concrete is greater as the thermal dilation of uncombined water is relatively large (i.e. 200x10$^{-6}$/°C) and the fabric of the microstructure has not been developed. The thermal expansion coefficient is considered minimized near the time of final set, when the initial fabric has been formed and the concrete is still in a wet condition. It is assumed that thermal dilation at the time of final set is 7.0 x 10$^{-6}$/°C, since the pore humidity correlates with the thermal expansion coefficient regardless of concrete type as shown in Figure 2.8.
Figure 2.8 (a) Model of coefficient of thermal expansion at early-age; (b) Pore humidity versus coefficient of thermal expansion

2.5 Autogenous and Drying Shrinkage

Early-age length change: autogenous shrinkage can be a fundamental cause of cracking if it is not minimized/or avoided. Aitcin et al. (1999) concluded that researchers who wait 24 hours to measure the initial length of low w/c or low w/b concrete can miss the greatest part of autogenous shrinkage as shown in Figure 2.9. A new testing procedure to measure maximum autogenous shrinkage that may develop in uncured concrete is urgently needed. In fact, the current standards in North America ASTM or CSA do not account for autogenous shrinkage in concrete. A study by (Baalbaki, Aïtcin, Lepage, & Dallaire, 1999) reported test results for early-age shrinkage measurements from a 250 by 250 by 900 mm specimen with 0.30 water/binder ratio concrete. They observed that the actual total shrinkage, at the first 50 hours, is due to the temperature increase of concrete and it is much smaller
than the autogenous shrinkage, occurring in isothermal conditions particularly during the first 24 hours. Additionally, from 4 to 16 hours, a minor expansion of concrete has even been observed.

Figure 2.9 ‘’Why ASTM C157 test cannot be used to measure autogenous shrinkage of concrete with low w/b’’(adopted from Aitcin, 1999)

Long term length change: Drying shrinkage is predominant with concretes having high w/c, however, in HPC concrete with low w/c, the contribution of autogenous shrinkage becomes substantial (Brooks, Cabrera, & Johari, 1998; Aitcin, Neville, & Acker, 1997; Aitcin, 2003; Sakata & Shimomura, 2004). Lower drying shrinkage of high-strength concretes can result from the very low water content, but the low w/c is responsible for increased autogenous shrinkage due to high capillary pressure. Figure 2.10 shows the autogenous shrinkage in proportion of total shrinkage for both high strength and conventional strength concrete. Therefore, it is significant to measure both autogenous
shrinkage and drying shrinkage in such concrete with cement binder as the drying shrinkage does not only give the accurate total length change.

Figure 2.10 Illustration of Concrete Shrinkage of a) conventional concrete and b) high strength concrete (Sakata & Shimomura, 2004).

2.6 Factors Influencing Early-Age Volume change

The mix composition has a significant influence on the volume change (autogenous, thermal, or drying shrinkage) as a result of the physical and chemical changes associated with the hydration of cement particles. The influencing factors of the early-age volume change magnitude are uncertain (Persson, 1997). However, the w/c-ratio and silica fume content have been found to be crucial factors that affect autogenous shrinkage. Decreasing the w/c-ratio and/or increasing the silica fume replacement has been found to provide faster development of early-age volume changes (Jensen &
25

Hansen, 1996). This is mainly due to the denser cement matrix and fine pore structure that results as a consequence of lowering the w/c-ratio and/or adding silica fume (Tazawa and Miyazawa, 1997).

2.6.1 Water-to-Cement Ratio

Water-cement ratio has been found to strongly influence the development of chemical shrinkage. The rate of development of chemical shrinkage has also been reported to be influenced by the change in water-cement ratio, a decrease in the water-cement ratio resulted in an increase in the rate of development of chemical shrinkage. Generally, decreasing the w/c results in greater autogenous strains (Holt, 2001, Zhang et al., 2003, Holt and Leivo, 2004) (see Figure 2.11).

Figure 2.11 Autogenous shrinkage resulting from changes in w/c ratio (Holt, 2001)
2.6.2 Admixtures

Weiss et al. (1998), Bentur (2003), Holt (2005), Esping (2006), Lura et al. (2006), Bentz et al. (2001) Lopes et al. (2013) concluded that SRA reduces both autogenous and drying shrinkage strains. It was observed by Weiss et al. (1998) that use of 2% of SRA by mass of cement reduced the free shrinkage by up to 45%. A reduction of approximately 60% in the early-age unrestrained shrinkage was observed within the first week after casting when an SRA dosage of 1-2% by mass of the cement was introduced in the mixture (D’Ambrosia, 2002). The cracks in the mortars were first observed above the central stress riser at approximately 90 minutes after placement. Figure 2.12a shows the average crack width distribution in mortars for three specimens with different SRA content measured after 24 hours of the cast. The addition of SRA results in a slight decrease in the frequency of plastic cracks and a more substantial reduction in the width of the plastic shrinkage cracks. Significant early-age shrinkage was observed with the addition of superplasticizer (SP) as a result of improving cement dispersion, which consequently increases the rate of hydration reactions (see Figure 2.12b) (Holt, 2005; Esping, 2006). In addition, it was pointed out that excessive SP dosage would delay the setting time and result in higher early-age drying shrinkage (Holt, 2004). Sant et al. (2011) investigated the influence of temperature on autogenous volume changes in cementitious materials containing shrinkage reducing admixtures (SRA). It was concluded that SRA do not substantially alter the temperature sensitivity of hydration reactions, however, it reduces the surface tension of the pore solution reducing early-age shrinkage.
Figure 2.12 (a) Influence of SRA on the width of the plastic shrinkage cracks (Lura, Mazzotta, Rajabipour, & Weiss, 2006); (b) Influence of SP on early age shrinkage (Holt, 2004)

2.6.3 Cement Binder

The effects of slag-silica ternary mixtures on thermal deformation, autogenous shrinkage and drying shrinkage of concrete have not been widely studied in the literature. However, some studies have been reported on the impact of slag. Wu et al. (Wu, Roy, & Langton, 1983) observed a reduction in autogenous shrinkage for low-alumina content slag cement (9.5%) versus OPC. On the other hand, opposite behaviour was observed by Lee et al. (2006) with high-alumina slag (14.6%), the autogenous shrinkage increases up to 37% when 50% of the cement was replaced by slag. A study was conducted at the University of Toronto by Perez (Angel-Perez, 2008) with slags having high and low alumina contents of 13.6%, and 10.2% respectively. The low-alumina slag exhibited less drying shrinkage in comparison to the high alumina one, but the shrinkage of the high-alumina slag mixture was reduced by intergrinding calcium sulfate.
Contradictory findings were observed in the literature for the effect of slag on the autogenous deformation. Some studies showed a reduction in autogenous deformation when cement was partially replaced by slag while others showed increased in early-age shrinkage. Wu et al. (1983) observed a reduction in chemical shrinkage for low-alumina content slag cement (9.5%) versus OPC as shown in Figure 2.13. On the other hand, the opposite behaviour was observed by Lee et al. (2006) with high-alumina content slag (14.6%), where the autogenous shrinkage increased up to 37% when 50% of cement was replaced by slag. Perez (2008) carried out an investigation on drying shrinkage using two types of slag having two alumina contents (high level 13.6%, and low level 10.2%). The low-alumina slag concrete exhibited less drying shrinkage in comparison to the one with high-alumina slag.

Figure 2.13 Cumulative chemical shrinkage for Portland and Slag Cement (Wu, Roy, & Langton, 1983).
The cement fineness plays a significant role in autogenous deformation and early-age volume change. Figure 2.14a shows the autogenous deformation versus time as function of cement fineness for four cements ground from the same clinker to different fineness. Interestingly, the setting time for each paste corresponds to the point where each curve first reaches a relative deformation of zero (in age ranges between 5-8 hours). Following the setting time, the two finest cements are observed to display substantial autogenous shrinkage, while the other coarser cements expand before shrinking. Figure 2.14b shows the results of early age cracking in restrained shrinkage tests for fine and coarse cements (Bentz et al. 2001). High fineness portland cement causes high temperature rise, as demonstrated by (Bentz, Bognacki, Riding, & Villarreal, 2011; ACI 207.1R-05, 2005) when high temperature is produced at early age, concrete will be prone to three major issues: thermal cracking, reduced ultimate in-place strength, and delayed ettringite formation (DEF).

Figure 2.14 (a) cement fineness and relative deformation versus time; (b) Residual stress versus time for fine and coarse cement. (Bentz et al. 2001).
The cement fineness also impacts on early-age autogenous shrinkage and thermal deformation (Alas-kar & Hooton, 2016). Tachibana et al. (1990) concluded that the high strength concrete with 10% silica fume by weight of cement and water-cement ratios (w/c = 0.22, 0.25, and 0.28) exhibited total drying shrinkage strains up to $610 \times 10^{-6}$; the normal-strength concrete (w/c = 0.57), on other hand, was about 50% higher. Studies (Powers, 1959; Neville, 1995; Meddah, Suzuki, & Sato, 2011; Jensen & Hansen, 1999) on the effect of silica fume on autogenous shrinkage on high performance/or high strength concrete have concluded that it causes a reduction of pore sizes in the cement paste, resulting in increased surface tension in small capillary pores and therefore increased autogenous shrinkage. Researchers (Atwell, 1974; Yanagida & Ota, 1979; Wainwright & Tolloczko, 1986) concluded that blends of OPC and GGBFS results in reduction of the peak temperature as well as the thermal strains, and thus the thermal cracking.

2.6.4 Aggregate

Researchers have reported that shrinkage of paste has approximately 1.7 times higher shrinkage as compared with mortar (Holt, 2001). A report completed for New Jersey State DOT (Nassif, Aktas, Suksawang, & Najm, 2007) recommended the use of larger aggregate to minimize cracking potential, the results show that concrete mixes with high Coarse Aggregate (CA) to Fine Aggregate (FA) ratio (CA/FA >1.48) reduces the restrained shrinkage and improves the cracking resistance. Liu (2013) also found that increasing the maximum nominal aggregate size from the current 19 mm to 25 mm, and optimizing total aggregate gradation in the HPC concrete mixture allowed a reduction in the total cementitious materials content of up to 25% while retaining the w/c at 0.33. This resulted in improvements in thermal properties, including hydration kinetics, heat evolution, and heat accumulation using semi-adiabatic calorimetry. As well, a reduction was observed in unrestrained drying shrinkage due to the overall reduction of water content in the concrete mixture. It is possible to reduce the cement
content in high performance concrete by increasing aggregate size from 19 to 25 mm and providing optimized gradation without compromising the workability, as is currently the practice of many US DOTs (e.g., Ohio DOT). In addition to the technical benefits, this would improve sustainability through the use of lower total cementitious materials contents. Qiao et al (2010) also recommended the use of larger size coarse aggregate and reduction in paste volume to minimize cracking potential. Naik et al. (2006) reported that the concrete made with crushed quartzite exhibited higher autogenous shrinkage than concrete mixture semi-crushed river gravel and crushed dolomite. Use of dolomitic limestone appears to be useful in reducing early-age autogenous shrinkage and drying shrinkage of concrete compared with using river gravel or quartzite as coarse aggregate. Use of crushed dolomitic limestone in concrete led to the lowest early-age drying shrinkage, followed by semi-crushed river gravel, and crushed quartzite stone. However, at later ages, the drying shrinkage became similar for most aggregates. River gravel often resulted in the highest drying shrinkage at later ages. It was concluded that the effect of the source of coarse aggregate on drying shrinkage appears to be noticeable but small. The thermal coefficients (i.e. thermal expansion, thermal conductivity, and specific heat) of aggregate play a significant role in volume change. The aggregate thermal properties exert a significant influence on the coefficient of thermal expansion of concrete at later ages. Riding et al. (2009) reported that concrete made with river gravel aggregate exhibited a higher coefficient of thermal expansion as well as up to 33% higher tensile stresses than with limestone aggregate. Additionally, Riding and co-authors found that using a coarse aggregate with a lower coefficient of thermal expansion and placing the concrete at night resulted in a 50% reduction in early-age thermal stresses.

Table 2.2 present the thermal properties of typical aggregates used in concrete (Kim, Jeon , Kim , & Yang, 2003; Lamond & Pielert, 2006; Horai, 1971; Waller , De Larrard, & Roussel, 1996).
Table 2.2 Thermal properties of coarse aggregate

<table>
<thead>
<tr>
<th>Aggregate</th>
<th>Thermal conductivity of aggregate (W/(m.K))</th>
<th>Specific heat of aggregate (J/(g.K))</th>
<th>Coefficient of thermal expansion of aggregate (10^-6/°C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Siliceous river gravel</td>
<td>5-8</td>
<td>0.75</td>
<td>10-12</td>
</tr>
<tr>
<td>Dolomitic Limestone</td>
<td>4-5</td>
<td>0.84</td>
<td>7-10</td>
</tr>
<tr>
<td>Limestone</td>
<td>2-3</td>
<td>0.84</td>
<td>3.5-6</td>
</tr>
</tbody>
</table>

The thermal expansion of concrete is similar to that of the aggregate used. Table 2.3 presents the thermal properties of concrete made with Siliceous river gravel, Dolomitic Limestone, and Limestone. The test results are based on previous studies (Scanlon & McDonald, 1994; Riding, Poole, Schindler, Juenger, & Folliard, 2009; Sakyi-Bekoe, 2008; Emanuel & Hulsey, 1977)

Table 2.3 Thermal properties of concrete made with different coarse aggregate

<table>
<thead>
<tr>
<th>Aggregate</th>
<th>Thermal conductivity of aggregate (W/(m.K))</th>
<th>Specific heat of aggregate (J/(g.K))</th>
<th>Coefficient of thermal expansion of aggregate (10^-6/°C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Siliceous river gravel</td>
<td>3.1-4.1</td>
<td>0.97</td>
<td>10.8-12.5</td>
</tr>
<tr>
<td>Dolomitic Limestone</td>
<td>3.3</td>
<td>0.97</td>
<td>5.3-9.9</td>
</tr>
<tr>
<td>Limestone</td>
<td>2.2-3.2</td>
<td>1.04</td>
<td>3.5-6</td>
</tr>
</tbody>
</table>

Use of water-saturated lightweight coarse aggregate (LWA), lightweight sand (LWS), or super-absorbent polymers (SAP) were decreased the autogenous shrinkage by up to 38, 45% and 50%, respectively (Liu, 2013; Youssef, 2013; Craeye, Geirnaert, & De Schutter, 2011). The LWA, LWS and
SAP provide extra pore space which acts as water reservoirs when saturated, and these materials are starting to be used in concrete for the purpose of internal curing. HPC concrete provides a low permeability, and therefore, it doesn’t allow external curing water to penetrate into the concrete element. Well-dispersed sources of internal water saturate the drying capillary pore network (which gets dry due to chemical shrinkage). Internal curing is an effective tool to mitigate autogenous shrinkage, however, it reduces the compressive strength. It should be mentioned that pre-wetted lightweight aggregate will not overcome the temperature rise effect on early age volume change, therefore, it was not used as mitigation tool in this study.

2.7 Early-age cracking

The temperature rise and stress developed in two hypothetical concretes shown in Figure 2.15 demonstrate the mechanism of restrained cracking stress due to thermally induced volume changes for two concretes as presented in the literature review of the MTO report (2012). Concrete A shows a rapid temperature rise to the peak temperature compared to concrete B. Concrete expands when concrete temperature rises, therefore compressive stresses developed rapidly for concrete A. Points T1 and T2 represent zero stress temperatures. Early-age thermal shrinkage occurred during the cooling process (beyond the peak temperature). In restrained concrete, the period beyond temperatures T1 and T2 causes tensile stresses, leading to potential for premature cracking. The higher the zero stress temperature, the higher the potential for thermally induced cracking. The timing of cracking depends on the rate of concrete cooling; the temperature at which tensile stresses build up to exceed the tensile strength is defined as the cracking temperature, CT. It can be seen that concrete A has a high cracking potential as the tensile stresses exceed its tensile strength at temperature CT, while concrete B has a low cracking potential; the rate of concrete cooling is low and tensile stresses remain lower than its
tensile strength. There is a strong correlation between temperature development and the cracking density (MTO, 2012).

![Concrete temperature curves and restrained tensile stress versus time at early age](image)

Figure 2.15 Concrete temperature curves and restrained tensile stress versus time at early age (MTO, 2012)
A restrained shrinkage testing protocol was designed and built at University of Toronto, in order to evaluate the stress development of the tensile stress and evaluate the cracking risk under fully restrained conditions. The restrained stress was measured by means of load cell. A study was conducted by Xinwei, Changren, & Hooton (2005) to evaluate the effect of water-cement ratio on the uniaxial restrained stress, the mixtures were four batches with portland cement, and 12 mm maximum size aggregate, the mixtures were labelled as M0.25, M0.30, M0.35 and M0.40, with 0.25, 0.30, 0.35 and 0.40 w/c, respectively. All the curves behave similarly, as the tensile stress increased with time. The lower the w/c, the higher the tensile stress, and therefore the higher risk of early-age cracking. The highest tensile stress of the specimen with water-cement ratio of 0.25 is 3.4 MPa at the age of 6 days when the specimen cracked. The splitting tensile strength at the age of cracking is 5.6 MPa. The ultimate tensile stress under restrained conditions is approximately 60% of the splitting tensile strength. The others had not broken until the experiments were stopped at the age of 7 days (see Figure 2.16).
A study by (Bentur, Igarashi, & Kovler, 2001) was conducted to evaluate the tensile restrained stress of dog-bone specimens. The isothermal restrained shrinkage tests (under controlled temperature of 30 °C) were conducted. The mixture consist of 444 kg/m³ of portland cement and 49 kg/m³ of silica fume concrete with w/c of 0.33. Two types of aggregate were used normal-weight aggregate (NWC) and lightweight aggregate (LWA). Two conditions, Air Dry (AD) and Saturated Surface-Dry (SSD), were used with LWA. The NWC mixture exhibited rapid stress development within the first 24 hours and kept developing sharply until cracking at an age of 144 hours, as shown in Figure 2.17. However, the other concrete LWA-AD and LWA-SSD did not show any tensile failure up to the age of 168 hours.

Figure 2.16 Stress development verus time (hours) (Xinwei, Changren, & Hooton, 2005)
hours. A Transportation Research Board (TRB) report (TRB, 2006) has classified cracking of concrete as presented in Table 2.4. The current study focuses on transverse cracking of the bridge decks, and its causes.

Figure 2.17 Development of restraining stresses for lightweight and normal-weight concretes

(LWC: lightweight concrete; NWC: normal-weight concrete)

Results of a study (Hedlund & Westman, 1997) on temperature and thermal stress development are displayed in Figure 2.18. Two mixtures were evaluated: 1) Normal Strength Concrete (NPC); 2) High Performance Concrete (HPC 3). The water to binder ratio for HPC 3 and for NPC are 0.31 and 0.40
respectively. The authors observed that with HPC3, tensile forces started to develop through the heating stage, and it was possibly related to effects of shrinkage due to self desiccation (drop in the internal humidity) due to the very low water to binder ratio. The NPC mixture exhibited lower stress development as well as a low peak temperature. However, the HPC 3 shows a higher peak temperature and rapid stress changing from compression to tension, leading to premature cracking, and that could be attributed to higher restrained tensile stress than ultimate concrete strain.

Figure 2.18 Temperature development and test results obtained in a thermal stress test-frame (Hedlund & Westman, 1997)
### Table 2.4 Cracking classifications (TRB, 2006)

<table>
<thead>
<tr>
<th>Type of cracking</th>
<th>Form of cracks</th>
<th>Primary cause</th>
<th>Time of appearance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plastic settlement</td>
<td>Over and aligned</td>
<td>Poor mixture design leading to excessive blending, excessive vibrations</td>
<td>10min to 3 h</td>
</tr>
<tr>
<td>Plastic shrinkage</td>
<td>Diagonal or random</td>
<td>Excessive early evaporation</td>
<td>30min to 6 h</td>
</tr>
<tr>
<td>Thermal expansion and contraction</td>
<td>Transverse</td>
<td>Excessive heat generation, excessive temperature gradients</td>
<td>1 day to 2-3 week</td>
</tr>
<tr>
<td>Drying shrinkage</td>
<td>Transverse, pattern or map cracking</td>
<td>Excessive mixture water, inefficient joint, large joint spacing</td>
<td>A week to months</td>
</tr>
<tr>
<td>Freezing and thawing</td>
<td>Parallel to the surface of the concrete</td>
<td>Lack of proper air void system, Non-durable coarse aggregate</td>
<td>After one or more winters</td>
</tr>
<tr>
<td>Corrosion of reinforcement</td>
<td>Over reinforcement</td>
<td>Inadequate cover, ingress of sufficient chloride</td>
<td>More than after 2 years</td>
</tr>
<tr>
<td>Alkali-aggregate reaction</td>
<td>Pattern and longitudinal cracks parallel to the least restrained side</td>
<td>Reactive aggregate plus alkali hydroxides plus moisture</td>
<td>Typically more than 5 years, but weeks with highly reactive material</td>
</tr>
<tr>
<td>Sulphate attack</td>
<td>Pattern</td>
<td>Internal or external sulphates promoting the formation of ettringite</td>
<td>1 to 5 years</td>
</tr>
</tbody>
</table>

An investigation carried out by Springenschmid et al. (1994) to evaluate the effect of initial concrete temperature on cracking magnitude at early age, they concluded that reducing the initial concrete temperature from 25 to 12°C contributed to reduce the cracking temperature by 12 to 19°C (see Figure 2.19). This significant reduction in the cracking temperature is attributed to a smaller temperature increase during the hydration phase as well as an increase of the tensile strength of the hardening concrete (Springenschmid & Breitenbücher, 1998; Breitenbücher & Mangold, 1994). Breitenbücher and Mangold (1994) suggested that increasing the initial concrete temperature by 10°C leads to an increase the cracking temperature by 13 to 15°C, so low fresh concrete temperature reduces the heat of hydration and therefore decreases the zero stress temperature and therefore reduces thermal deformation which is generated during the cooling phase and therefore reduces the tensile stress. Taylor, (1986) and Verbeck and Helmuth (1968) concluded that low curing temperature helps develop stronger C-S-H products, resulting in an increase in tensile strength. Springenschmid and
Breitenbücher (1994) suggest increased tensile strength of the hardening concrete can be explained by the bonds formed between the hydration products.

Figure 2.19 Influence of initial concrete temperature on cracking sensitivity (Springenschmid and Breitenbücher 1998)
A study by Breitenbücher and Mangold (1994) to evaluate the early age stress cracking of different cement received from different cement plants. A total of seventeen concrete mixtures were tested in a restrained test machine. Both temperature history and stress development over 7 days were monitored. The cracking temperatures varies from 6 to 22°C, even though the temperature developments for concrete were different, the wide range of cracking temperatures can only be partly attributed to the various temperature histories as well as different chemical compositions and/or fineness (see Figure 2.20).

Figure 2.20 Effect of Cement content and cement sources on cracking sensitivity (Breitenbücher and Mangold 1994)
2.8 Research Motivations and Objectives

Due to the high cementing materials content typically used in HPC concrete, it tends to be more vulnerable to developing high temperature gradients between the concrete temperature and ambient temperature. Susceptibility to thermal cracking in concrete depends on the heat evolved during cement hydration. Therefore, the influence of chemical and physical properties of GU8SF cement and GGBFS on the thermal characteristics of HPC needs to be determined. It is important to optimize the heat of hydration and volume change stability to avoid cracking of HPC concrete. Information on early-age volume change of ternary blended cement (OPC+SF+GGBFS) is scarce. Therefore, the focus of this study is to experimentally evaluate the synergic effect of ternary blended cement binders (OPC+SF+GGBFS) for mitigating the early-age volume instability, as well as on the durability requirements. The factors that influence the heat of hydration of concrete include the placement temperature of the concrete, the volume and the hydration heat characteristics of the cement (hydration of heat of cement, hydration temperature). The primary objective of this research is to prevent or minimize the occurrence of transverse cracking in the HPC concrete bridge decks through both use of new tools for selection of the HPC Mixes that will result in lower heat of hydration cementing materials and provide better control of temperature differentials at early ages. Studies showing the influence of temperature on the development of volume change are scarce.

The research objectives were addressed as follows:

1. Evaluation of the development of volume change by means of several experimental measurements (thermal strain, autogenous strain).

2. To lower the concrete temperature by utilizing higher replacement of cement by GGBFS and evaluate its impact on early-age deformation, mechanical properties of paste.
3. To develop and recommend changes, if needed, in construction practices for the MTO to use in specifications related to reducing early-age volume change and cracking potential in high-performance concrete bridge decks.

4. To develop an effective, convenient and promising prequalification tests and limits for HPC that would help reduce the risk of early-age cracking of bridge decks.

5. To develop and implement a new testing and analysis method using a semi-adiabatic specimen apparatus as a tool for evaluation of thermal cracking resistance of concrete.

6. To establish an interrelationship between different test results to evaluate the most sensitive parameters influencing early-age volume change.
Chapter 3. Experimental Program

3.1 Introduction

The experimental program was carried out as detailed in this chapter. The main mix design variables considered in this investigation are:

1. Three sources of GU8SF.
2. Two sources of GGBFS and three replacement amount (25%, 35%, and 50%).
3. Two total binder contents (465, 350 kg/m$^3$).
4. Shrinkage-reducing admixture, SRA.
5. Control of initial concrete temperature (15, 23 °C).

The experimental program consists of various tests to assess early-age volume change (thermal deformation, autogenous shrinkage), and later-age volume change (drying shrinkage, autogenous shrinkage), durability (rapid chloride permeability, sorptivity, de-ice salt scaling), mechanical properties (compressive strength, splitting strength, modulus of elasticity), and thermal behavior (heat of hydration).

3.2 Mix Design

A standard MTO HPC (50MPa) bridge deck mix design was used as the control (with GGBFS replacements of 25%). Portland cement interground with eight percent silica fume cement (CSA type GU8SF) was used along with separately added slag (GGBFS). The mixtures consist of 465 kg/m$^3$ of cementitious material and a w/c of 0.33. The total binder content was selected based on a 50 MPa mix design as provided from one of the MTO ready mix concrete suppliers. The main mix design variables are the cement and slag types as well as total binder content. Three sources of Blended
Hydraulic Cement type GU8SF were used. The cements were labelled as follows: cement A (low-fineness, high-alkali), cement B (high-fineness, low alkali), and cement C (high-fineness, high-alkali). The ground granulated blast-furnace slags (GGBFS) were supplied from two manufacturers in Ontario. The slags were named as follows: GGBFS I (low-alumina content) and GGBFS II (high-alumina content). Table 3.1 shows the chemical and the physical properties of the cements and slags. Figure 3.1 and Figure 3.2 present cumulative particle size distribution analysis of different cements and GGBFS were used in this study, respectively.
Table 3.1 Cement Binder Compositions

<table>
<thead>
<tr>
<th></th>
<th>GU8SF cement</th>
<th>GGBFS</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Binder ID</strong></td>
<td>A</td>
<td>B</td>
</tr>
<tr>
<td><strong>Chemical Analysis</strong> (%)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>LOI</td>
<td>2</td>
<td>1.9</td>
</tr>
<tr>
<td>SiO₂</td>
<td>25.1</td>
<td>26.4</td>
</tr>
<tr>
<td>Al₂O₃</td>
<td>4.4</td>
<td>4.6</td>
</tr>
<tr>
<td>Fe₂O₃</td>
<td>2.7</td>
<td>2.9</td>
</tr>
<tr>
<td>CaO</td>
<td>57.3</td>
<td>56.1</td>
</tr>
<tr>
<td>MgO</td>
<td>2.6</td>
<td>2.2</td>
</tr>
<tr>
<td>SO₃</td>
<td>3.7</td>
<td>3.78</td>
</tr>
<tr>
<td>K₂O</td>
<td>-</td>
<td>0.55</td>
</tr>
<tr>
<td>TiO₂</td>
<td>-</td>
<td>0.45</td>
</tr>
<tr>
<td>Total Alkali</td>
<td>0.91</td>
<td>0.59</td>
</tr>
<tr>
<td><strong>Physical analysis</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Blaine (m²/kg)</td>
<td>582</td>
<td>854</td>
</tr>
</tbody>
</table>
Figure 3.1 Cumulative particle size distribution analysis of three GUb8SF cements

Figure 3.2 Cumulative particle size distribution analysis of two GGBFS
The maximum nominal size of the coarse aggregate for all mixtures was 19mm, except that 25mm was used for one mixture to reduce the cement binder content. The 19 mm dolomitic crushed stone was obtained from Dufferin’s Milton Ontario quarry and the natural sand from the St Marys Sunderland pit in Ontario. The saturated surface-dry (SSD) relative density for the sand was 2.68, while absorption was 0.63%. The SSD relative density for the 25 mm and 19 mm coarse aggregates were 2.688 and 2.7, respectively, while absorptions were 0.53 and 1.42 %, respectively.

Table 3.2 presents the concrete mixture IDs and mix proportions. The effect of alumina content of the GGBFS on early thermal deformation was evaluated. The percentage of GGBFS was varied in the process of achieving the desirable early-age strength, as well as the 28 and 56 day transport properties of HPC mixes. Other factors such as initial concrete temperature was also modified. Reducing the initial concrete placing temperature was tested to evaluate the effect of early-age temperature on volume age. It has been noted from an MTO report (2012) that bridge decks with low cracking density were typically cast with low initial concrete temperature 15 ºC and longer onset of temperature rise due to hydration. Therefore, two initial concrete temperatures (15 ºC and 23 ºC) were used, in order to see the effects of low and high initial temperature on early-age behaviour of the HPC. The maximum nominal size of the coarse aggregate for all mixtures was 19 mm, except that 25mm was used for one mixture to reduce the cement binder content. The 19 mm dolomitic crushed stone was obtained from the Dufferin Milton Ontario quarry and the glacial sand from St. Marys Sunderland pit in Ontario.
Table 3.2 Concrete mix proportions

<table>
<thead>
<tr>
<th>Mixture ID</th>
<th>Ternary Binder %</th>
<th>GU8SF (kg/m³)</th>
<th>GGBFS (kg/m³)</th>
<th>Water (kg/m³)</th>
<th>Aggregate (kg/m³)</th>
<th>Fresh concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Fine</td>
<td>Coarse</td>
<td>Density (kg/m³)</td>
</tr>
<tr>
<td>A-II-25S</td>
<td>75% GU8SF (A)+25%GGBFS (II)</td>
<td>349</td>
<td>116</td>
<td></td>
<td>619 1068</td>
<td>2375</td>
</tr>
<tr>
<td>A-II-35S</td>
<td>65% GU8SF (A)+35% GGBFS (II)</td>
<td>163</td>
<td>302</td>
<td></td>
<td>619 1068</td>
<td>2359</td>
</tr>
<tr>
<td>A-II-50S</td>
<td>50% GU8SF (A)+50% GGBFS (II)</td>
<td>233</td>
<td>233</td>
<td></td>
<td>619 1068</td>
<td>2337</td>
</tr>
<tr>
<td>C-II-25S</td>
<td>75% GU8SF (C)+25%GGBFS (II)</td>
<td>349</td>
<td>116</td>
<td></td>
<td>619 1066</td>
<td>2391</td>
</tr>
<tr>
<td>C-II-35S</td>
<td>65% GU8SF (C)+35% GGBFS (II)</td>
<td>163</td>
<td>302</td>
<td></td>
<td>619 1068</td>
<td>2380</td>
</tr>
<tr>
<td>C-II-50S</td>
<td>50% GU8SF (C)+50% GGBFS (II)</td>
<td>233</td>
<td>233</td>
<td></td>
<td>619 1068</td>
<td>2377</td>
</tr>
<tr>
<td>C-I-25S</td>
<td>75% GU8SF (C)+25%GGBFS (I)</td>
<td>349</td>
<td>116</td>
<td></td>
<td>619 1068</td>
<td>2368</td>
</tr>
<tr>
<td>C-I-35S</td>
<td>65% GU8SF (C)+35% GGBFS (I)</td>
<td>163</td>
<td>302</td>
<td></td>
<td>619 1068</td>
<td>2354</td>
</tr>
<tr>
<td>C-I-50S</td>
<td>50% GU8SF (C)+50% GGBFS (I)</td>
<td>233</td>
<td>233</td>
<td></td>
<td>619 1068</td>
<td>2366</td>
</tr>
<tr>
<td>B-II-25S</td>
<td>75% GU8SF (B)+25%GGBFS (II)</td>
<td>349</td>
<td>116</td>
<td></td>
<td>619 1068</td>
<td>2384</td>
</tr>
<tr>
<td>C-II-25S (25mm max aggregate)</td>
<td>75% GU8SF (C)+25%GGBFS (II)</td>
<td>260</td>
<td>87</td>
<td></td>
<td>747 773 (25mm agg.)</td>
<td>2393</td>
</tr>
</tbody>
</table>

*Note: water content is adjusted based on the moisture content of sand and coarse aggregate. Dosing of water reducer range (0.125-0.250% of total cement) and superplasticizers admixture range (0.7-1.1% of total mass of cement) were adjusted to achieve 130-170mm slump. Shrinkage reducer admixture is 2% of total mass of cement. All aggregate quantities shown are in surface saturated dry condition.
### 3.3 Test Matrix

Table 3.3 presents the test matrix including both standard tests and proposed new test methods to evaluate the early-age thermal and autogenous deformation in conjunction with temperature development in semi-adiabatic conditions.

<table>
<thead>
<tr>
<th>No.</th>
<th>Category</th>
<th>Testing</th>
<th>Objective</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Mechanical</td>
<td>Compressive strength (ASTM C39)</td>
<td>Provides strength development</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td>Splitting tensile Strength (ASTM C496)</td>
<td>Provides the ultimate tensile strength resistance to cracking</td>
</tr>
<tr>
<td>3</td>
<td>Properties</td>
<td>Static elastic modulus (ASTM C 469)</td>
<td>Modulus of elasticity</td>
</tr>
<tr>
<td>4</td>
<td>Volume</td>
<td>Early-age thermal deformation</td>
<td>Evaluate field-like internal strain and temperature</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td>Early-age autogenous shrinkage under constant temperature (up to 7 days)</td>
<td>To separate the AS from TD to accurately measure AS</td>
</tr>
<tr>
<td>7</td>
<td>Change</td>
<td>Autogenous (not standard for concrete) + Drying Shrinkage (MTO LS-435 R23, 2006)</td>
<td>Evaluate the free total length change (of sealed and unsealed specimen 23 °C)</td>
</tr>
<tr>
<td>8</td>
<td></td>
<td>Uniaxial semi-adiabatic restrained shrinkage</td>
<td>Evaluate the thermal cracking potential under semi-adiabatic condition</td>
</tr>
<tr>
<td>9</td>
<td>Transport</td>
<td>RCPT (ASTM C1202)</td>
<td>Chloride penetration, salt scaling</td>
</tr>
<tr>
<td>10</td>
<td>Properties</td>
<td>Sorptivity (ASTM C1585)</td>
<td>Rate of absorption</td>
</tr>
<tr>
<td>11</td>
<td>Thermal</td>
<td>Heat of hydration (ASTM C1679) concreante</td>
<td>Evaluate the heat of hydration development</td>
</tr>
</tbody>
</table>
3.3.1 Volume Change methods

3.3.1.1 Autogenous shrinkage

The test set-up was designed to assess the autogenous shrinkage of HPC mixtures for up to 168 hours at constant temperature to de-couple the thermal deformation from autogenous deformation. The test set up used here was selected to reduce the prism cross section and increase the number of copper loops in order to reduce the temperature difference between the copper wall and the center of the specimen to within 0.5±0.25 °C to ensure sufficient decoupling of autogenous and thermal deformations. A schematic drawing of the test set up and mold cross section for measuring autogenous strain are presented in Figures 3.3a and 3.3b. Shrinkage was measured using concrete prisms of 75x75x500 mm. An insulated isothermal chamber was controlled with a constant temperature 20 °C water bath (i.e. temperature stability ± 0.005 °C). The water was circulated through the copper tubes soldered to the outer walls of the copper mold. The copper walls were insulated with 50mm thick rigid foam insulation. The length change was measured at both ends using external linear variable differential transducers (LVDTs) in addition to the use of an embedded strain gauge in the middle of the prism (see Figure 3.3b). The LVDT’s stroke/gauging range and sensitivity were ±1.27 mm and 0.05 micro volt/mm, respectively. The LVDTs were connected to stainless steel pins embedded in the concrete at each end of the prism, the internal clear length between the pins is 350 mm as shown in Figure 3.3b. The strain gauge and thermocouple embedded in each concrete cylinder and were connected to a computer-controlled data logger, 20-30 minutes right after casting the concrete cylinders, and the measured thermal strain and temperature versus of each specimen were saved using DaisyLab software. The embedded strain gauges used in this study (Tokyo Sokki Kenkyujo, KM-120 series) which can measure the strain of concrete that undergoes a transition from plastic to hardened state. The modulus of the strain gauge is low, around (3.7 GPa), and waterproofed so is suited for
internal strain measurement during the very early-age of curing. The coefficient of thermal expansion of the strain gauge is $11 \times 10^{-6}/^\circ\text{C}$. The mold was oiled (to minimize friction between the copper wall and the specimen), and was lined with one layer of heavy duty polythene plastic sheet (0.25mm thick). Test results on identical mixes showed that this apparatus has good reproducibility, with an error of ±3%.

![Figure 3.3a Cross section of Isothermal Autogenous Shrinkage Test Set-up](image-url)
3.3.1.2 Thermal Deformation

The thermal deformations were measured in a semi-adiabatic condition. Two insulated 150 x 300 mm cylinder molds were surrounded with 100 mm thick rigid polystyrene insulation and filled with concrete just after mixing. The test set-up for obtaining the thermal deformation and temperature profile is shown in Figure 3.4. The room temperature was controlled at 25 ± 2 °C. The internal strain and the hydration temperature of the specimens were obtained every 10 minutes by data acquisition. Several methods were developed to measure the thermal dilation/or coupled thermal deformations. The strain gauge and thermocouple were vertically embedded at mid height in each concrete cylinder and were connected to a computer-controlled data logger, 20-30 minutes right after casting the concrete cylinders, and the measured thermal strain and temperature versus of each specimen were saved.
using DasyLab software. The embedded strain gauges used in this study (Tokyo Sokki Kenkyujo, KM-120 series). The maximum operating temperature of strain gauge KM-120 series is 70 °C. In addition, they are impermeable to moisture absorption; thus, they can produce stability for long-term strain measurement as well.

Table 3.4 Previous methods to measure thermal deformation

<table>
<thead>
<tr>
<th>References</th>
<th>Sample Size, mm</th>
<th>Insulation thickness (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(Termkhajornkit, Nawa, Nakai, &amp; Saito, 2005)</td>
<td>125 diameter x 250 height cylinder</td>
<td>25</td>
</tr>
<tr>
<td>(Liwu &amp; Min, 2006)</td>
<td>150 diameter x 300 height cylinder</td>
<td>50</td>
</tr>
<tr>
<td>(Lee, Lee, Lee, &amp; Kim, 2006)</td>
<td>100x100x400</td>
<td>N.A.</td>
</tr>
<tr>
<td>(Hashida &amp; Yamazaki, 2002)</td>
<td>100x100x400</td>
<td>N.A.</td>
</tr>
</tbody>
</table>
3.3.1.3 Long-term Autogenous and drying shrinkage:

Six prismatic specimens for each HPC mixture (75x75x285 mm) were cast to evaluate the drying and autogenous shrinkage (3 for each tests). The drying shrinkage samples were then stored in an environmental chamber with 50 ±5% relative humidity and temperature of 23 ±2 ºC. The autogenous shrinkage prisms were sealed with four layers of adhesive aluminum tapes. This type of sealing was found to be efficient and exhibited very minimal weight loss (Brooks et al., 1998). The strain readings were taken for both drying and autogenous shrinkage at 7, 14, 28, 56, 91, 180, and 360 days.
3.3.2 Hydration kinetics

3.3.2.1 Semi-adiabatic hydration temperature

The test takes two samples (150x300 cylinder) of total 11 litres in volume, and it contains two temperature sensors: one located in the center of the chamber and one on the outside. As a result, mass concrete condition can be simulated. Prior to casting, the chamber was oiled with demolding agent and a plastic bag was placed on top. One more layer of demolding agent was applied on the inside of the plastic bag before casting. The chamber was filled with concrete in two layers. Each layer was rodded 25 times with a 16 mm tamping rod and was compacted by slightly lifting and dropping the edge ten times. This test ran for seven days (or 168 hours).

3.3.2.2 Heat of hydration

Testing for isothermal heat of hydration was done in accordance with ASTM C1679 – 09 standard practice for measuring hydration kinetics of hydraulic cementitious mixtures using isothermal calorimetry, and the baseline temperature was set to 23 °C. A Calmetrix I-CAL 8000 was used for this experiment. The calorimeter has eight channels, and each channel takes a specimen contained in a 125 ml plastic cup. Since the sample size is relatively small for concrete specimens, two specimens from each mixture were tested. The test duration was 168 hours, and readings were taken 10-15 minutes apart. Due to the distance between mixing and test locations, the start time of the test was inevitably delayed. Time of mixing was recorded so data could be corrected to begin at zero maturity. It is acknowledged that the sample volume of 125 ml allowed in this test was small for concrete, especially when aggregate with maximum nominal size of 25 mm was used, as result could vary depending on how much paste or aggregate was selected during sampling. Therefore, wet sieving of
concrete sample was done in accordance to ASTM 172. Fresh concrete was sieved through 4.75mm sieve to remove large coarse aggregate prior to filling the 125ml sample container.

3.3.3 Uniaxial restraint test

The uniaxial restrained dog-bone shrinkage specimen were used to evaluate the initial stress development of different GGBFS replacement under semi-adiabatic condition, one end was fixed and the other was connected to manual adjustable cell load. Two strain gauges were embedded into the concrete at different location and one thermocouple (type-T) was inserted to monitor the temperature development (See Figure 3.5a,b). The load cell records the compressive and tensile stress induced sample deformation. The strain gauges, the load cell was recorded by a computer. To reduce the friction between the concrete and the mold, the mold was lined with polycarbonate sheet, and the concrete was cast in one layer of heavy duty polyethylene plastic sheet (0.25 mm thick) placed on the polycarbonate sheets. The bond between the polythene plastic sheet and the concrete is excellent, so the movement of the specimen due to shrinkage occurs between the polythene plastic sheet and the polycarbonate sheets. This polythene plastic sheet also serves to seal the specimen, that drying shrinkage is avoided and the length change is a result of thermal and autogenous shrinkage. The concrete specimen is 75 mm x 75 mm in cross section x 1200 mm working length.
Figure 3.5a Test set-up for thermal deformation

Figure 3.5b Test set-up for thermal deformation
3.3.4 The mechanical, transport properties, salt scaling properties

Cylindrical specimens were used for compressive and splitting strength and rapid chloride permeability (RCPT). The strengths were tested on cylinders of 100 mm in diameter and 200 mm in height. For RCPT, four 50 mm thick samples were cut from two 100mm diameter cylinders on the day of testing, and prepared according to ASTM C1202-10. For water sorptivity tests, three 50mm thick samples were conditioned and tested according to ASTM C1585-13. Samples were conditioned for seven days, three days oven drying at 50ºC followed by sealed storage for four days at 50ºC to obtain a more uniform moisture distribution and an internal relative humidity of 50-70% (Shahroodi, 2010).

3.3.5 Salt scaling

De-icer salt scaling test of concrete surfaces is an important concrete durability issue, particularly in northern regions, where the cold climate results in the extensive use of deicing salts. A 250 mm x 300 mm x 75 mm slab was cast for the A-II-25S, A-II-35S, A-II-50S, C-II-25S, C-II-35S, and C-II-50S mixtures for evaluation of salt scaling. These slabs were also demolded at 24 h and stored in the moist curing room for 14 days of moist curing followed by 14 days of air drying. 25 mm high Styrofoam strips were affixed atop the recessed border with silicone sealant. The result was the formation of a dike around the raised region to allow ponding of a 3% NaCl solution to the required depth of 5 mm on the test surface. Two methods of scaling were used MTO-LS 412 and CSA A23.2 – 22C. MTO-LS 412 procedures, where the top samples of slab were ponded with 3%NaCl solution and then subject to freeze thaw cycles. In CSA A23.2 – 22C, the slabs were ponded with salt solution and then covered with a lid to avoid evaporation for 7 days at room temperature prior to start the 50 freezing and thawing cycles.
Chapter 4. Thermal Deformation and Hydration Kinetics

4.1 Introduction

This study was undertaken to evaluate the interplay between chemical compositions, physical properties, and the replacement levels of blended GUb8SF cements (high-fineness and low-fineness) and GGBFS (high-alumina and low-alumina). The effect of alkali content of GUb8SF cement is also considered in this study. The impact of GUb8SF properties on the magnitude of volume change and hydration kinetics of HPC is important to discover. Several researchers (Bamforth, 1980; Lee et al., 2006; Baalbaki et al., 1999) agree that temperature rise may lead to premature cracking due to high tensile stress development due to differential volume changes. This chapter aims to investigate the early-age volume changes and thermal properties of HPC mixtures under semi-adiabatic conditions. This was achieved by evaluating the effects of GGBFS alumina content and replacement levels on thermal deformation and hydration kinetics. The mixture identification code designations are listed in Table 4.1. The test variables include; the level of GGBFS replacements (25% (reference), 35%, 50%), source of cement (A, B, C), source of GGBFS (low-alumina content (I); and high-alumina content (II)), and two total cement binder (i.e. the presence of 25mm max nominal aggregate). The mix design was previously presented are defined in Table 3.2. The restrained shrinkage was used to evaluate the effects of GGBFS replacement on cracking at early ages. The reactivity indices of the two GGBFS used in the current study are calculated for the chemical compositions using Equation 4.1 which was developed by (Cheron & Lardinois, 1968), and it is based on a linear relationship between mechanical strength and the hydraulic index. This ratio was derived from tests on 56 differing chemical compositions. The reactivity index varied from 1.4 to 2.2.

\[
Reactivity\ index\ of\ GGBFS = \frac{CaO+1.4MgO+0.56Al_2O}{SO_2} \quad \text{Eq. 4.1}
\]
Table 4.1 shows the concrete mixture identities and including the GGBFS replacements, alumina content, and reactivity index. Also, the Blaine, and alkali content of cements A, B, and C.

### Table 4.1 Mixtures for thermal deformation

<table>
<thead>
<tr>
<th>Mix ID</th>
<th>GGBFS</th>
<th>GUb8SF</th>
<th>remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Replacement level%</td>
<td>Alumina content%</td>
<td>reactivity index</td>
</tr>
<tr>
<td>A-II-25S</td>
<td>25</td>
<td></td>
<td></td>
</tr>
<tr>
<td>A-II-25S (15 °C)</td>
<td>25</td>
<td></td>
<td></td>
</tr>
<tr>
<td>A-II-35S</td>
<td>35</td>
<td></td>
<td></td>
</tr>
<tr>
<td>A-II-50S</td>
<td>50</td>
<td></td>
<td></td>
</tr>
<tr>
<td>C-II-25S</td>
<td>25</td>
<td></td>
<td></td>
</tr>
<tr>
<td>C-II-35S</td>
<td>35</td>
<td></td>
<td></td>
</tr>
<tr>
<td>C-II-50S</td>
<td>50</td>
<td></td>
<td></td>
</tr>
<tr>
<td>C-I-25S</td>
<td>50</td>
<td></td>
<td></td>
</tr>
<tr>
<td>C-I-35S</td>
<td>50</td>
<td></td>
<td></td>
</tr>
<tr>
<td>C-I-50S</td>
<td>50</td>
<td></td>
<td></td>
</tr>
<tr>
<td>C-II-25S (25mm agg.)</td>
<td>25</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B-II-25S</td>
<td>25</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

61
The fineness of cementitious materials as well as the heat of hydration are also factors that affect both the pore structure of capillary network and shrinkage development (Meddah & Tagnit-Hamou, 2008; Bhanumathidas & Kalidas, 2003). The rate of heat release during cement hydration is an important indicator of the progress of hydration and the formation of both solid products and empty pores. For mixtures containing 50% GGBFS, the heat generated was the lowest regardless the cement fineness or type of slag used (low-alumina or high-alumina). The actual replacement levels of SCM for three GGBFS replacement levels are as follows: 1) 25% GGBFS mixtures contain 6% SF and 69% OPC; 2) 35% GGBFS mixtures contain 5.2% SF and 69.8% OPC; 3) 50% GGBFS mixtures contain 4% SF and 46% OPC. The higher peak temperature with 25% GGBFS (with 6% SF, 69% OPC) replacement is attributed to a higher content of portland cement and silica fume compared to 35% GGBFS (with 5.2% SF and 69.8% OPC) and 50% GGBFS (with 4% SF and 46% OPC).

4.2 Research significance and objective

An MTO report (2012) surveyed 20 bridge decks which exhibited early-age premature transverse cracking issues. The report shows that concretes with high hydration temperatures exhibited higher cracking density and that could be attributed to high thermal dilation. It was reported that the temperature development and the cracking density have a strong correlation (MTO, 2012). In response to the MTO report, this chapter is focused on evaluating the early-age volume change due to thermal response of different concretes (i.e. high GGBFS replacements, cement fineness, cement alkali content, and GGBFS alumina content).

4.3 Thermal deformation and hydration temperature

The semi-adiabatic strain development in a sealed specimen is a result of autogenous shrinkage plus the thermal dilation that is caused by cement hydration. The strain gauge is influenced by temperature
rise of concrete. Therefore, the actual thermal strain of concrete is adjusted, by subtracting the coefficient of thermal expansion of the strain gauge \((8 \times 10^{-6} / ^\circ C)\) and multiply it by the temperature change as shown in Equation 4.1.

\[
\varepsilon_{\text{adjusted strain}} = \varepsilon_{\text{measured strain}} - \Delta T \times \alpha_{\text{COE strain gauge}} \tag{Equation 4.1}
\]

Where:

\(\varepsilon_{\text{adjusted strain}} = \) concrete thermal strain; \(\varepsilon_{\text{adjusted strain}} = \) actual measured strain; \(\Delta T = \) concrete temperature change with time; \(\alpha_{\text{COE strain gauge}} = \) coefficient of thermal expansion of the strain gauge.

The thermal strain analysis presents the effect of silica fume blended cement (GUb8SF) compositions and fineness on the temperature profile as well as the strain development. GUb8SF cements A and C have relatively similar chemical compositions with alkali contents of 0.91\%, however, GUb8SF Cement B has a low alkali content (0.59\%). The alumina \((\text{Al}_2\text{O}_3)\) content of GGBFS II (10.3\%) is higher than GGBFS I (7.63\%). The chemical compositions and physical properties of all blended cements and GGBFS are presented in Table 3.1. Detailed schematic drawings of thermal deformation test setup are provided in Figure 3.4. Mixtures with higher slag replacement have reduced peak temperature and thermal strain as well. Different researchers agree that GGBFS reduces the early-age temperature profile as the level of GGBFS increases (Wainwright & Tolloczko, 1986; Bamforth, 1980; Lee et al., 2006; Swamy, 1986). However, there is limited data on the interaction of blended cement with silica fume and GGBFS on early-age thermal properties. The initial strain change of the concrete occurs during the transition from the plastic state to the hardened state. The strain due to plastic state deformation involves the transition process from liquid to semi-liquid state and transformation to the harden state (i.e. initial set is the start of the hardened state). The strain developed during the plastic stage is relatively high compared to the hardened state. Researchers (Kasai et al., 1972; Byfors 1980)
stated that concrete has a high tensile strain capacity at very early-ages when concrete is still in its liquid phase. However, strain capacity decreases sharply and reaches its lowest level during the viscoelastic (transition) phase as the concrete hardens. The plastic state strain also involves the effect of settlement (Hammer, 1998). In the hardened state, the initiation point of the thermal deformation (autogenous shrinkage + thermal dilation) started around the time of initial setting where temperature rise starts to take place, and the peak temperature of the concrete is around final setting (Taylor, 1965; Neville, 1995). It should be mentioned that it is difficult to accurately predict the time of initial setting. The observed start of shrinkage corresponded to the onset of the rapid rise in temperature of the concrete, which was most likely to be the period between the initial and the final setting. Researchers (Aitcin P., 1998; Zhang, Tam, & Leow, 2003) mentioned that the autogenous shrinkage starts at the initial setting time of concrete. Typical starting ages for all the specimens was between 10-24 hours. Neville and Brooks (2010) mentioned that time of set can be determined by the temperature profile; initial set corresponds to a rapid rise in temperature, and final set corresponds to the peak temperature. False set can be differentiated due to the lack of heat production.

4.3.1 Low-fineness cement A

Figure 4.1 shows the hydration temperature, and thermal plus autogenous deformation history curves for the low fineness cement A with inclusion of different replacement levels of high-alumina slag (GGBFS II) over a testing period of 168 hours; each curve represents the average of two tests. Three HPC mixtures with slag replacement levels A-II-25S, A-II-35S and A-II-50S (details presented in Table 4.1) are plotted in Figure 4.2. The strain started to develop rapidly around the initial setting time in conjunction with the temperature rise due to the generated heat of hydration. The A-II-25S and A-II-35S mixtures exhibited a higher strain rate between initial and final set. The rate of deformation changed in conjunction with the rapid temperature evolution between 13 and 24 hours for
mixtures A-II-25S and A-II-35S, followed by a slower change in strain rate beyond 24 hours. However, mixture A-II-50S shows a flatter strain rate in comparison to A-II-25S and A-II-35S. At an age of 168 hours, the measured ultimate strains of A-II-35S and A-II-50S were reduced compared to A-II-25S by 18% and 35%, respectively. The peak temperatures for the A-II-35S mixture (54 °C) and for the A-II-50S mixture (50 °C) were reduced compared to A-II-25S (58 °C).

Figure 4.1 Measured strain and semi-adiabatic temperature rise of concrete with low-fineness cement and different high-alumina GGBFS II replacements
4.3.2 High-fineness cement C

The high fineness cement was used with two different GGBFS (high alumina and low alumina slag).

4.3.2.1 High-alumina slag

The temperature history and thermal plus autogenous deformation developed over the first 168 hours for high-fineness GUb8SF cement C with different slag replacement levels of GGBFS II (Mixtures C-II-25S, C-II-35S and C-II-50S) are plotted in Figure 4.2. All mixtures exhibited strain development that started around the initial setting time of 11-16 hours. The rate of strain development for C-II-25S and C-II-35S between initial and final set was high. The strain developed rapidly for C-II-25S and C-II-35S, followed by a reduction in the rate of strain beyond 24 hours. However, mixture C-II-50S shows a relatively flatter strain rate in comparison to C-II-25S and C-II-35S. At 168 hours, the measured ultimate strains of C-II-35S and C-II-50S were reduced compared to C-II-25S by up to 27% and 41%, respectively. The maximum temperatures of specimens C-II-25S, C-II-35S and C-II-50S were 63.7, 59.0 and 54.8 °C, respectively. The maximum temperature of specimen C-II-25S was approximately 8% and 18% higher than that of C-II-35S and C-II-50S, respectively.
Figure 4.2 Measured strain and semi-adiabatic temperature rise of concrete with high-fineness cement and different replacement levels of high-alumina GGBFS II

4.3.2.2 Low-alumina slag

The effect of low-alumina GGBFS on curves of coupled thermal and autogenous strain for high-fineness cement with three levels of low-alumina slag replacement (GGBFS I) is shown in Figure 4.3. Mixtures C-I-25S, C-I-35S and C-I-50S are described in Table 4.1. The first slope (i.e. between 13 and 26 hours) where the strain developed rapidly in mixture C-I-25S and C-I-35S in conjunction with rapid temperature rise. After the temperature peaks at around 24-26 hours, a reduction in the rate
of strain rate was observed with the C-I-25S and C-I-35S mixtures. However, mixture C-I-50S shows a flatter deformation slope in comparison to C-I-25S and C-I-35S. At end of the test period, the strain of C-I-35S and C-I-50S was reduced compared to C-I-25S by up to 28% and 43%, respectively. The maximum temperature mixture for the 25% slag mixture was approximately 7% and 19% times higher than that of the 35% and 50% slag mixtures, respectively.

![Graph showing measured strain and semi-adiabatic temperature rise of concrete with high-fineness cement and different replacement levels of low-alumina GGBFS I](image)

Figure 4.3 Measured strain and semi-adiabatic temperature rise of concrete with high-fineness cement and different replacement levels of low-alumina GGBFS I
4.3.3 Low and high alkali content of GUb8SF cements

Figure 4.4 presents the temperature and thermal plus autogenous strain curves versus time. Both mixtures have 25% GGBFS II replacement. The major difference was the source of cement. Cement B (Na$_2$O=0.59%) was used to evaluate the effect of alkali content on the early-age thermal deformation. The concrete with Cement C (Na$_2$O =0.91%) exhibited a higher peak temperature (63.7$^\circ$C) compared to cement B (57$^\circ$C). The ultimate strain at 168 hrs of concrete made with cement C reached 310 x $10^{-6}$ versus 224 x $10^{-6}$ for cement B. The results showed that the high-alkali cement C has a significant effect on early age thermal strain development as well as the temperature history. It was observed that time to peak temperature for cement C was 18hrs, however, for cement the time to peak temperature was approximately 34 hrs. This indicates that there is a more rapid onset of temperature development with high-alkali cement compared to low-alkali cement, in agreement with previous studies (Odler & Wonnemann, 1983; Jawed & Skalny, 1978). It was mentioned that setting time is extended as alkali content (Na$_2$O$_{eq}$) is reduced. Many researchers have investigated the effect of alkali content on heat kinetics of the cement system, and they conclude that higher alkali content also increases the temperature rise.
Figure 4.4 Measured strain and semi-adiabatic temperature rise of concretes made with high and low alkali cements with 25% replacement of high-alumina GGBFS II

4.3.4 Binder content

Reducing the total cement binder content from 465 to 347 kg/m$^3$ was used to evaluate the effects of how higher aggregate volume can reduce the risk of cracking by reducing the heat evolution and, therefore reducing the early thermal deformation. It is evident that both temperature rise and ultimate thermal strain of the low-cement binder mix were substantially reduced. The temperature and measured ultimate strain were reduced by 24% and 37%, respectively.
Figure 4.5 Measured strain and semi-adiabatic temperature rise of concrete made with high-fineness cement at different cement contents: Low level (347 kg/m³) and high level (465 kg/m³)

4.3.5 Initial concrete temperature

Reducing the initial concrete placing temperature can significantly influence the thermal properties of concrete. Figure 4.6 presents the test results for a concrete cast at two different initial temperatures (23 and 15 °C). Both mixes had the same mix proportions and a slag replacement level of 25%. The black bold curve shows the 23 °C concrete mixture and it has an earlier temperature onset (at age of 12 hrs) compared to the concrete cast at low-temperature (at age of 22 hrs).
Figure 4.6 Measured strain and semi-adiabatic temperature rise of concrete with low-fineness cement cast at: low initial temperature (15 °C) and normal initial temperature (23 °C)

4.4 Heat of Hydration

Heat of hydration (HOH) was also determined using an isothermal calorimeter as another method of assessing heat generation during cement hydration besides the temperature development in semi-adiabatic tests described in Section 4.3. The heat of hydration of concrete was measured in accordance with ASTM C1679 – 14 (2014). Fresh concrete was sieved through 4.75mm sieve to remove large coarse aggregate up 125ml sample container. Cement fineness and GGBFS alumina content directly
influence reactivity and consequently the hydration rate. The heat evolution curves are presented in Figures 4.7, 4.8 and 4.9. The cumulative heat of hydration curves are presented in Figures 4.10, 4.11 and 4.12. The mixtures with high-alumina slag and different cement fineness A-II-25S, A-II-35S, A-II-50S, C-II-25S, C-II-35S, and C-II-50S are presented in Figure 4.7. The mixtures made with high-fineness cement with low-alumina slag (C-I-25, C-I-35S and C-I-50S) as well as high-alumina slag (C-II-25S, C-II-35S, and C-II-50S) are plotted in Figure 4.8. The high-alkali cement (C-II-25S) and low-alkali cement (B-II-25S) show the effect of alkali content on hydration kinetics. In these figures, the x-axes of the heat evolution were plotted in log scale to clearly distinguish the differences between the curves. The high-alumina GGBFS increased the heat of hydration regardless of cement content or fineness compared to mixtures made with low-alumina GGBFS. The effect of GUb8SF fineness was more pronounced in C-II-25S, C-II-35S, and C-II-50S mixtures compared to counterpart A-II-25S, A-II-35S, and A-II-50S mixtures, as shown in Figure 4.7. It was also observed that increasing the cement alkali content from 0.59% to 0.91% with cement B and C, respectively, increased the heat of hydration and total energy released. To further explain these trends, the values of the hydration peak and the total energy released over a period of 168 hours of hydration were compared for each mixture and are discussed in Section 4.6.3.
Figure 4.7 Effect of high and low fineness cement and different GGBFS II replacements on heat evolution

Figure 4.8 Effect of high and low alumina GGBFS on heat evolution
Figure 4.9 Effect of high and low alkali cements with 25% GGBFS replacement on heat evolution

Figure 4.10 Effect of high and low fineness cements and different GGBFS II replacements on the cumulative heat of hydration
Figure 4.11 Effect of high and low alumina GGBFS on the cumulative heat of hydration

Figure 4.12 Effect of high and low alkali cement on the cumulative heat of hydration
4.5 Strain under uniaxial restraint

The effect of three GGBFS levels (25%, 35% and 50%) on the uniaxial restrained shrinkage test was conducted to evaluate the cracking potential of the HPC mixtures. The uniaxial restrained shrinkage test used a dog-bone test set up (schematic drawings are presented in Figure 3.5a and Figure 3.5b). Each sample was fully restrained (i.e. \( \Delta L = 0 \)), so when tensile strain developed, the load cell was periodically manually adjusted to zero. As the concrete shrank, the internal longitudinal tensile strain in the concrete was measured with an embedded strain gauge placed horizontally at mid depth. The 25% GGBFS mixture exhibited a higher hydration temperature under semi-adiabatic conditions where the sample was surrounded with 50mm thick insulation (as detailed in Figure 3.5b). Figure 4.13 shows the tensile strain from the strain gauge at the fixed end (in the left side where the fixed end is shown in Figure 3.5a). The strain induced in the 50% GGBFS mixture was considerably smaller than in the 25% and 35% GGBFS concretes (see Figure 4.13). The lower strain of the 50% GGBFS content is attributed to the autogenous shrinkage plus thermal dilation being smaller compared to that of the mixtures with low GGBFS levels (25% and 35%). In the 25% GGBFS concrete, a tensile strain was developed immediately as the shrinkage took place. The restraining stress began to increase gradually when the 25% concrete exhibited substantial shrinkage combined with thermal expansion at setting time (i.e. between initial and final setting). The strain rate development of 25% GGBFS mixture was rapid due to high content of silica fume blended cement which accelerated the hydration process and therefore developed higher tensile strain.
The diminished or lower cracking potential of the 50% GGBFS mixture is attributed to lower heat generated due to the hydration. The current finding agrees with previous work conducted by Darquennes et al. (2011), which concluded that binary mixtures of 42% slag cement concrete exhibited a later age of cracking compared to 0% slag concrete; that was attributed to the slower rate of reaction.

### 4.6 Discussion

The results indicate that the temperature rise under semi-adiabatic conditions (which mimic thick concrete elements and large pours in the field) was substantially reduced as the slag replacement was increased. Consequently, this will also reduce the volume change as well as the restrained stress. The
reduction of temperature rise and heat of hydration of mixtures with 35% and 50% slag replacement compared to 25% is in agreement with the previous study by Thomas & Mukherjee (1994). Cement fineness, alkali content of cement, and alumina content of slag, significantly influence thermal strain, hydration temperature, and hydration kinetics.

4.6.1 Effect of GUb8SF cement fineness

As the purpose of this study was to assess the effects of silica fume blended silica fume cement (GUb8SF) fineness on thermal properties (strain, temperature, and heat of hydration), it was critical to select cements with similar mineralogical composition, but of different grinds. The effect of cement fineness on the ultimate thermal strain and temperature rise of semi-adiabatic specimens is substantial. It was evident that concrete with 25% GGBFS replacement exhibited a higher peak temperature and shortened time to peak temperature compared to 35% and 50% GGBFS replacements. The test results agree with previous findings that temperature rise in concrete is mainly driven by the heat generated by the hydration of cement (Tazawa & Miyazawa, 1995; Bentz, Bognacki, Riding, & Villarreal, 2011; Bentz, et al., 2001). The high-fineness cement (cement C) exhibited a higher peak temperature comparing to lower fineness cement (cement A) specimens with same GGBFS replacement levels, as shown in Figure 4.14. High-alumina slag exhibited higher peak temperature and total energy released compared to low-alumina GGBFS. The increase in cement fineness from 582 m²/kg to 787 m²/kg in GUb8SF cement shortened the time to peak by 8 to 26%, as presented in Figure 4.14. The hydration temperature peak values were approximately 9% higher with high-fineness cement at different GGBFS levels compared to counterpart mixtures made with low-fineness cement (see Figure 4.14). The increase in the cement fineness increases the surface area exposed to water which, in return, increases the hydration rate and thus reduces the initial setting time and time to peak temperature.
Figure 4.14 Peak temperature and time to peak temperature of high and low fineness cements versus GGBFS II replacements

Temperature rise which represents the magnitude between the onset temperature (i.e. at initial set) and peak temperature (i.e. at final set) are presented in Figure 4.15. The high-fineness cement gave higher heat rise as well as thermal strain compared to the low-fineness cement (Cement A).
The influence of GU8SF fineness and content on the peak of the heat evolution and total heat energy are presented in Figure 4.16 and Figure 4.17. Each point in the curve is the average of two tests and the corresponding coefficient of variation was less than 5% for all mixes (see Appendix A for the raw data). In all mixtures made with cement A and C, the maximum peak decreased as the level of GGBFS increased compared to the corresponding control mix (i.e., 25% GGBFS). This is due to the acceleration effect of higher cement content and agrees with observations reported in the literature (Meddah & Tagnit-Hamou, 2008). The high-fineness cement (cement C) shows higher heat evolution and total heat energy compared to low-fineness cement (cement A), this is attributed to the higher surface area of cement C which promotes higher reactivity of cement grains (Taylor, 1997).
Figure 4.16 Heat evolution peaks of high and low fineness cements and different GGBFS II replacements

Figure 4.17 Total heat energy peaks of high and low fineness cements and different GGBFS II replacements
4.6.2 Effect of GGBFS alumina content

The rate of temperature rise (i.e. the slope) which represents the magnitude between the onset of temperature (i.e. the initial set) and the peak temperature (i.e. final set) are presented in Figure 4.18. The high-alumina slag mixes have higher rates of heat rise compared to the low-alumina slag mixtures.

![Chart showing average rate of temperature rise of high and low alumina GGBFS for different GGBFS replacements.](chart)

Figure 4.18 Average rate of temperature rise of high and low alumina GGBFS for different replacements
Figure 4.19 shows that reducing alumina content of the slag from 10.63% to 7.63% extended the time to peak up to 30% (i.e. 25 to 36 hours for 50% GGBFS). The hydration temperature peak values were approximately 19% lower in low alumina GGBFS mixtures compared to counterpart mixtures made with high-alumina GGBFS mixtures (see Figure 4.20). Figure 4.20 and Figure 4.21 show the heat evolution and total heat energy peaks of concrete made with high-alumina and low-alumina GGBFS. It is evident that concrete made with low-alumina slag had lower heat of hydration compared with high-alumina slag.

Figure 4.19 Peak temperature and time to peak of high and low alumina GGBFS for different replacements
Figure 4.20 Heat evolution peaks of high-fineness cement with high and low alumina slag and different GGBFS replacements

Figure 4.21 Total heat energy peaks of high-fineness cement with high and low alumina GGBFS and different GGBFS replacements
4.6.3 Effect of alkali content in GUb8SF cement

The reduction in alkali content from 0.91% to 0.59% in the GUb8SF cement extended the time to peak by 43% (i.e. time to peak increased from 18 to 32 hours). The hydration temperature peak values were approximately 10% lower with low-alkali cement made compared to a counterpart mixture made with high alkali cement. The ultimate thermal strain was also substantially reduced at 168 hours for low-alkali cement by 27% compared to high-alkali cement. The influence of GU8SF alkali content on the hydration kinetics peaks is presented in Figure 4.22. The higher the alkali content, the higher the evolved heat and heat energy peaks by up to 30%.

Figure 4.22 Effect of alkali content on GUb8SF cement on heat evolution and total heat energy peaks
4.6.4 Effect of GGBFS replacements on cracking potential of uniaxial restraint specimen

Figure 4.23 presents the rate of tensile strain to cracking versus slag replacements. The rate of tensile strain to cracking is the ultimate strain at age of cracking divided by the age of cracking. It should be mentioned that the 50% mixture did not crack, however, the ultimate strain of 50% mixture, which was used to calculate the rate of tensile strain, was at end of test period. The rate of tensile strain development to time of cracking in the 25% GGBFS replacement was high compared the other mixtures. It is clear a rate of tensile strain was reduced as GGBFS increase as shown in Figure 4.23.

![Figure 4.23](image_url)

Figure 4.23 Effect of GGBFS on the rate of tensile strain to cracking versus GGBFS replacements
4.7 Conclusions

Based on the test results in this chapter, the following conclusions are summarised:

1. The physical effect of GUb8SF cement (i.e. high-fineness vs low-fineness) influences the thermal strain. The high fineness increases thermal strain due to the acceleration in the rate of heat of hydration.

2. The reduction in alkali content from 0.91% to 0.59% in GUb8SF cement extended the time to reach the peak temperature and reduced the ultimate thermal strain and peak hydration temperature as well as heat of hydration.

3. The alumina content of GGBFS influences both thermal strain, peak temperature (i.e. high-alumina slag mixtures exhibited higher thermal strain and hydration temperature compared to low-alumina slag mixtures).

4. Regardless of the type of GGBFS (alumina content) or GUb8SF cement (fineness or alkali content), increasing replacement levels of GGBFS to 35% and 50% reduced overall thermal properties (thermal strain, hydration temperature, and heat of hydration).

5. Increasing the level of GGBFS replacements to 50% reducing the risk of cracking compared to 25% and 35% mixtures.
Chapter 5. Early-Age Autogenous Shrinkage

5.1 Introduction

The early-age autogenous shrinkage (under isothermal conditions) test evaluates the net volume change by deducting the thermal dilation. To remove dilation due to heat of hydration of the cement binder, the test specimen was maintained at constant temperature (20 °C) using water circulation in copper tubes soldered to copper molds. Previously, other researchers (Bjøntegaard & Sellevold, 1998; Bjøntegaard & Sellevold, 2001; Tazawa & Miyazawa, 1997; Lura, 2003) used different techniques by controlling room temperature or by circulating water through tubes to separate the thermal dilation from autogenous shrinkage. Table 5.1 presents the specimen dimensions, presence of insulation, and the test temperature for these previous experiments used to decouple the thermal deformation from autogenous deformation.

The dimensions of the concrete mold used in this thesis is 75 x 75 x 500mm. Figure 5.1 shows typical concrete temperatures (at the center of the cross-section). The temperature was measured and automatically recorded every 5 minutes using a data logger. The test set-up used here (see Figures 3.3a and 3.3b in Chapter 3) was developed and selected to reduce the prism cross-section and increase the number of copper pipe cooling loops in order to reduce the temperature difference between the copper mold walls and the center of the specimen to within 0.5 ±0.25 °C to ensure sufficient decoupling. The effect of different isothermal temperatures (10, 20, and 30 °C) were used to evaluate the temperature-sensitivity of early autogenous shrinkage.

Concretes made with the high-fineness blended cement C and low-fineness blended cement A were evaluated at three GGBFS levels of cement replacements (25% (reference level), 35%, and 50%),
The difference in results between two duplicate samples was less than 3% which indicates good reproducibility of the testing protocol.

Table 5.1 Previous methods to decouple autogenous shrinkage and thermal dilation

<table>
<thead>
<tr>
<th>Reference</th>
<th>Sample Size (w x h x l) mm</th>
<th>Insulation thickness (mm)</th>
<th>Temperature °C</th>
<th>Decoupling technique to maintain Constant Temperature</th>
</tr>
</thead>
<tbody>
<tr>
<td>(Morabito, 2001)</td>
<td>100x100x620</td>
<td>100</td>
<td>20</td>
<td>controlled water circulation</td>
</tr>
<tr>
<td>(Bjøntegaard &amp; Sellevold, 2001)</td>
<td>100x100x500</td>
<td>35</td>
<td>20</td>
<td>controlled water circulation</td>
</tr>
<tr>
<td>(Lura, 2003)</td>
<td>150x150x1000</td>
<td>40</td>
<td>10, 20, 30, 40</td>
<td>controlled water circulation</td>
</tr>
<tr>
<td>(Lee, Lee, Lee, &amp; Kim, 2006)</td>
<td>100x100x400</td>
<td>N.A.</td>
<td>20±1</td>
<td>Controlled room temp.</td>
</tr>
<tr>
<td>(Hashida &amp; Yamazaki, 2002)</td>
<td>100x100x400</td>
<td>N.A.</td>
<td>20</td>
<td>Controlled room temp.</td>
</tr>
<tr>
<td>(Slatnick, Riding, Folliard, Juenger, &amp; Schindler, 2011)</td>
<td>150x150x520</td>
<td>20</td>
<td></td>
<td>controlled water circulation</td>
</tr>
</tbody>
</table>

Figure 5.1 Typical temperature profile of concrete under controlled temperature with water circulation
It was evident from Figure 5.1 that increasing the insulation thickness as well as increasing the number of copper tubes sufficiently increased the efficiency of test chamber by decreasing the thermal gradient at the centre of the cross section. Figure 5.2 presents the autogenous shrinkage test results from both the external linear variable displacement transducers (LVDT) and the embedded strain gauge. It was noted that LVDT measurements show a strong linear relationship with the embedded strain gauge measurements.

Figure 5.2 Autogenous shrinkage test results from both the external LVDT) and the embedded strain gauge

5.2 Research significance

The main objective of this chapter is to develop, build and operate an efficient test procedure which can decouple/or eliminate the thermal dilation from autogenous shrinkage of concrete. The variation
along the cross section would influence the readings, one of the main objective of this test to reduce the differential temperature as low as possible and to be less than 1 °C. The early-age autogenous shrinkage apparatus was then used to evaluate different HPC mixtures containing two blended cements of different fineness and various levels of one type of slag.

5.3 Test results

The autogenous shrinkage tests were conducted at isothermal temperature at early age when hydration temperature rise of concrete is starting, therefore it was essential to eliminate the effect of thermal deformation (which results from the coefficient of thermal expansion multiplied by temperature difference from the initial temperature). For simplicity, the 20 °C temperature for decoupling was selected because in the field, the initial concrete placing temperatures usually range between 10-30 °C. Therefore, the decoupled baseline temperature is 20 °C as shown in Figures 5.3 and 5.4, for the six mixtures (variables include high- and low-fineness cement as well as slag replacement levels of 25, 35, and 50%).

5.3.1 Low-fineness cement A

Figure 5.3 presents the plots of the early-age autogenous strains vs time of the low-fineness mixtures made with different slag levels of cement replacements (at a controlled temperature of 20°C). As shown in Figure 6.3, the black, gray and light lines represent the mixtures with 25%, 35% and 50% GGBFS replacements, respectively. The rate of autogenous strain increases rapidly in concrete with 25% GGBFS replacement (i.e. corresponding to a high level of silica fume blended cement) which supports the argument reported by several researchers (Baalbaki, Aïtcin, Lepage, & Dallaire, 1999; Lura, 2003; Brooks, Cabrera, & Johari, 1998) that higher cement binder contents and low water cement ratios exhibit higher rates of reaction and therefore higher drops of internal relative humidity
(self-desiccation) causing higher capillary tension (see the description in Section 2.3.1). The higher the GGBFS replacement percentage (35% or 50%), the lower the autogenous strain. The A-II-25S exhibits a higher rate of strain development within the first 24 hours (i.e. beyond setting time at an age of 12 hours) in comparison to the higher slag replacement levels 35% and 50% (i.e. setting time of 35% and 50% beyond 16 and 24 hours, respectively). The higher GGBFS replacement slows down the reaction at early ages and therefore reduces the ultimate autogenous strain at 168 hours. Shrinkage values at 168 hours (end of test period) were 50, 160, and 225 microstrain for 25%, 35% and 50% slag replacements, respectively. The delay in onset of of shrinkage was around 12-24 hours, because of using a type D combined water reducing and retarding admixture.

Figure 5.3 Early-age autogenous shrinkage (at 20 °C) versus age (hours) of concrete with low-fineness cement and different high-alumina GGBFS II replacements
5.3.2 High-fineness cement C

Figure 5.4 presents the autogenous shrinkage versus age in hours for mixtures containing the high-fineness cement C with inclusion of different replacement levels of slag over a testing period of 168 hours. Three HPC mixtures C-II-25S, C-II-35S and C-II-50S were tested, corresponding to slag replacement levels of 25%, 35% and 50%, respectively. The strain rate of autogenous shrinkage changed rapidly for the concrete with 25% slag. The C-II-25S and C-II-35S mixtures exhibited a higher strain rate than the mixture C-II-50S. At 168 hours, the measured ultimate strains of C-II-35S and C-II-50S were reduced compared to C-II-25S by 26% and 75%, respectively. Autogenous shrinkage values at 168 hours (end of test period) were 64, 189, and 257 microstrain for mixtures with 25%, 35% and 50% slag replacements, respectively. These values are higher than for companion mixtures made with the low-fineness cement as shown in Section 5.3.1.

![Graph showing autogenous shrinkage versus age](image)

Figure 5.4 Early-age autogenous shrinkage (at 20 °C) versus age (hours) of concrete with high-fineness cement and different high-alumina GGBFS II replacements
5.3.3 Isothermal temperatures (10, 20, 30 °C)

The early-age autogenous shrinkage of low-fineness cement A with inclusion of 35% slag at different curing temperatures (10, 20, 30 °C) are shown in Figure 5.5. Varying the isothermal temperature between 10, 20, 30 °C is to show the effect of different isothermal temperature on autogenous shrink-age behaviour. At 7 days, shrinkage was lowest to highest as temperature increased from 10°C through 20°C and 30°C curing. The rate of shrinkage development at 30°C exhibited more rapid shrinkage development than at the 10 and 20°C curing temperatures. The 30°C curing temperature resulted in earlier development of autogenous shrinkage than at the other curing temperatures. Higher curing temperatures leads to a higher rate of autogenous shrinkage due acceleration of hydration re-
actions (Lura, Guang, & Van Breugel, 2004).

Figure 5.5 Early-age autogenous shrinkage cured at 10, 20, 30 °C temperatures
5.4 Discussion

5.4.1 Effect of cement fineness and slag replacements

The results shown in Figures 5.3 and 5.4 indicate that the strain development under isothermal temperature conditions (at T=20 °C) exhibited increases associated with higher portland cement (69%) and silica fume (6%) content (i.e. at 25% GGBFS). This finding is supported by others (Hashida & Yamazaki, 2002). Mixtures with higher cement clinker contents produce hydrates more quickly and thus exhibit a higher autogenous shrinkage (or chemical shrinkage rate) at early-ages. The effect of fineness of GGBFS on autogenous shrinkage of concrete is shown in Fig. 5.6. The figure shows a clear trend of increasing autogenous shrinkage when the fineness of GGBFS increases. It was observed that the GGBFS fineness had a significant effect on the rate of autogenous shrinkage. As expected, the concretes with the higher-fineness blended cement exhibited greater autogenous strains.

The results are in agreement with previous studies (Bentz, et al., 2001; Bjøntegaard & Sellevold, 2001), where higher silica fume contents (when 25% GGBFS was used), resulted in the greatest autogenous deformation. Several authors (Baroghel-Bouny, Mounanga, Loukili, & Khelidj, 2004; Chen, 2013; Darquennes, Staquet, & Espion, 2011) claim that binder composition influenced the early-age hydration rate. Increasing the content of slag or limestone filler induced a dilution effect, causing a reduction in the amounts of hydrates formed at early age, in comparison with portland cement. The reductions of autogenous shrinkage with low-fineness cement compared to high-fineness cement at slag replacement levels of 25%, 35% and 50% were 13%, 22%, and 38%, respectively (see Figure 5.6).
Figure 5.6 Effect of slag replacements and cement fineness on ultimate autogenous shrinkage strain at age of 7 days (168 hours)

Figure 5.6 indicates that the autogenous shrinkage of concrete made with the high-fineness cement was greater compared to low-fineness cement. This effect can also be explained by the surface area of the SF-blended cement particles. The finer cement particles have a larger surface area and hence a faster rate of reaction. It has been shown that high-fineness cement increases the hydration process (see Section 4.4). As there is less free water, self-desiccation in the cementitious matrix results in a greater capillary pore pressure. This leads to larger autogenous shrinkage of the concrete with 25% slag content in conjunction with high content of portland cement and silica fume compared to smaller shrinkage observed at higher slag replacements (35% and 50%). It should be mentioned that (Meddah & Tagnit-Hamou, 2008) observed finer pores with the addition of silica fume plus portland cement paste compared to non-silica fume cement paste. Therefore, the capillary pore pressure that develops
in the binder matrix is much greater as the radius of the curvature of the meniscus in the capillaries is less in fine pores. Wittmann described the mechanisms of the capillary pore pressure as inversely proportional to the radius of the capillary pores.

### 5.4.2 Effect of isothermal temperature

Increasing the curing temperature caused an increase in the rate of autogenous shrinkage at the end of the test period as shown in Figure 5.7. A similar trend was observed by others (Tazawa et al. 1995, Bjøntegaard 1999, Lura 2003). At all the tested temperatures, concrete cured at 30°C showed higher shrinkage at 168 hours than companion concretes cured at 20°C. The phenomenon is attributed to the faster reaction with 30°C resulting in production of smaller pores and denser structures (Jensen & Hansen, 1999).

![Figure 5.7 Autogenous shrinkage strains at age of 168 hours versus curing temperatures](image)

Figure 5.7 Autogenous shrinkage strains at age of 168 hours versus curing temperatures
5.5 Conclusions

1. Embedded strain gauges exhibited a good correlation with the external LVDT readings.

2. The temperature variation between the copper mold of the test apparatus to the centre of the cross-section was less than 0.75 °C.

3. Autogenous strains of four HPC mixtures were measured at constant temperature. Concrete containing 25% GGBFS replacement showed higher development of strain at early-age and that is attributed to the higher (6%) silica fume and portland cement (69%) contents.

4. Higher slag replacement (50%) and lower silica fume content (4%) reduced the overall ultimate autogenous strain by 40-50%.

5. Increasing curing temperature exhibited higher shrinkage strains.

6. Low-fineness cement showed reduction in autogenous shrinkage strains ranged from 13 to 38%.
Chapter 6. The Mechanical, Transport, Salt Scaling Properties

6.1 Introduction

In this chapter the mechanical properties (compressive strength, splitting strength, and modulus of elasticity), transport properties (rapid chloride permeability test, bulk electrical resistivity, and sorptivity), as well as salt-scaling resistance of HPC concrete mixtures are evaluated. The test variables included: the source of cement (A, B, C), the level of GGBFS replacement of cements (25% (reference), 35%, 50%), the source of GGBFS (I and II). The Ontario Ministry of Transportation (MTO) originally specified HPC with 28-day compressive strength of at least 50MPa and a maximum RCPT of 1000 coulombs. All the mixtures met these MTO requirements. It should be mentioned that in 2015, the MTO lowered the 28-day HPC strength requirement to 40 MPa. The salt scaling tests were conducted using concrete mixtures made with high-fineness cement C and low-fineness cement A at three slag levels (25%, 35% and 50%). The mixture identification and supplementary cementitious material (SCM) replacements levels are provided in Table 6.1. Additionally, SCM replacements levels and types of testing (i.e. mechanical, transport, salt-scaling) are presented in Table 6.1.
Table 6.1 Test matrix of mechanical and durability tests

<table>
<thead>
<tr>
<th>Mix ID</th>
<th>Cementitious materials</th>
<th>Mechanical</th>
<th>Transport</th>
<th>Scaling</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>GGBFS</td>
<td>GU8SF</td>
<td>Compressive strength</td>
<td>Splitting strength</td>
</tr>
<tr>
<td>A-II-25S</td>
<td>25%</td>
<td>69%</td>
<td>6%</td>
<td>x</td>
</tr>
<tr>
<td>A-II-35S</td>
<td>35%</td>
<td>59.8%</td>
<td>5.2%</td>
<td>x</td>
</tr>
<tr>
<td>A-II-50S</td>
<td>50%</td>
<td>46%</td>
<td>4%</td>
<td>x</td>
</tr>
<tr>
<td>C-II-25S</td>
<td>25%</td>
<td>69%</td>
<td>6%</td>
<td>x</td>
</tr>
<tr>
<td>C-II-35S</td>
<td>35%</td>
<td>59.8%</td>
<td>5.2%</td>
<td>x</td>
</tr>
<tr>
<td>C-II-50S</td>
<td>50%</td>
<td>46%</td>
<td>4%</td>
<td>x</td>
</tr>
<tr>
<td>C-I-25S</td>
<td>25%</td>
<td>69%</td>
<td>6%</td>
<td>x</td>
</tr>
<tr>
<td>C-I-35S</td>
<td>35%</td>
<td>59.8%</td>
<td>5.2%</td>
<td>x</td>
</tr>
<tr>
<td>C-I-50S</td>
<td>50%</td>
<td>46%</td>
<td>4%</td>
<td>x</td>
</tr>
<tr>
<td>C-II-25S (25mm agg.)</td>
<td>25%</td>
<td>69%</td>
<td>6%</td>
<td>x</td>
</tr>
<tr>
<td>B-II-25S</td>
<td>25%</td>
<td>69%</td>
<td>6%</td>
<td>x</td>
</tr>
</tbody>
</table>

*GU8SF consists of General Use cement (GU) interground with silica fume (SF). Both GU and SF represent the actual cementitious binder percentages.
6.2 Research significance and objective

In this chapter the durability of concretes is evaluated with proposed early-age shrinkage mitigation measures (i.e. high GGBFS replacements, low fineness cement, low alumina GGBFS). The evaluation (i.e. mechanical, transport, and durability tests) is to ensure that high GGBFS levels will satisfy the MTO standard limits (i.e. RCPT <1000 coulombs, compressive strength >50 MPa, and salt-scaling mass loss<0.8 kg/m²).

6.3 Mechanical properties

The compressive strength, splitting tensile strength, and modulus of elasticity of all mixtures are presented and plotted in the following subsections. The day after casting, the specimens for each mix were demolded and stored in a moist curing room at 23±2 °C. All the mixtures met the 50 MPa compressive strength target by 28 days.

6.3.1 Compressive strength test

Testing was performed on 100mm x 200mm concrete cylinders according to ASTM C39-10. The cylinder ends were ground flat and loading was applied at a rate of 2 kN/s until failure. Compressive strength tests were performed on three specimens at 3, 7, 28 and 56 days. At early age, typical lower values in compressive strength were observed with 50% slag replacement. The raw data and statistical analysis of compressive strengths are presented in Appendix A.

The compressive strengths of low-fineness cement mixtures made and high-alumina GGBFS (A-II-25S, A-II-35S, and A-II-50S) are presented in Figure 6.1. The coefficients of variation were below 6%. At early age, the use of 50% GGBFS decreased the compressive strength up to 11%, compared
to the control mixture with 25% slag replacement. At 56 days, the strength of all mixes was similar (75 to 78 MPa).

Figure 6.1 Compressive strength of A-II-25S, A-II-35S, and A-II-50S mixtures

Figure 6.2 presents the compressive strength of high-fineness cement mixtures with high-alumina GGBFS (C-II-25S, C-II-35S and C-II-50S). Figure 6.3 presents the compressive strength of mixtures with high-fineness cement together with low-alumina GGBFS (C-I-25S, C-I-35S, and C-I-50S). The coefficients of variation were below 5.5% for both high and low alumina mixtures. The addition of 50% GGBFS decreased the 3 and 7 day compressive strengths by up to 11% compared to the control mixture made with 25% slag replacement. At 28 days, a slight increase was observed with 35% slag replacement mixture compare to the 25% control. At 56 days, the compressive strengths of high-alumina slag mixtures ranged between 79 to 87 MPa, and for the low-alumina slag mixtures, 56 day
compressive strengths ranged between 77-79 MPa. The coefficients of variation were below 5%. The addition of 50% GGBFS decreased the early-age (up to 7 days) compressive strengths by up to 20%, compared to the control mixture with 25% slag replacement. At later ages, a slight increase was observed with 35% slag replacement compared to 25% slag replacement. At 56 days, the strength of all mixes ranged between 79 to 83 MPa.

![Figure 6.2 Compressive strength of C-II-25S, C-II-35S, and C-II-50S mixtures](image)

Figure 6.2 Compressive strength of C-II-25S, C-II-35S, and C-II-50S mixtures
As expected, use of high-fineness cement increased the compressive strength at early ages. Higher cement fineness and high slag alumina content influenced the hydration rate and increased the volume of hydration products, thus reducing the porosity and increasing the strength (the heat of hydration was reported in Chapter 4). The fine silica-fume particles of GUb8SF cement filled the voids between the larger particles, which reduced the porosity and increased the strength. The chemical effect of GGBFS compositions (i.e., the difference between high-alumina GGBFS II and low-alumina GGBFS I) increased the strength. Increasing the slag content did not seem to affect the compressive strength at late ages (28 days and 56 days). At early ages though, it would be expected that with increasing slag content, the concrete would experience lower compressive strengths (i.e. 3 and 7 days) and a higher gain in compressive strength over time (at 28 and 56 days).
6.3.2 Splitting tensile strength

Testing was performed on 100mm x 200mm concrete cylinders according to ASTM C496. Splitting tests were performed on three cylinders at each of 3, 7, 28 days. At early-age, typical reductions in splitting strength were observed with 35% and 50% slag replacement. The raw data and statistical analysis of splitting strengths are presented in Appendix A. The splitting strength results of low-fineness cement mixtures made and high-alumina GGBFS are presented in Figure 6.4. The coefficients of variation were below 6.5%. At 3 and 7 days, the 50% GGBFS decreased the splitting strength up to 18% compared to 25% GGBFS. Figure 6.5 and Figure 6.6 present the test results for high-fineness cement mixtures made with high-alumina and low-alumina GGBFS, respectively. The splitting strengths of high-alumina slag mixtures was higher than for low-alumina GGBFS by 30-40% at 3 and 7 days, but at 28 days the percentage differences were reduced to between 10-20%.

Figure 6.4 Splitting strength of A-II-25S, A-II-35S, and A-II-50S mixtures
Figure 6.5 Splitting strength of C-II-25S, C-II-35S, and C-II-50S mixtures
6.3.3 Elastic modulus

All concretes were tested in accordance with ASTM C469 on moist cured specimens at the ages of 3, 7, and 28 days. Cylinder ends were ground at the age of 3 days, just prior to the first age of test. An initial loading (40% of the ultimate load) was applied to each specimen to ensure the compressometer was installed correctly and that gauges were operating properly. Two test cycles were performed on each specimen. During each cycle, four stress and strain readings were recorded with the maximum stress being at 40% of the ultimate.
The results of the elastic modulus of low-fineness cement and high-fineness cement mixtures with different slag replacements are presented in Figure 6.7 and Figure 6.8, respectively. Each bar represents the average of three tests and the corresponding coefficient of variation was less than 6% for all mixtures. Concretes made with 25% slag replacement showed higher elastic modulus than counterpart mixtures made with 35% and 50% slag replacements at age of 3 and 7 days. The elastic modulus in high-fineness cement mixtures was higher by 7-34% compared to low-fineness cement mixtures.

Figure 6.7 Elastic modulus of A-II-25S, A-II-35S, and A-II-50S mixtures
The elastic modulus results for low-alumina slag mixtures are presented in Figure 6.9. The elastic modulus of low-alumina mixtures were lower in by 3-11% than counterpart mixtures made with high-alumina slag at 3, 7 and 28 days. At 28 days, the elastic modulus values were similar for all slag replacements.
The superior early-age mechanical properties were attributed to the combined influence of accelerated of cement and the supplementary cementitious materials hydration (GGBFS and SF) especially silica fume (Johari, Brooks, Kabir, & Rivard, 2011). It is worth noting to that the higher fineness SF particles provide nucleation sites for cement hydrations (i.e. calcium silicate hydrate, C–S–H, and Ca(OH)$_2$). The SF particles also act as microfiller, densifying the transition zone, thus, enhancing the binder-aggregate bond and increasing the concrete strength (Detwiler & Mehta, 1989; Goldman & Bentur, 1993).

Figure 6.9 Elastic modulus of C-I-25S, C-I-35S, and C-I-50S mixtures
6.4 Transport properties

Resistance to fluid transport is an important durability index for concrete. One of the advantages of high performance concrete is higher resistance to fluid transport. Progress of hydration as concrete aged (cement hydration or secondary hydration of added SCMs), results in more produced C-S-H. Therefore, cement paste voids are filled with more hydration products and porosity and pore connectivity is reduced (Neville, 1995).

6.4.1 Rapid chloride permeability test

The rapid chloride permeability tests (RCPT) were performed to ensure the HPC mixtures met the MTO durability requirements. Four 50-mm thick specimens were saw cut from the center of two 100×200 mm moist cured cylinders and tested at ages of 28 and 56 days. Specimens were vacuum saturated one day prior to testing. Specimens and water were placed under vacuum independently for three hours, and the specimens were then submerged in the de-aired water while under vacuum and the vacuum was maintained for another hour. The MTO specifies a maximum rapid chloride permeability of 1,000 coulombs (Low) at 28 to 32 days of age for HPC, and the four HPC mixtures reached RCPT results well below this limit at the age of 28 days. Since, for non-MTO applications, concrete used for bridge decks also has to meet either exposure class C-1 or C-XL (CSA Standard A23.1, 2014). Table 6.2 presents the RCPT charge passed corresponding the chloride ion penetrability. Results from the rapid chloride penetration test are shown in Figures 6.10, 6.11, and 6.12.
Table 6.2 RCPT charge passed classification (ASTM C1202)

<table>
<thead>
<tr>
<th>Charge Passed (Coulomb)</th>
<th>Chloride Ion Penetrability</th>
</tr>
</thead>
<tbody>
<tr>
<td>&gt;4000</td>
<td>High</td>
</tr>
<tr>
<td>2000</td>
<td>Moderate</td>
</tr>
<tr>
<td>1000</td>
<td>Low</td>
</tr>
<tr>
<td>100</td>
<td>Very low</td>
</tr>
<tr>
<td>&lt;100</td>
<td>Negligible</td>
</tr>
</tbody>
</table>

Figure 6.10 RCPT results for low-fineness cement with different slag levels
Figure 6.11 RCPT results for high-fineness cement with different levels of high-alumina slag
6.4.2 Bulk electrical resistivity test

The bulk electrical resistivity of concrete is not yet a CSA or ASTM standard test. It has been used by many researchers (Charmchi, 2015; Shahroodi, 2010) as an index of fluid penetration resistance of concrete (i.e. higher bulk electrical resistance indicates lower penetration). The bulk electrical resistivity is measured by placing two electrodes on top and bottom surfaces of 100 x 200 mm cylinders. The ratio of the potential to the current passing is the concrete’s electrical resistance according to Ohm’s law (Shahroodi, 2010; Charmchi, 2015). Electrical resistivity, $\rho$, can be calculated from the Equation 6.1:

$$\rho = \frac{\pi d^2}{4 s L} \frac{P}{l}$$  \hspace{1cm} \text{Eq. 6.1}
where, d is the section diameter (mm), L is the length of the cylinder (mm), I is the current passed through the specimen (A), and P is the voltage applied (V).

Figures 6.13 and 6.14 present the bulk resistivity of concrete at ages of 28 and 56 days for concretes made with low-fineness cement and high-fineness cement with different high-alumina slag replacements (25% (reference), 35%, and 50%). Figure 6.15 presents resistance for the high-fineness cement with low-alumina slag mixtures. Each result represents the average of 4 tests on cylinders. The typical trends were observed for low-fineness and high fineness with same where 25% slag replacements to have higher resistivity compared to 35% and 50% at age of 28 days. At later ages, however, the high slag replacements (i.e. 35% and 50%) exhibited higher resistivity and therefore lower penetration. Gudimettla & Crawford (2015) concluded that the reduction in permeability and the increase of bulk resistivity at later ages is due to continued hydration. The high-fineness cement concretes exhibited increases in bulk resistivity from 28 to 56 days between 5 and 30%, however, for low-fineness cement mixtures the increase ranged between 4 and 20%.

Low-alumina slag mixtures at 56 days exhibited increased bulk resistivity, however, the difference in bulk resistivity between the slag replacements (i.e. 25%, 35%, and 50%) were small at 56 days. The bulk resistivity of low-alumina slag mixes were consistently lower than that of high-alumina slag mixes by approximately 6-25%.
Figure 6.13 Bulk resistivity of concretes with low-fineness cement with different high-alumina slag levels
Figure 6.14 Bulk resistivity of concretes with high-fineness cement with different high-alumina slag levels
Figure 6.15 Bulk resistivity of concretes with high-fineness cement with different low-alumina slag levels

6.4.3 Water sorptivity test

Fluid penetration is the fundamental cause of poor durability (e.g. saturation can cause freeze-thaw damage/ or chloride ingress leads to steel reinforcement corrosion). The rate of water absorption is an important measure of hardened concrete durability (i.e. lower rate of absorption leads to better durability) since absorption is a relatively rapid process. Moreover, studying the water absorption of concrete surfaces is essential for assessing durability since values are different from the rate of absorption of a sample taken from the interior because the exterior surface is often subjected to less than perfect curing, and is more exposed to potentially adverse conditions (ASTM C1585, 2013). The rate of absorption, $i$, is calculated using the sorptivity equation (Shahroodi, 2010):
\[ i = \frac{\Delta \text{mass}}{A \cdot \rho} \]  \hspace{1cm} \text{Eq. 6.2}

where, \( i \) is cumulative water absorption (mm), \( A \) is cross-section area of the specimen which is in contact with water (mm\(^2\)), and \( \rho \) is the density of water (g/mm\(^3\)).

The initial and secondary water sorptivity values of concretes made with high-fineness cement (C-II-25S, C-II-35S and C-II-50S) and low-fineness cement mixtures (A-II-25S, A-II-35S and A-II-50S) are shown in Table 6.3. The effect of using high-fineness of cement C versus mixes with low-fineness cement A was obvious on initial and secondary sorptivity of different slag replacements (see Table 6.3). It was evident that longer curing, 56 day results showed reductions in sorptivity for both low-fineness and high-fineness cement mixtures.

<table>
<thead>
<tr>
<th></th>
<th>28 days</th>
<th>56 days</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Initial slope</td>
<td>Secondary slope</td>
</tr>
<tr>
<td>C-II-25S</td>
<td>9</td>
<td>2.3</td>
</tr>
<tr>
<td>C-II-35S</td>
<td>8.5</td>
<td>3.5</td>
</tr>
<tr>
<td>C-II-50S</td>
<td>10</td>
<td>4.1</td>
</tr>
<tr>
<td>A-II-25S</td>
<td>11.5</td>
<td>3.3</td>
</tr>
<tr>
<td>A-II-35S</td>
<td>11.9</td>
<td>3.9</td>
</tr>
<tr>
<td>A-II-50S</td>
<td>12.4</td>
<td>4.3</td>
</tr>
</tbody>
</table>

Table 6.3 Average water sorptivity values for concrete (10\(^{-4}\) mm/sec\(^{1/2}\))
6.5 Salt scaling test

The CSA A23.2-22C test results are compared to a modified version of the ASTM C672 scaling resistance test designated as MTO LS-412. Based on the poor scaling performance in the LS-412 test, the Ontario Ministry of Transport MTO places limits on supplementary cementing material contents for bridge decks of 25% for slag replacements, 10% for fly ash, or 25% total for any combination of the two. In the LS-412 test method, the material scaling off the surface of a concrete specimen is collected after every five cycles of freezing and thawing, dried and weighed, and reported as cumulative mass loss in kg/m² after 50 cycles of freezing and thawing. The MTO LS-412 standard limits the maximum mass loss to 0.80 kg/m². Table 6.4 presents the major differences between MTO LS-412 and CSA 23.3-22C salt scaling test procedures.

Table 6.4 Major difference between MTO LS-412 and CSA salt scaling test procedures.

<table>
<thead>
<tr>
<th>#</th>
<th>MTO LS-412</th>
<th>CSA A23.2 – 22C</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Start freeze thawing cycle</td>
<td>immediately after ponding the saline solution (3%NaCl)</td>
</tr>
<tr>
<td></td>
<td>Remarks</td>
<td>7 days after ponding the saline solution (3%NaCl)</td>
</tr>
<tr>
<td></td>
<td>Does not allow for sufficient chloride penetration prior to freezing</td>
<td>The solution penetrates prior to freezing</td>
</tr>
<tr>
<td>2</td>
<td>Size, number of specimens</td>
<td>Rectangular prisms measuring 300mm<em>200mm</em>75mm (Surface area 0.06m²) : 2 specimens per mixture</td>
</tr>
<tr>
<td>3</td>
<td>Curing of specimens</td>
<td>14 days at 100% RH followed by 14 days at 50% ± 5% RH</td>
</tr>
<tr>
<td>4</td>
<td>Freezing and thawing cycles</td>
<td>50 cycles of 24hours (Freezing at -18 ± 3°C for 16 ± 1h; followed by Thawing at 23°C ± 2°C for 8 ± 1h; 45-55%RH)</td>
</tr>
<tr>
<td>5</td>
<td>Mass loss measurements</td>
<td>After every 5 cycles</td>
</tr>
</tbody>
</table>

Scaling tests were performed on the low-fineness concrete mixtures A-II-25S, A-II-35S, A-II-50S, and high-fineness concrete mixtures C-II-25S, C-II-35S, C-II-50S. The tests were performed using
the CSA testing procedure. Additionally, A-II-25S, A-II-35S, A-II-50S mixtures were tested in conformance with the MTO LS-412 testing procedure. The fresh air contents of the mixtures are listed in Table 6.5. The targeted air content was between 5-8% which satisfies the MTO requirements.

<table>
<thead>
<tr>
<th>Mix ID</th>
<th>air content</th>
</tr>
</thead>
<tbody>
<tr>
<td>C-II-25S</td>
<td>6.2%</td>
</tr>
<tr>
<td>C-II-35S</td>
<td>4.9%</td>
</tr>
<tr>
<td>C-II-50S</td>
<td>5.3%</td>
</tr>
<tr>
<td>A-II-25S</td>
<td>7.1%</td>
</tr>
<tr>
<td>A-II-35S</td>
<td>5.9%</td>
</tr>
<tr>
<td>A-II-50S</td>
<td>4.6%</td>
</tr>
</tbody>
</table>

Figure 6.16 presents the scaling results for both the CSA procedure and the MTO procedure. The test results suggest that using higher fineness of GU8SF cement improves the salt scaling resistance, however, low-fineness cement and increases in the proportion of GGBFS have a negative impact on scaling resistance.

Figure 6.17 presents cumulative scaling mass loss as after 50 freeze-thaw cycles for different slag levels (25%, 35% and 50% GGBFS) and corresponding GUb8SF cement fineness (low and high). Increasing the cement fineness from 582 to 787 m²/kg resulted in the most significant reduction in scaling by up to 45%. The 25% and 35% slag content mixtures, regardless of cement fineness, met the MTO mass loss limits. The 50% slag mixture made with high-fineness cement performed better in scaling than with the low-fineness cement. Further discussion is provided in Section 6.6.3.
Figure 6.16 Mass loss after 50 cycles for concretes made with low-fineness cement and different replacements levels of high-alumina GGBFS II using two different test methods

Figure 6.17 Mass loss after 50 cycles for concretes with low and high fineness cements and different replacements levels of high-alumina GGBFS II (CSAA23.2-22C)
6.6 Discussion

6.6.1 Mechanical properties

It is obvious that high slag replacement levels reduce the early-age mechanical properties up to 7 days (compressive strength, splitting strength, elastic modulus), but enhance the later-age properties beyond 28 days. The high-fineness blended cement mixtures showed higher early strength compared to low-fineness cement. The high-alumina slag improved both early-age and later-age strengths compared to low-alumina slag. The highest early-age splitting tensile strength was observed for 25% GGBFS concretes (also with 6%SF content) followed by those prepared with 35%, and 50% GGBFS with corresponding 5.2%SF and 4%SF contents, respectively.

The effects of cement fineness on mechanical properties (compressive, splitting, and elastic modulus) were pronounced. The high-fineness cement showed higher compressive strength by 10-14%. The slag alumina content and replacement levels was also influencing the mechanical properties with an increase ranging between 5-16%. The effect of cement fineness and slag alumina content on the thermal kinetics properties are substantial which explains why the high fineness cement and high-alumina slag mixture exhibited high mechanical properties at early ages. As explained in chapter 4, it was evident that concretes with 25% GGBFS replacement produced high-peak temperatures compared to 35% and 50% GGBFS replacements.

6.6.2 Transport properties

Higher slag replacements improved the resistance to chloride penetration as the concrete aged. The RCPT results show a substantial influence of blended cement fineness; the high-fineness values were lower by 7-33%. The high-alumina slag mixtures exhibited improved resistance to chloride penetration by 16-21% at 28 days, however, the low-alumina slag mixtures at 56 days showed reductions in
RCPT in comparison to high-alumina slag mixtures by 3-15%. The bulk resistivity was higher by 3-13% for high-fineness cement mixtures in comparison to lower fineness cement regardless of the slag level. The high-alumina slag mixture showed a higher resistivity compare to low-alumina slag ranging between 6-25%.

### 6.6.3 Salt-scaling and sorptivity

Use of the high-fineness cement improved concrete properties, lowered fluid penetration and also improved the salt-scaling resistance. It was noted that the high-fineness cement mixtures exhibited a lower sorptivity compared to low-fineness cement concretes at 28 days (i.e. when scaling test specimens were exposed to sodium chloride ponding). It was interesting that there was a very good linear relationship between scaling mass loss in the CSA 23.2-22C test method versus initial sorptivity as shown in Figure 6.18. The good correlation between initial sorptivity and scaling mass loss emphasizes that fast fluid penetration into concretes having coarser pores results in higher scaling and, conversely, a lower rate of fluid penetration promotes less scaling.
The current study shows that as slag replacement levels increase, the sorptivity increases marginally. It was interesting that as slag replacements increased, the cumulative mass loss increased for both the low-fineness blended cement as well as for high fineness cement concretes. The differences in scaling mass loss between high-fineness and low-fineness cement mixtures lie between 10-45%. The sorptivity of high-fineness cement concretes was less than those with low-fineness cement by approximately 21%. It should mentioned that Hooton & Vassilev (2012) conducted a study on the effect of slag replacement levels (0%, 20%, 35%, and 50%) on binary mixtures made with portland cement. The test method was used was ASTM C672 (similar to MTO LS-412). Their test results are presented in Figure 6.19. Their findings with mass loss coincide with the current findings with ternary mixtures, as shown in Figure 6.20. However, Hooton & Vassilev reported that use of a high-fineness slag exhibited a substantial reduction in scaling resistance at cement replacement levels of 35% and 50%.

Figure 6.18 Linear relationship between scaling mass loss (CSA test method) and initial sorptivity
Figure 6.19 Salt scaling test results of two slag fineness replacements data extracted from (Hooton & Vassilev, 2012)

Figure 6.20 Salt scaling test results of two blended GUb8SF cements of different fineness at three slag replacements levels
Figure 6.21 presents the difference between the scaling results from two scaling test methods where the major difference is the period of ponding of the sodium chloride solution prior to commencement of freezing cycles. As stated earlier, the MTO LS-412 does not provide a period of ponding prior to scaling freeze and thaw cycles whereas the CSA A23.2-22C allows 7 days of ponding. The MTO test seems to be more aggressive by not allowing capillary pores to be penetrated by NaCl solution. The freezing temperature of deicer salt solutions is lower than for non-saline water. The rate of absorption of NaCl is less than for non-saline water, therefore the saline water needs more time to sufficiently penetrate. This could be attributed to the osmotic effect causing a flow of solution from the non-saline water (weak solution) to the saline water (strong solution). Thus saline water on the chloride contaminated top surface of concrete is drawn into the pores where there is a lower density solution (non-saline water). It should be mentioned that McCarter et al. (1992) observed that sorptivity of chloride contaminated water into concrete is lower than non-chloride contaminated water over a period of 24 hours. The non-chloride contaminated water advanced to a depth of between 40 to 50 mm from the surface; however, the chloride solution only penetrated up to 20 to 25 mm over a period of 24 hours. Their argument that the chloride front moves into the concrete at a slightly slower rate than the water in which the chlorides were dissolved, suggesting that chloride binding affects the amount of chloride ions being transported by moisture flow. This shows the significant effect of chloride binding when penetration into concrete occurs by absorption.
Figure 6.21 Salt scaling test results from two scaling test methods with three GGBFS replacement levels

The salt scaling results of concretes made with two different cement fineness and three GGBFS replacements (25, 35, and 50%) concrete revealed some significant differences. The higher the cement fineness, the lower the mass loss and that was attributed to lower fluid transport properties (RCPT, sorptivity) as well as greater tensile strength compared to the lower fineness cement. The slag replacement significantly influences the scaling mass loss. Increasing slag replacement to 50% is potentially an alternative to the MTO slag limit (i.e. MTO maximum slag replacement limit is 25%). For example, concrete placed in summer where concrete will cure for over four months and then will achieve greater mechanical and transport properties prior to late season when freezing starts to occur in Ontario. However, a one month curing regime for salt scaling maybe aggressive for high slag concrete (i.e. above MTO slag replacement limit).
It is seen from Figure 6.18 that salt scaling resistance is strongly correlated with sorptivity. The current results agree with a similar relationship reported by (Gagné et al. 2011). Another study (Liu and Hansen, 2014) suggested that the pore structure of the paste hydration products plays a decisive role with respect to salt scaling. Concrete sorptivity is an indicator of capillary suction which has been proposed to be the primary transport mechanism under sub-freezing conditions in promoting ice growth in the surface-region capillary pores that eventually leads to scaling.

6.7 Conclusions

1. The use of high-fineness GUb8SF cement enhances the mechanical properties as well as transport properties of concrete. The mechanical properties of low-fineness cement concretes were reduced marginally at early-ages but increased at later ages.

2. The results have shown that the utilization of high slag replacements contribute to the development of long-term strength and durability of the HPC mixtures regardless of the chemical or physical properties of the cement binder.

3. The mixtures with 25% and 35% slag replacements display similar electrical resistivity. Electrical resistivity and chloride penetration of HPC appear to be less sensitive to cement fineness but are directly proportional to level of slag replacement especially at 28 days. This supports the argument that high-slag replacements can result in a significant reduction in transport properties at ages beyond 28 days.

4. Salt-scaling is strongly related to initial sorptivity which supports the argument that reducing the penetrability of concrete leads to a reduction in salt scaling.
Chapter 7. Late age Autogenous and Drying Shrinkage

7.1 Introduction

This chapter presents the test results of long-term length change (beyond 7 days) due to autogenous and drying shrinkage to evaluate the impact of different crack mitigation strategies (increase GGBFS replacements, use of low-cement fineness and presence of shrinkage reducing admixture (SRA)). Increasing the level of replacement by GGBFS in ternary mixtures with silica fume blended cements decreased both the drying and autogenous shrinkage. The method of mitigation used in this study (high replacements of GGBFS) was to reduce the total heat generated by the ternary cement binder (OPC/SF/GGBFS), and therefore, decrease both early-age thermal and autogenous length changes (as explained in Chapters 4 and 5). Table 7.1 presents several methodologies which were used by other researchers to evaluate autogenous shrinkage using sealed samples.
Table 7.1 Previous by published test methods for sealed shrinkage (or autogenous shrinkage)

<table>
<thead>
<tr>
<th>Author</th>
<th>Size (w x h x l) mm</th>
<th>Type of sealing</th>
<th>Max. time days</th>
</tr>
</thead>
<tbody>
<tr>
<td>(Khayat &amp; Long, 2010)</td>
<td>75x75x300</td>
<td>Adhesive aluminum sheet</td>
<td>56</td>
</tr>
<tr>
<td>(Rols, Mbessa, Ambroise, &amp; Pera, 1999)</td>
<td>70x70x280</td>
<td>Plastic film</td>
<td>91</td>
</tr>
<tr>
<td>(Brooks, Cabrera, &amp; Johari, 1998)</td>
<td>75x75x286</td>
<td>Foil film</td>
<td>80</td>
</tr>
<tr>
<td>(Sakata &amp; Shimomura, 2004)</td>
<td>100x100x400</td>
<td>Plastic film</td>
<td>180</td>
</tr>
<tr>
<td>(Lim &amp; Wee, 2000)</td>
<td>100x100x400</td>
<td>Adhesive aluminum tapes</td>
<td>91</td>
</tr>
<tr>
<td>(Hedlund &amp; Westman, 1997)</td>
<td>100x100x400</td>
<td>Plastic foil film</td>
<td>180</td>
</tr>
<tr>
<td>(Wiens, Meng, Schroeder, &amp; Schiessl, 1997)</td>
<td>100 dia. x 300 height (cylinder)</td>
<td>aluminium foil</td>
<td>180</td>
</tr>
<tr>
<td>(D’Ambrosia, 2011)</td>
<td>75x75x250</td>
<td>Foil film</td>
<td>91</td>
</tr>
</tbody>
</table>

7.2 Research significance

The current MTO provision uses the ASTM C157 test set-up, but using a shorter 7 days moist curing period compared to 28 days in ASTM C157. However, the actual length of HPC concrete is a combination of autogenous shrinkage and drying shrinkage. Therefore, it is important to accurately measure the autogenous shrinkage and drying shrinkage separately. This study involve mixtures containing
similar total binder content and water-cement ratio. Free shrinkage due to autogenous and drying shrinkage is crucial to account the total strain development over time. This chapter established experimental data of both types autogenous and drying shrinkage. The conclusion of these mitigation strategies (i.e. increase slag replacements, use of low fineness slag) provides a potential approach that can be used to limit cracking resulting from drying and autogenous shrinkage.

### 7.3 Long-term length change

The later-age length change readings for both autogenous and drying shrinkage were recorded at age of 7, 14, 28, 56, 91, 180, and 360 days. Typical trends for strain in sealed (autogenous) and un-sealed (drying) concrete prisms are presented in Figure 7.1. As shown in Fig. 7.1, the drying shrinkage exhibited relatively higher strains than autogenous shrinkage. Several authors (Aıtcin, 2003; Brooks, Cabrera, & Johari, 1998) mentioned that autogenous shrinkage should be taken in consideration when evaluating the total shrinkage strain of high performance concrete. Therefore, the drying and autogenous shrinkage should be evaluated separately to obtain an accurate estimate of total shrinkage at a desired age (i.e. most standards such as ASTM C157, MTO LS-435 or CSA A23.2 specify an age of 28 days).
Figure 7.1 Typical behaviour of drying and autogenous shrinkage (made with low-fineness cement and 25% GGBFS II)

7.3.1 Autogenous Shrinkage

The long-term autogenous shrinkage results of concretes made with low-fineness and high-fineness cements with different level of slag replacements are presented in Figures 7.2 and 7.3, respectively. It is obvious that concrete with 25% slag exhibited rapid length change compared to other mixtures (i.e. between 7 to 28 days). As the slag content increased, the autogenous shrinkage reduced over the test span of 360 days. This could be attributed to slower hydration reactions. The low-fineness cement mixtures made with 25% slag exhibited 15-55% higher shrinkage compared to 35% and 50% slag levels of replacements for both cement high-fineness and low-fineness cements (see Figure 7.2). The high-fineness cement mixtures, on other hand, exhibited a reduction by using 35% and 50% slag ranging between 18-40% lower than the 25% slag mixture, as shown in Figure 7.3.
Figure 7.2 Autogenous shrinkage versus time for low-fineness cement made with different slag replacements

Figure 7.3 Autogenous shrinkage versus time for high-fineness cement made with different slag replacements
Figure 7.4 presents the plots of autogenous shrinkage of low-fineness and high-fineness cement mixtures with and without SRA admixture. The solid and dashed lines represent concretes made with high-fineness and low-fineness cements, respectively. The gray and black colours represent SRA or non-SRA mixtures, respectively. The shrinkage plots for mixtures containing 2% of SRA by mass cement binder exhibited substantial reduction through the entire test period (7-360 days). The high-fineness cement mixtures with or without SRA exhibited higher autogenous shrinkage compared to those with low-fineness cement. The 7 to 28 day shrinkage strains were lower for mixtures made with SRA; about 68-72% lower compared to non-SRA mixtures. At later ages (i.e. beyond 28 days till 360 days), the SRA mixtures had lower length change than non-SRA mixtures by 26-28%. Figure 7.5 presents the autogenous shrinkage of low-alkali cement (B-II-25S) and high-alkali cement (C-II-25S) concretes. It was obvious that the low-alkali cement shows reduced shrinkage over the test period. The reduction was about 23% at 7 to 28 days. After 28 days, the difference between the high-alkali cement and low-alkali cement length change was about 18%.
Figure 7.4 Autogenous shrinkage versus time for high-fineness cement and low-fineness made with/without SRA admixture

Figure 7.5 Autogenous shrinkage versus time for concretes made with high-alkali cement and low-alkali cement
7.3.2 Drying shrinkage

Figures 7.6 and 7.7 show the plots of linear drying shrinkage versus time over 360 days. The most shrinkage was observed for 25% GGBFS mixes, A-II-25S (Figure 7.4) and C-II-25S (Figure 7.5). This could be attributed to low-slag replacements in conjunction with the high portland cement content (69% of total cement binder) and high silica fume content (6%). As the slag replacements increased to 35% and 50%, the lowest drying shrinkages were observed. The low-fineness cement made with 35% and 50% slag replacements, at ages 7-28 days, exhibited reductions ranging from 31-52% compared to 25% slag level of cement replacement as shown in Figure 7.6. After 28 days, however, the range of reduction was 20-35%. For mixtures made with high-fineness cement, the 35% and 50% slag concretes show a significant reduction in drying shrinkage ranging between 39-57% at ages 7-28 days and for ages beyond 28 days, the reduction ranged from 12-33%.

![Figure 7.6 Drying shrinkage versus time for low-fineness cement made with different slag replacements](image-url)
Figure 7.7 Drying shrinkage versus time for high-fineness cement made with different slag replacements.

Figure 7.8 presents the drying shrinkage curves of low-fineness and high-fineness cement concretes with and without SRA admixture. The solid and dashed lines represent concretes made with high-fineness and low-fineness, respectively. The gray and black colors represent SRA or non-SRA mixtures, respectively. The drying shrinkage plots for SRA mixtures showed a reduction from 7-360 days. Shrinkage with high-fineness cement was higher in mixtures with or without SRA compared to low-fineness cement. The 7-28 days reduction in drying shrinkage of SRA mixtures was about 62% compared to non-SRA mixtures. At ages beyond 28 days, the SRA mixtures had lower drying shrinkage than non-SRA mixtures by 36-42%. Figure 7.9 presents the autogenous shrinkage of the low-alkali cement (B-II-25S) and high-alkali cement (C-II-25S) mixtures. It was clear the low-alkali cement shows a reduction over the entire test span. The reduction was about 17% from 7 to 28 days. At
ages beyond 28 days, the average reduction between the low-alkali cement and high-alkali cement was about 21%.

Figure 7.8 Drying shrinkage versus time for high-fineness cement and low-fineness made with/without SRA admixture
The free autogenous and drying shrinkage were effectively reduced for mixtures using the different mitigation methods (35% or 50% slag replacements, presence of low-fineness cements, using low-alkali cement, or presence of SRA). The test results shrinkage agree with previous findings of binary mixtures/or ternary mixtures using SRA admixture (Liu, 2013; D'Ambrosia, 2011). However, contradictory results were reported by others on the effect of slag replacements on drying shrinkage. Some researchers mentioned increasing the slag replacements will increase the drying shrinkage, others suggested the opposite. These contradictory results might be attributed to different chemical and physical properties of the slags used by the different researchers.
7.4 Discussion

An evaluation of length change at 28 days and 360 days is presented. Several standards specify 28 days as a benchmark for evaluating drying shrinkage. The discussion includes the effect of cement fineness, slag replacement, presence of SRA admixture, and alkali content of cement. Total shrinkage is provided in Section 7.4.4 to quantify and for discussion the total shrinkage (autogenous plus drying shrinkage).

7.4.1 Effect of fineness of silica fume blended cement and GGBFS replacements

The cement fineness plays a significant role on later age length change (i.e. beyond 7 days of wet curing). The impact of blended silica fume cement on the autogenous and drying shrinkage is presented and discussed. The values of the autogenous shrinkage testing at 28 and 360 days are presented in Figures 7.10 and 7.11, respectively. Each bar in the figures represents the average of three prisms and minimum and maximum values are also shown. Concrete mixes made with low-fineness cement showed consistent reduction in autogenous shrinkage for all slag replacements (see Figure 7.10). The high-fineness concretes also showed a reduction in autogenous shrinkage at all slag replacements, as shown in Figure 7.11. Increasing the slag contents from 25% through 35% and 50% decreased the autogenous shrinkage regardless of the cement fineness.
Figure 7.10 Autogenous shrinkage at 28 days for low-fineness and high-fineness cements made different slag replacements (25%, 35%, and 50%)

Figure 7.11 Autogenous shrinkage at 360 days for low-fineness and high-fineness cements made different slag replacements (25%, 35%, and 50%)
The drying shrinkage results at 28 and 360 days are presented in Figures 7.12 and 7.13, respectively. At 28 days, concrete mixtures made with low-fineness cement exhibited a reduction in drying shrinkage ranging 14-24% for all slag replacements compared to the companion mixes with high-fineness cement (see Figure 7.12). At 360 days, low-fineness showed a reduction in drying shrinkage ranging between 10-19% (see Figure 7.13). As the slag level of replacements from 25% through 35% and 50%, the drying shrinkage decreased.

Figure 7.12 Drying shrinkage at 28 days for low-fineness and high-fineness cements made with different slag replacements (25%, 35%, 50%)
Figure 7.13 Drying shrinkage at 360 days for low-fineness and high-fineness cements made with different slag replacements (25%, 35%, and 50%)

As was reported in Chapter 4, reduced thermal strain and hydration temperature that increasing slag replacements could potentially reduce the hydration rate and thermal strain and therefore the development of early- and later-age shrinkage. The high-fineness blended cement also plays a significant role in increasing the early and later-age shrinkage compare to low-fineness blended cements due to increase in surface tension caused by a drop in external and internal relative humidity due to higher rate of hydration. The current findings on ternary cement binders made with high-fineness and low-fineness cements shows that high-cement fineness mixes shrink more than low-fineness cements. This aligns with previous conclusions by Bentz (2001) on the impact of using fine and coarse portland cements.
7.4.2 Effect of SRA admixture

Figures 7.14 to 7.17 present the effect of the SRA admixture on different concrete mixtures at 28 and 360 days. The autogenous shrinkage of concretes made with high-fineness and low-fineness cements are presented in Figure 7.14 and 7.15, respectively. A substantial reduction in autogenous shrinkage was observed due to the use of SRA admixture for both cements (see Figures 7.14 and 7.15). The reduction autogenous shrinkage ranged between 28-68% for concretes made with SRA admixture (i.e. low-fineness mix A-II-25S (SRA) compared to companion concrete made without an addition of SRA admixture (i.e. low-fineness mix A-II-25S). Further, the reduction in drying shrinkage, ranged from 42 to 62%, as shown in Figure 7.16. The high-fineness concretes made with SRA admixture (C-II-25S (SRA)) exhibited a reduction ranging from 28-72% in autogenous shrinkage compared to non-SRA mix (C-II-25S); however, a range of 36-61% was observed for C-II-25S (SRA) mix compared to C-II-25S mix in drying shrinkage values.
Figure 7.14 Autogenous shrinkage at 28 and 360 days for low-fineness made with/without SRA admixture

Figure 7.15 Autogenous shrinkage at 28 and 360 days for high-fineness made with/without SRA admixture
Figure 7.16 Drying shrinkage at 28 and 360 days for low-fineness made with/without SRA admixture.

![Graph showing drying shrinkage comparison](image)

Figure 7.17 Drying shrinkage at 28 and 360 days for high-fineness made with/without SRA admixture.

7.4.3 Effect of alkali content in blended silica fume cements

Autogenous and drying shrinkage values are presented in Figures 7.18 and 7.19, respectively. The low-alkali cement B exhibited a reduction in autogenous shrinkage ranging between 18-24% compared to a companion mix with high-alkali cement C, as shown in Figure 7.18. The drying shrinkage, exhibited a reduction between 17-22% (see Figure 7.19).
Figure 7.18 Autogenous shrinkage at 28 and 360 days for concretes made with high-alkali cement C and low-alkali cement B

Figure 7.19 Drying shrinkage at 28 and 360 days for concretes made with high-alkali cement C and low-alkali cement B
7.4.4 Total shrinkage

Several researchers (Aitcin, 1998; Sakata & Shimomura, 2004; Tazawa & Miyazawa, 1995) claim that autogenous shrinkage in normal-strength concrete (i.e. w/c >0.4) is minor and therefore it could be considered as negligible. However, for high-strength concrete/high performance concrete (i.e. low w/c <0.4, with higher cement binder content), autogenous shrinkage may not be ignored. The ACI 116R defines drying shrinkage as ‘‘shrinkage resulting from the loss of moisture.’’ The actual shrinkage strain of a specimen under drying conditions without being subjected to applied load and temperature change includes both the autogenous and drying shrinkage. Sakata & Shimomura, (2004) mentioned that JSCE (Japan Concrete for Civil Engineers) Specification 2002 calculates the total shrinkage of high strength concrete (i.e. higher binder and lower water-cement ratio less than 0.4) using Equation 7.1.

\[ \varepsilon_{cs} = \varepsilon_{as} + \varepsilon_{ds} \]  

Eq. 7.1

Where, \( \varepsilon_{cs} \) = shrinkage strain of concrete; \( \varepsilon_{as} \) = autogenous shrinkage strain; \( \varepsilon_{ds} \) = drying shrinkage strain

Figure 7.20 presents the summation of autogenous and drying shrinkage values for different mixtures using the JSCE equation to quantify the total shrinkage strain of concrete. The mixtures contained cements with high-fineness (reference), low-fineness, and low-alkali and made with 25% slag, the 35% and 50% slag, as well as SRA. It was obvious that concrete made with SRA admixtures and 50% slag shows around 55-60% reduction in total shrinkage compared to the other mixtures.
From Figure 7.20, can be observe that all the mixtures show reductions in total shrinkage to well below the 400 microstrain (i.e. the limit of free shrinkage of MTO and CSA A23.2 standards). It should be mentioned that the CSA A23.1 shrinkage limit for low-shrinkage concrete considers only the drying shrinkage without taking into account the autogenous shrinkage.

7.5 Conclusions

The effect of blended silica fume cement fineness on free autogenous and drying shrinkage was evaluated in this chapter. Mixtures included concretes containing different replacement levels of slag cement (25, 35, and 50%) cast with high-alumina slag, or presence of shrinkage reducing admixture.

The key findings are as follows:
1. Decreasing the fineness of blended silica fume cement decreases free shrinkage (i.e. autogenous and drying).

2. As the partial replacement of blended silica fume cements with slag increases, the free shrinkage decreases compared to mixtures containing 25% slag replacement. Between 7 and 28 days, significant reduction in free shrinkage were observed for 50% slag replacement of cement compared to 25% and 35%.

3. Mixtures containing SRA admixture exhibited substantial reduction of free shrinkage than the companion non-SRA mixtures, shrinkage was 61-72% less at 28 days and 26-42% less at 360 days compared to the non-SRA mixtures.

4. Mixtures containing 50% replacement of cement with slag cement exhibited average reductions in shrinkage, on, of 58% at 28 days and 35% at 360 days compared to the companion concrete made with high-fineness or low-fineness cement 25% slag.

5. The free shrinkage results obtained for low-alkali cement mixture (B-II-25S) was lower compared to counterpart high-alkali cement mixture (C-II-25S) in the range from 16 to 22% over the test span.
Chapter 8. Analysis of test results and discussion

This chapter presents the discussion of the test results which covered measurement of thermal strain and restraint shrinkage. The discussion is on the analysis and evaluation of the interrelations between temperature peak and thermal strain as well as the restraint strain.

8.1 Peak temperature and thermal strain

The peak temperature versus ultimate thermal (i.e. at end of test period 7 days) of semi-adiabatic concrete specimens (150 x 300 mm cylinders) is shown in Figure 8.1. The data plotted in Figure 8.1 were from all mixtures regardless of the chemical composition or physical properties of the cementitious binder. It was evident that as peak temperatures increased, the ultimate strains at 7 days (the end of the test period) were increased. The relationship was very strong between both the peak temperature and ultimate strain.

Figure 8.1 Relationship between peak temperature and ultimate strain of semi-adiabatic specimens
Total hydration temperature (i.e. generated due to heat of hydration) is mainly dependent on the chemical composition of the ternary cementitious binder. Figure 8.2 presents the temperature increase, $\Delta T$ ($T_{\text{peak}} - T_{\text{initial concrete}}$) versus thermal strain. It is evident that higher datum temperature causes higher ultimate strain.

![Graph showing relationship between datum temperature and ultimate thermal strain of semi-adiabatic specimens]

Figure 8.2 Relationship between datum temperature and ultimate thermal strain of semi-adiabatic specimens

### 8.2 Effect of hydration-temperature on cracking potential

At early-age, cracking potential is influenced by hydration temperature, autogenous shrinkage, rate of stress development, and tensile creep as well as mechanical properties (i.e. strength, elastic modulus) (Shen, Jiang, Jiao, Shen, & Jiang, 2017; Bentur, Igarashi, & Kovler, 2001). The high strength concrete (i.e. containing binders with high portland cement content) exhibits higher early-age elastic modulus of elasticity compared to normal concrete. This is explained by the fact that the compressive
strength and elastic modulus at 1 to 2 days is roughly more than 80% of its 28 day values (Mihashi & Leite, 2004). Figure 8.3 presents the rate of tensile strain that caused cracking versus slag replacements. The rate of tensile strain to cracking is the ultimate strain at age of cracking divided by the age of cracking. The rate of tensile strain development to time of cracking increases as the datum temperature increased. It is clear that the rate of tensile strain was reduced as GGBFS increased, as was previously shown in Figure 4.23.

Figure 8.3 The rate of tensile strain to cracking versus datum temperature of semi-adiabatic specimens

The increase in temperature change resulted in earlier age of cracking. This indicated an acceleration of cracking as result of the thermo-activation of the early-age hydration reactions; with higher temperature, the self-desiccation, the autogenous shrinkage, and Young’s modulus develop more quickly, inducing an increase in the rate of internal stress development (Shen et al., 2017).
8.3 Multiple linear regression analysis and modeling

A multiple regression model for the analysis of thermal strain and temperature data from semi-adiabatic tests (Chapter 4) is described in which the dependent variables (ultimate thermal strain and temperature) can be predicted from independent variables (slag replacements (S); alumina content in slag ($S_{\text{ALUMINA}}$); cement fineness (CF)). The test results from nine mixtures were used in the analysis (average of two specimens per mixture).

8.3.1 Prediction of ultimate thermal strain

The model contains four variables to evaluate the most influential factors on thermal strain, and also for estimation of the ultimate strain (autogenous plus thermal strain). The analysis was conducted at a confidence level of 95%. Table 8.1 presents the beta coefficients of the independent variables. The beta coefficients of independent variables were classified from high to low as follows: $S_{\text{ALUMINA}} > S > CF$. The highest beta coefficient indicates the pronounced independent variable. It was obvious that alumina content in slag was the most influential variable on thermal strain. Equation 8.1 presents the multi-linear regression model used to estimate the ultimate thermal strain at end of 7 days.

Table 8.1 Beta coefficients of ultimate thermal strain

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<td>Intercept</td>
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<tr>
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<tr>
<td>Alumina content in slag</td>
<td>X Variable 2</td>
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<tr>
<td>Cement fineness</td>
<td>X Variable 3</td>
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Ultimate thermal strain = 0.43 x CF – 6.22 x S + 17.22 x S_{ALUMINA} + 76.7 \quad \text{Equation 8.1}

where:

CF = cement fineness m\(^2\)/kg; S = % slag replacement level in total cement binder; S_{alumina} = % alumina content in slag;

Figure 8.4 presents a comparison between measured thermal strain and predicted thermal strain of semi-adiabatic specimens. It is clear that there is a very good relationship. The best fit regression value (R\(^2\)) was 0.82.

![Graph](image)

**Figure 8.4 Predicted versus measured ultimate thermal strain of semi-adiabatic specimens**

The beta coefficient of the multi-regression analysis (on 18 semi-adiabatic test specimens) shows that alumina content of slag is the most pronounced factor in the model (low-alumina slag reduced strain).
This indicates that the chemical composition of slag cement is an important factor on ultimate combined thermal plus autogenous strain. Slag replacement level came second (higher slag replacement reduced strain), followed by the blended cement fineness (lower cement fineness reduced strain).

8.3.2 Prediction of peak temperature

The model contains four variables to evaluate the most influential factors on peak temperature variables, and also to estimate the peak temperature. The analysis was conducted at a confidence level of 95%. Table 8.2 presents the beta coefficients of the independent variables. The beta coefficients of independent variables were classified from high to low as follows: \( CF > S > S_{ALUMINA} \). The highest beta coefficient indicates the pronounced independent variable. It was obvious that cement fineness was the most influential variable on peak temperature. Equation 8.2 presents the multi-linear regression model used to estimate the peak temperature of semi-adiabatic specimens.

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<tr>
<td>Cement fineness</td>
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</table>

Peak temperature = 2.33 \( \times \) CF – 0.356 \( \times \) S + 0.0279 \( \times \) \( S_{ALUMINA} \) +30.23

Equation 8.2

where:
CF = cement fineness m$^2$/kg; $S =$ % slag replacement level in total cement binder; $S_{alumina} =$ % alumina content in slag;

Figure 8.5 presents a comparison between measured peak temperature and predicted peak temperature of semi-adiabatic specimens. It is clear that there is a very good relationship. The best fit regression value ($R^2$) was 0.94.

Figure 8.5 Predicted versus observed peak temperatures of semi-adiabatic specimens
Chapter 9. Conclusions and Recommendations

HPC concrete provides high early-age mechanical properties (high in compressive and splitting strength as well as modulus of elasticity) and lower rates of fluid transport (lower chloride ingress and rate of fluid penetration). However, the higher risk of early-age cracking is counterproductive to durability and is mainly attributed to the high cement binder contents typically used in HPC. The main objective of this study was to examine methods for reducing the early age volume change of HPC concrete in bridge decks, particularly in concrete containing GGBFS. MTO limits GGBFS replacements levels of 25%, due to concern with de-icer salt scaling, inadvertently impact on long term durability by increasing the risk of early-age cracking.

The key findings are based on this research are summarized as follows:

9.1 Test set-ups and instrumentation

Chapter 4 presents a simple method for evaluating the thermal strain of concrete cylinder under semi-adiabatic conditions using embedded strain gauge. This test matrix used two different blended cement fineness with/without similar chemical compositions and the effect of GGBFS replacement levels on the hydration temperature and thermal deformation up to age of 168 hours. The semi-adiabatic test method was an efficient way to evaluate the early-age thermal strain development and hydration temperature of 150x300mm cylinders. Embedded strain gauges have a very good relationship with strain measured using external LVDTs (i.e. $R^2 = 0.99$). It was evident that embedded strain gauges can efficiently be used for volume change measurements. Chapter 5 presents the test results of a modified design for measuring autogenous shrinkage. Modifications to increase circulation tubes (i.e. from 3 to 5 per copper wall) and increasing the insulation from 25mm to 50mm were efficient in maintaining
a constant temperature of the concrete sample (i.e. to eliminate thermal dilation). The error range of two duplicate samples was less than 3% which indicates good reproducibility of the testing protocol.

9.2 Volume change

The effect of chemical and physical properties of the cement binder significantly influences the volume change behaviour of HPC mixtures. The physical effect of GUb8SF cement (i.e. high fineness vs low fineness) influences the thermal strain. The high fineness increases thermal strain due to the acceleration in the rate of heat of hydration. The chemical composition of GGBFS (i.e. high-alumina vs low-alumina) influences both thermal strain and temperature peaks.

The potential for volume change is not equal for all cement types and compositions. In decreasing order of impact on volume change, and increasing order of the critical temperatures required for volume change to occur, blended silica fume cements may be categorized as follows with respect to their risk of cracking:

High-fineness cement > Low-fineness cement

Increasing the levels of slag above 25%, as a partial replacement for blended silica fume cement resulted reduced volume change when cured at high hydration temperature, increased the long-term durability, typically slowed the rate of volume change, and delayed the cracking potential. However, the effectiveness of slag in controlling thermal strain was greatest with slag with higher alumina content, \( \text{Al}_2\text{O}_3 \).

High-Alumina GGBFS II > Low-Alumina GGBFS I
9.3 Early-age cracking

Increasing the level of GGBFS contents up 50% reduced the risk of cracking potential compared to 25% and 35% mixtures. A strong relationship between the cracking potential and datum temperature (i.e. difference between the initial concrete temperature and the peak temperature) in semi-adiabatic tests indicates the dependency of thermal deformation on cracking potential.

9.4 Durability

Use of the high-fineness cement exhibits superior performance in terms of mechanical, transport, salt scaling properties compare to low-fineness cement at different slag replacement levels (25%, 35%, 50%). The high-fineness cement increases the compressive strength and splitting strength at early-ages. However, all the mixtures regardless the differences in the chemical compositions or the physical properties of blended cement shows a very low RCPT (i.e. below 1000 coloumbs). The high cement fineness cement exhibited a lower RCPT and higher resistivity compare to the lower fineness cement.

9.5 Statistical analysis and modeling

The multi-variable regression analysis and modeling is conducted to predict the ultimate thermal strain and peak temperature based on basic parameters such as cement fineness, slag replacements level, and alumina content of the slag. A very good correlation was found between the predicted and measured ultimate strain of concrete in semi-adiabatic conditions.
9.6 Recommendations

9.6.1 For MTO

1. Increasing the GGBFS replacement allowed to 35% is recommended as it is effective in reducing both thermal and autogenous strains (thus reducing the risk of transverse cracking) while still providing adequate salt scaling resistance.

2. In the case where the GGBFS replacement is increased above 35%, it is recommended to use high-fineness blended cement and it is preferable for these concrete mixtures to be cast in summer to ensure development of sufficient maturity prior to winter (i.e. when concrete starts to be exposed to freezing and thawing cycles). Therefore, it is possible to increase the MTO GGBFS replacement limits to 50% for HPC, when most bridge deck construction occurs in summer (i.e. ambient temperature range 20-35 °C), perhaps up to September 1.

3. Control of maximum delivery concrete temperature to 15 °C is an effective in reducing the thermal and autogenous strains.

9.6.2 For future study

1. The effect of blended GUb8SF cement fineness (i.e., cement A (580 m²/kg) compared to cement C (787 m²/kg)) on the volume change, mechanical, and transport properties of HPC concrete was significant at ages up to 7 days. It is recommended to investigate a wider range of cement fineness, which may be available in the Canadian market.

2. Based on the results of this thesis, for HPC mixtures containing low-fineness GUb8SF cement, limiting the GGBFS replacement levels to a maximum of 35% was critical to limit the damage caused by de-icer scaling damage.
3. While the GGBFS available in Ontario has low alumina and composition is consistent, on a global scale, a wider range of slag composition can influence volume change. Therefore, it is suggested to assess GGBFS performance with a higher range of alumina contents, different magnesium contents, as well as fineness effects on the reactivity and stress development, study of the reaction products formed based on the slag composition, and microstructural modeling of slag hydration and stress development.
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Appendix A

Experimental average values for mechanical properties

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### Experimental values for transport properties

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