Effect of Pre-wetted Perlite and Fibers on Autogenous and Plastic Shrinkage of High Strength Concrete

by

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Abstract

This thesis deals with the mitigation of plastic shrinkage in high strength concrete through internal curing and fiber reinforcement. In addition, the effects of mixture proportioning, internal curing and fiber reinforcement on elimination of autogenous shrinkage in mortar were studied. Internal curing can prevent cement paste from selfdesiccation by providing small water reservoirs within the mortar or concrete. Fiber reinforcement can potentially increase the early age tensile strength.

Plastic shrinkage cracks were observed only on the control concrete specimens. That is, the addition of fibers and partial replacement of fines with pre-wetted perlite both provided viable mechanisms to mitigate or reduce plastic shrinkage in high strength concrete. The fibers had a negligible effect on compressive strength. However, there was a significant decrease in compressive strength for mixes containing 25% perlite by volume of fine aggregate. Based on these results, 0.15% fibers by volume appears to be more useful than 25% perlite by volume for mitigating plastic shrinkage in high strength concrete.

The mixes without pre-wetted perlite underwent autogenous shrinkage; while the internally cured mixes underwent some swelling. That is, partial replacement of fines with pre-wetted perlite provided viable mechanism to completely eliminate autogenous shrinkage in high strength mortar. The mortar mix with fly ash exhibited the least amount of autogenous shrinkage. Therefore, in cases where addition of pre-wetted perlite is not desirable due to the loss in mechanical properties, sacrifices, fly ash can be considered as a good replacement for internal curing agent.

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Abbreviations

- ACI: American Concrete Institute
- AD: Air-Dried
- HPC: High Performance Concrete
- HSC: High Strength Concrete
- IC: Internal Curing
- IC Agent: Internal Curing Agent
- ITZ: Interfacial Transition Zone
- LWA: Light Weight Aggregate
- NDT: Non- Destructive Test
- NSC: Normal Strength Concrete
- NWA: Normal Weight Aggregate
- NWC: Normal Weight Concrete
- RH: Relative Humidity
- SAP: Super Absorbent Polymers
- SCM: Supplementary Cementitious Materials
- SSD: Saturated Surface Dry
- w/c: Water- Cement Ratio
- w/cm: Water- Cementitious Ratio
- w/b : water-Binder Ratio

Chemical Abbreviations

- Al₂O₃ : Aluminum oxide
- C₃A: Tricalcium aluminate
- C₄AF:Tetracalcium Aluminoferrite
- C₂S:Dicalcium silicate
- C₃S:Tricalcium silicate
- CaO : Lime (Calcium Oxide)
- Fe₂O₃: Iron oxide
- SiO₂: Silica (Silicon Oxide)

ASTM Standards

| ASTM C 143 | Standard Test Method for Slump of Hydraulic-Cement Concrete |
|-------------|---|
| ASTM 192 | Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory |
| ASTM C 231 | Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method |
| ASTM C 305 | Mechanical Mixing of Hydraulic Cement Pastes and Mortars of Plastic Consistency |
| ASTM C 403 | Standard Test Method for Time of Setting of Concrete Mixtures by Penetration Resistance |
| ASTM C 496 | Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens |
| ASTM C 1579 | Standard Test Method for Evaluating Plastic Shrinkage Cracking of Restrained Fiber Reinforced Concrete |
| ASTM C 1698 | Standard Test Method for Autogenous Strain of Cement Paste and Mortar |

Chapter One: Introduction

1.1 Introduction

The properties of concrete depend upon many factors: mix design, addition of chemical and mineral admixtures, and curing practices, amongst others. Improper curing practices in particular can significantly lower the strength and durability of concrete. In harsh environmental conditions, the water may evaporate too quickly for the cement to hydrate fully and for the potential strength to develop. In high strength concrete, which does not have enough water to begin with for the complete hydration of the cement, once the bleed water has evaporated from the surface, the concrete surface will undergo plastic shrinkage, while the concrete interior will exhibit self-desiccation. Therefore, early age shrinkage will start to occur. This thesis deals with the impact of different mix proportions and internal curing on early age plastic and autogenous shrinkage in high strength concrete. Early age shrinkage can cause surface cracks (due to plastic shrinkage) or internal micro-cracks (due to autogenous shrinkage). Plastic shrinkage manifests itself in concrete soon after it is placed. It can be thought of as a reduction in volume of the concrete surface due to water evaporation from the fresh, exposed surfaces. Autogenous shrinkage is a reduction in volume of concrete as a result of cement hydration after final set. Prior to final set, autogenous shrinkage and chemical shrinkage can be considered to be the same.

Surface and internal micro-cracks reduce the overall aesthetics, strength and long term durability of the concrete and should be prevented. One technique for controlling early age shrinkage of high strength concrete is internal curing, in which saturated light weight aggregates are used as an internal curing agent to provide additional water for the cement paste hydration. In order to compare the effects of internal curing and fibers in controlling plastic shrinkage, pre-wetted perlite as a saturated lightweight aggregates, and the addition of polypropylene fibers were examined. The effect of internal curing on the reduction or elimination of autogenous shrinkage was studied on mortars without and with pre-wetted perlite as a saturated lightweight aggregate.

Pre-wetted perlite provides concrete with additional internal water, since it is fully saturated before being added to the mix. It helps in mitigating both early age plastic shrinkage and autogenous shrinkage in high strength concrete. It should be noted that the conventional curing water that is used for mitigating plastic shrinkage cannot be used to combat autogenous shrinkage. This is particularly important in concrete with low water cement ratios, as surface water cannot penetrate into very dense mixes. Perlite offers an opportunity to ensure that there is an even distribution of "extra" water internally after casting. Fibers can help to mitigate plastic shrinkage by increasing the tensile strength of the concrete at early ages by bridging cracks; they therefore can prevent cracking from occurring, or at least help to reduce crack widths.

1.2 Scope of Thesis

The purpose of this study was to determine the effect that saturated light weight aggregate, in the form of pre-wetted perlite, could have on reducing or eliminating early age plastic and autogenous shrinkage. For studying early age plastic shrinkage, a number of concrete and mortar mixes were designed, containing pre-wetted perlite, polypropylene fibers, and combinations of both. Some mixes were used to compare the performance of internally cured concrete with that of fiber reinforced concrete in mitigating plastic shrinkage. Polypropylene fibers are commonly used to reduce or eliminate the effects of plastic shrinkage in concrete and mortar. In addition, various mortar mixes containing fly ash, silica fume or the two in combination as supplementary cementing materials were prepared. They were used to study the effect of internal curing in reducing or eliminating autogenous shrinkage.

Early age plastic shrinkage was studied on concrete and mortar using both ASTM C 1579 "Standard Test Method for Evaluating Plastic Shrinkage Cracking of Restrained Fiber Reinforced Concrete" and the "early age shrinkage test for mortars" (as suggested by Dr. P. McGrath). For concrete, four different mixes were prepared, and compressive strength, elastic modulus, crack width, crack area and crack reduction ratio were determined for each specimen; to study the early age plastic shrinkage of mortar, sixteen different mix designs were prepared and elastic modulus, split tensile strength, crack width and crack area were determined. ASTM C 1698 "Standard Test Method for Autogenous Strain of Cement Paste and Mortar" was followed for studying autogenous shrinkage on mortar. Eight different mortar mixes were prepared; four of these contained pre-wetted perlite as an internal curing agent. The change in weight and autogenous strain of the specimens were determined.

Chapter Two: Literature Review

2.1 High Performance and High Strength Concrete

American Concrete Institute (ACI) defines high performance concrete (HPC) as "a concrete meeting special combinations of performance and uniformity requirements that cannot always be achieved routinely using conventional constituents and normal mixing, placing and curing practice". Hence, high strength concrete (HSC) can be considered as a type of HPC with enhanced compressive strength. Based on the ACI definition, the minimum 28 day compressive strength requirement for HSC is 60 MPa.

The water-binder (w/b) ratio, volume of coarse aggregate and density of HSC are all different from those of normal strength concrete. According to ACI 211.4R¹ for w/b ratios below 0.42, complete hydration does not occur. However, complete hydration is not necessary to achieve high strength, since the un-hydrated cement particles are themselves very hard and strong and perfectly bonded to the hydrated cement paste. Using this reasoning and the fact that the compressive strength depends on both the degree of cement hydration and how close together the cement particles are in the initial cement paste, the w/b ratio for HSC is always lower than 0.42 (Mindess and Aïtcin 186). The low w/b ratio of HSC leads to low workability; superplasticizers are used to compensate for the loss of concrete workability, by de-flocculating the cement particle system. When silica fume is used in HSC mixes, further pore refinement is achieved at the microscopic level by the pozzolanic reaction between the silica fume and calcium hydroxide (CH). The reaction

¹ ACI 211.4R is the "Guide for Selecting Proportions for High-Strength Concrete with Portland Cement and Fly Ash"

between amorphous silica fume and CH produces secondary calcium silicate hydroxate (C-S-H). Since C-S-H makes up most of the cement paste and occupies more volume than the original cementing materials, the pozzolanic reaction, Equation 1, reduces the porosity in the paste, which leads to an increase in strength and durability.

$CH + S + H \rightarrow C-S-H$

Equation 1

In general, HSC has a higher cement paste content than conventional concrete. This means HSC has a higher content of cement paste which makes it vulnerable to selfdesiccation and shrinkage. Also, the dense structure of high strength concrete does not allow the surface curing water to migrate into the interior; therefore, surface curing water cannot be considered as an effective external water supply for the cement hydration process in HSC. The water required for continuation of the cement hydration therefore comes from the capillary pores. This generates a water-air interface in partially filled capillary pores, which results in surface tension caused by the menisci in the capillary pores. Capillary pressure, as shown in Figure 1, depends on the surface tension and the radius of the menisci. High capillary pressure causes the relative humidity (RH) in the cement paste to drop. According to Kovler et al (Kovler and Jensen 5-13) self-desiccation is a "phenomenon of the reduction of relative humidity on hardening cement paste due to consumption of capillary water in the progress of cement hydration". When the RH in cement paste drops below 80%, self-desiccation occurs which lowers the rate of strength gain considerably (Mindess, Young, and Darwin 617). The capillary stresses generated by self-desiccation of the cement paste cause autogenous shrinkage. This type of shrinkage is only associated with low w/b concretes (Kovler and Jensen 1-3).



Figure 1: Capillary Pressure Mechanism

2.2 Shrinkage

Shrinkage, or cement contraction, is a time-dependent inherent property of cement paste, which may have considerable physical and mechanical effects on concrete structures. Shrinkage, which is due to loss of water from the cement paste, can be divided into early age and long term contraction. Deformations that occur up to 24 hours after the time of placement are considered as early age shrinkage, while the volume changes after that are considered as long term shrinkage. Figure 2 shows schematically the types of shrinkage that can occur. Concrete codes generally only consider the long term shrinkage in concrete design calculations; they tend to ignore the early age shrinkage.

As shown in Figure 2, thermal, drying and autogenous deformations can cause early age shrinkage in a concrete element. Thermal shrinkage in a concrete element is due to the dissipation of the heat of hydration in the concrete. Drying shrinkage is due to loss of water to the concrete's surroundings. Surface free energy, capillary tension, movement of interlayer water and disjoining pressure (Hansen 323-328) are the major mechanisms causing drying shrinkage. The amount of drying shrinkage in HSC is less than in NSC, as high strength concrete has a lower w/b ratio. The shrinkage that HSC undergoes is largely autogenous shrinkage, which is caused by to loss of free water through cement hydration in low w/b ratio mixes.





The effects of shrinkage can be mitigated by mineral additives, fibers, chemical admixtures, better control of curing condition, and internal curing (Kovler and Jensen 1-3). Early-age restrained shrinkage creates internal tensile stresses in a concrete element at an early age, when the concrete has not yet developed enough tensile strength to resist the induced internal stresses. This study focuses on the mitigation of shrinkage effects in high strength concrete by using polypropylene fibers, pre-wetted perlite as an internal curing agent, or the two in combination.

High strength concrete at early ages has low tensile strength and low fracture, resistance which make it particularly vulnerable to shrinkage cracking (Mihashi and Leite 141-154). Cracking negatively impacts the structural integrity, durability and aesthetics of

a concrete structure. The tensile strength development in early age concrete can be divided into three phases: liquid, setting and hardening, Figure 3. During the liquid phase, the concrete has not yet developed any rigidity; therefore, it can withstand the applied stresses simply by adjusting its shape (Holt 446). Around the time of initial setting, layers of rigid structure start to form in the cement paste. After final set the cement paste starts to develop mechanical strength at a rapid rate which allows it to begin to withstand induced stresses. Many stress induced deformations are, therefore, related to the transition phase between the initial and final setting times. In Figure 3, initial set refers to the time at which the cement can no longer easily be handled, and final set refers to the onset of the development of mechanical strength in the cement paste (Mehta 321-326). In the following sections, the relationship between cement hydration and shrinkage is first addressed; then plastic, chemical and autogenous types of shrinkage are described.



Figure 3: Concrete Early Age Phases (Mindess, Young, and Darwin 201)

2.2.1 Relationship between Cement Hydration and Shrinkage

Plastic, chemical and autogenous shrinkage (to be discussed in the following sections) are consequences of the cement hydration reactions. When cement hydrates, the absolute volume of the hydration products is less than that of the reactant materials; this difference in the absolute volumes creates porosity in the cement paste. Le Chatelier evaluated the reduction in the absolute volume of the paste after hydration at about 8%. Also, Le Chatelier (Mindess and Aïtcin 186) showed experimentally the important role that curing plays on the behavior of cement paste. The results of his experiments showed that:

- when cement paste is cured in water, it expands
- when cement paste is cured in air, it shrinks.

In recent years, it has been shown that when cement paste is cured under water in adiabatic conditions, it swells significantly. A study by Vernet (Mindess and Aïtcin 186) explained that this significant increase in the apparent volume of the paste in quasiadiabatic condition is due to the rapid growth of crystals such as portlandite and ettringite.

When concrete is cured in water, the pores formed due to cement hydration are filled with water; hence, no menisci develop. Therefore, the cement paste does not shrink; however, when cement hydration occurs in air, menisci are formed in the partially empty pores. The formation of menisci generates tensile stress in the cement paste at early ages and causes it to undergo shrinkage.

2.2.1.1 Schematic Representation of Hydration Reaction

Jensen and Hansen (Mindess and Aïtcin 186) developed a simplified schematic representation of the cement hydration reactions (Figure 4), in which the x-axis represents the degree of cement paste hydration and the y-axis represents the relative volumes occupied by the reactants. Point "I" in Figure 4, divides the y-axis into the relative volumes of water and cement. This schematic representation assumes that cement paste does not contain entrapped air; it also does not consider the time to reach full hydration.



Figure 4: Schematic Representation of Hydration Reactions (Mindess and Aïtcin 186)

Using this schematic representation, Figure 5 shows what happens for a fully hydrated cement paste with w/c ratio of 0.42. At complete hydration,[point (0, 1)], no capillary water or unhydrated cement are left in the paste, but there are pores in the cement paste which depending on the curing conditions, could be empty or filled with water. At full hydration these pores are due to "self-desiccation"; this refers to the autogenous shrinkage which is primarily associated with HSC. Autogenous shrinkage can be prevented in HSC if the pores are filled by water to prevent the formation of menisci.

However, as discussed earlier, external (surface) curing water alone is not the appropriate method of curing HSC, as its dense structure does not allow the transfer of surface curing water into the body of the concrete; therefore, the water required to fill the empty pores must be supplied from an internal source. This can be achieved by *internal curing* of the concrete with saturated lightweight aggregates or super absorbent polymers (SAP).



Figure 5: Schematic Hydration of Cement for w/c=0.42(Mindess and Aïtcin 186)

Figure 6 shows schematically what occurs for a cement paste with w/c=0.36 which is internally cured. In this case, at complete hydration, there are no empty pores, capillary water or un-hydrated cement. Therefore, the amount of water provided to the mix was enough to fill the pores of the cement paste completely; hence, the autogenous shrinkage was completely eliminated in the cement paste. These results can be predicted by using the Power's hydration equations which are given in Equations 2 and 3, where V_c, V_u and α , respectively, are capillary volume, unhydrated cement volume and degree of hydration (Aïtcin).

$$V_c = \frac{w}{c} - 0.36 \alpha \rightarrow Capillary \ volume = 0.36 - 0.36 \times 1 = 0$$
Equation 2
$$V_u = (1 - \alpha) \times 0.32 \rightarrow unhydrated \ cement \ volume = (1 - 1) \times 0.32 = 0$$
Equation 3



Figure 6: Schematic Hydration of Cement for w/c=0.36 (Mindess and Aïtcin 186)

2.2.2 Plastic Shrinkage

After concrete placement the top surface of the concrete can undergo a volume reduction. This is usually referred to as *plastic shrinkage* because the volume change occurs when the concrete is still in the plastic phase (Mehta 3-14). Environmental conditions such as temperature, relative humidity, wind speed and their combination govern the drying rate of a concrete surface, and determine the magnitude of plastic shrinkage. Plastic shrinkage can cause cracks on the surface of the concrete. Although these cracks do not have much impact on the mechanical properties of the concrete, they can provide a pathway for aggressive fluids into the concrete, thus reducing durability and service life. The chance of plastic shrinkage occurrence is higher when the coarse aggregate volume is

reduced, as shown in Figure 7; therefore, in comparison with traditional concrete, there is more chance for high strength concrete to undergo plastic shrinkage.



Figure 7: Effect of Aggregate on Plastic Shrinkage (Mindess, Young, and Darwin 617)

Plastic shrinkage is mostly due to the evaporation of the bleed water. Bleeding is a type of segregation that occurs in wet concrete. The solid ingredients are suspended in water in a wet concrete mixture; as the solid ingredients settle some of the water rises to the surface of the concrete and some of it gets trapped under either rebar or coarse aggregates. The water that rises to the surface of the concrete is called bleeding water (Mehta 3-14). Qi (Qi 6) explained the plastic shrinkage mechanism by describing the role that bleed water and evaporation rate play during concrete's early-age phases. During the liquid phase, the rate of bleeding exceeds the evaporation rate; hence, not only does no shrinkage occur, some expansion can also be expected. During the transition phase the

evaporation rate exceeds the rate at which bleed water rises to the surface and plastic shrinkage occurs. Wittman (Wittmann 49-56) considered the capillary pressure that forms between the surface particles of the specimen during this phase as the cause of plastic shrinkage. He defined a "critical point", which is a threshold for the capillary pressure value based on the w/c ratio of the mix (Wittmann 49-56). According to Wittman (Wittmann 49-56), before the critical point the capillary pressure is high enough to draw water out of the cement paste pores; however, beyond this point the capillary pressure is not sufficient to remove water from the capillary pores, which makes the concrete susceptible to plastic shrinkage. The maximum amount of plastic shrinkage occurs just before the critical point when the water is no longer evenly distributed in the cement paste (Mindess, Young, and Darwin 617). During the transition phase, the concrete has not yet gained enough tensile strength to resist the internal stresses caused by its volume contraction; therefore, these induced stresses can cause the concrete to crack. As the concrete reaches its final set, it starts its transformation to a rigid material; when the specimen has become fully rigid, a reduced rate of plastic shrinkage is observed.

Another type of deformation that occurs during the plastic phase is plastic settlement, which is due either to a change in the depth or cross section of a concrete element, or differential settlement. Plastic settlement occurs above restraining elements, such as large coarse aggregates or rebar. The concrete above the restraining element is locally restrained, while the concrete adjacent to it has no restraint and can easily settle. This difference in settlement ability can cause cracks above the reinforcement. Plastic settlement cracks are distinguished from plastic shrinkage crack by their shape. Plastic settlement cracks usually represent the restraining element shape, while the plastic shrinkage cracks can be either random or parallel (CCAA).

2.2.3 Chemical Shrinkage

Chemical shrinkage is defined as internal shrinkage resulting from the cement hydration reactions. This volume reduction is due to the difference between the specific volumes of the cement and water and that of the cement hydration products. The volume reduction due to chemical shrinkage depends on the chemical composition of the cement and the w/c ratio, because these factors both significantly impact the cement hydration reactions (Grondin et al. 151-165).

Since autogenous shrinkage occurs simultaneously with chemical shrinkage, in order to understand the mechanism of autogenous shrinkage, a thorough understanding of the amount of chemical shrinkage, its rate of occurrence and its method of calculation are important. Holt (Holt 446) distinguished between the total amount of chemical shrinkage and the rate of chemical shrinkage. She suggested that the total amount of chemical shrinkage depends on the cement chemistry and the degree of hydration; the rate of chemical shrinkage depends on the w/c ratio and the cement fineness. Two common methods of chemical shrinkage calculations were used by Holt (Holt 446): one was based only on the molecular weights of the cementing materials and the cement hydration products; the second method considered both molecular weight and densities of the cement compounds and the hydration products. The molecular weight method is not accurate for calculating chemical shrinkage, because it is very difficult to know the exact volume of each of the cement hydration products at any point in time. The second method is more commonly used for calculating chemical shrinkage; it considers the simplified cement hydration reactions that are shown in Equations 4 to 7. In this method the chemical shrinkage of each cement compound reaction is calculated by subtracting the molecular weight of the hydration products from that of the initial reaction components, as shown in Table 1. The total chemical shrinkage of the cement is then calculated by summation of the chemical shrinkage value and multiplied by the percentage of each compound present in the cement. An example calculation which was done by Holt (Holt 446) is shown in Equation 8, using the cement chemical shrinkage values and cement compositions that are given in Table 1 and Table 2.

| $2C_3S + 11H \rightarrow C_3S_2H_8 + 3CH$ | Equation 4 |
|--|------------|
| $2C_2S + 9H \rightarrow C_3S_2H_8 + CH$ | Equation 5 |
| $C_3A + 6H \longrightarrow C_3AH_6$ | Equation 6 |
| $C_4AF + 2CH + 10 H \longrightarrow C_3AH_6 + C_3FH_6$ | Equation 7 |

| Compound | Chemical Shrinkage Value (cm ³ /g) |
|------------------|---|
| C ₃ S | 0.0532 |
| C ₂ S | 0.0400 |
| C ₃ A | 0.1113 |
| C4AF | 0.1785 |

| Compound | Mass % |
|------------------|--------|
| C ₃ S | 55 |
| C ₂ S | 20 |
| C ₃ A | 10 |
| C4AF | 8 |

Table 2: Mineral Composition of Typical Type 10 Cement

TotalCS = 0.0532 * 0.55 + 0.04 * 0.2 + 0.1113 * 0.1 + 0.1785 * 0.08 = 0.0627 (cm³) Equation 8

The amount of shrinkage or swelling caused by each of the cement compound reactions is shown in Figure 8. Apart from C_2S that undergoes swelling, the other cement compounds undergo shrinkage. The C_3A shrinkage rate is significantly higher than those for C_3S and C_4AF ; this can be related to its higher rate of reaction.



Figure 8: Effect of Cement Composition on Autogenous Shrinkage

2.2.4 Autogenous Shrinkage

Autogenous shrinkage is the macroscopic volume reduction of concrete in a sealed condition (when no moisture is transferred to or from the concrete). This type of shrinkage was defined by Jensen et al (Jensen and Hansen 1859-1865) as "the bulk deformation of a closed, isothermal, cementitious material system not subjected to external forces". Kovler et al (Kovler and Jensen 5-13) stated that it is the combination of autogenous shrinkage, swelling and thermal deformations in a sealed load-free system that causes the apparent volume change in concrete. Autogenous shrinkage is measured at a constant temperature and in a sealed condition, because the total amount of drying shrinkage at a given time is generally substantial compared to the autogenous shrinkage. Therefore, by sealing the specimen the drying shrinkage is avoided and only deformations related to the cement hydration are allowed (Jensen and Hansen 1859-1865).

Autogenous shrinkage occurs in a cement paste as a result of cement hydration. Unlike other types of shrinkage that can be controlled by appropriate curing, casting and placing procedures, autogenous shrinkage can only be controlled by concrete mixture design (Holt 446). Lura (Lura, Jensen, and van Breugel 223-232) considered changes in surface tension, capillary tension and disjoining pressure as the most likely mechanisms for autogenous shrinkage. Bouasker (Bouasker et al. 13-22)reported that early-age shrinkage that occurred in a cement paste under autogenous conditions was due to capillary depression and chemical shrinkage. He considered autogenous shrinkage as a physico-chemical phenomenon which occurred due to a compressive force generated in the empty capillary pores, which resulted in self-desiccation of the cement paste.

Early age autogenous shrinkage, depending on the time of its occurrence, is either due to chemical shrinkage or self-desiccation. According to Yodsudjai (Yodsudjai and Wang) from the time of addition of water to the cementitious mixture until initial set, chemical shrinkage is the driving force for autogenous shrinkage, and the autogenous shrinkage value is the same as the chemical shrinkage value. However after initial set, when the concrete starts to form a self-supportive skeleton, autogenous shrinkage becomes more and more due to self-desiccation and less due to chemical shrinkage. Long term autogenous shrinkage occurs in the cement paste only because of self-desiccation (Holt 446). Figure 9 summarizes the test results from the bottle chemical shrinkage and volumetric autogenous shrinkage tests on the same cement paste which were performed by Hammer (Tazawa 411). The results of these tests showed that because the capillary pressure is negligible up to initial set, autogenous and chemical shrinkage values were identical. However, after initial set autogenous shrinkage depends on self –desiccation, while the chemical shrinkage depends on the degree of hydration. The difference in the mechanisms between autogenous shrinkage and chemical shrinkage causes the difference in their shrinkage values.



Figure 9: Chemical and Autogenous Shrinkage Behavior of the Same Cement Paste (Holt 36)

Despite the relationship between autogenous and chemical shrinkage as explained earlier, there is a distinct difference between the definitions of these two types of shrinkage. Chemical shrinkage is the absolute volume reduction which includes external shrinkage and the cumulated voids, whereas autogenous shrinkage is the external (apparent) volume reduction of the concrete element. Figure 10 illustrates this concept for a cement paste in a sealed condition. Until the initial set, the chemical shrinkage and autogenous shrinkage amounts are the same; however after initial set, the autogenous shrinkage which has occurred before initial set can be distinguished from that which occurred after initial set. Also, the cement paste after initial set is no longer plastic and cannot easily deform in order to adjust for the induced internal stresses; hence, the voids generated by the cement hydration cannot escape, which results in the formation of contraction pores in the cement paste.



Figure 10: Chemical and Autogenous Shrinkage Differences

Tazawa et al (Tazawa and Miyazawa 1633-1638) studied the impact of aggregates on autogenous shrinkage. They found that the amount of autogenous shrinkage is reduced by the inclusion of coarse aggregate. This is due to the fact that the addition of coarse aggregates results in a reduction of cement paste volume in concrete, which is the portion of the concrete that undergoes shrinkage; also, coarse aggregates restrain the shrinkage induced in the cement paste.

In another study by Tazawa et al (Tazawa and Miyazawa 281-287) the impact of w/c ratio, SCM, type of cement and superplacticizer on autogenous shrinkage of cement paste was studied. The autogenous shrinkage of cement pastes with w/c ratios which varied in the range of 0.17 to 0.40 is shown in Figure 14; (the curve abbreviation names are
explained in Table 3). The test results showed that w/c ratio and autogenous shrinkage have an inverse relationship. In other words, the measured autogenous shrinkage in the cement paste with w/c=0.17 was significantly higher than that measured in the cement paste with w/c =040. In addition, comparing the two different mix designs, one with silica fume and one without, (N23, 10, 0.6 and N23, 0, 0.6), the cement paste with 10% silica fume experienced more autogenous shrinkage in comparison to the mix with no silica fume.



Table 3: Explanation of Figure 14 Curve Names

| Name | Type of cement | W/C | Silica Fume % | Superplacticizer |
|------------|----------------|------|---------------------|------------------|
| N40,0,0 | Normal | 0.40 | 0.0% | 0.0% |
| N30,0,0 | Normal | 0.30 | 0.0% | 0.0% |
| N23,0,0.6 | Normal | 0.23 | 0.0% | 0.6% |
| N23,10,06 | Normal | 0.23 | 10.0% | 0.6% |
| N17,10,2.0 | Normal | 0.17 | 10.0% | 2.0% |

Figure 11: Influence of w/c Ratio on Autogenous Shrinkage (Tazawa and Miyazawa 281-287) Alumina cement and high early strength cements which have high C₃A contents showed higher early age autogenous shrinkage compared to general use cement. Different types and dosages of superplasticizers exhibited either no or very small effects on reducing autogenous shrinkage (Tazawa and Miyazawa 281-287).

2.2.3.1 Autogenous Shrinkage Measurement Methods

The methods for measuring autogenous shrinkage have improved over time. Prior to using ASTM C1698, volume and linear measurement methods were used. These two methods are explained in a study done by Jensen et al (Jensen and Hansen 1859-1865). In the volumetric method, a balloon filled with fresh cement paste was immersed in a water tank. The change in the balloon's volume was measured by monitoring the amount of displaced water. The advantage of this method was that the change in volume could be measured immediately after placement. However, this benefit was considered secondary compared to the problems associated with the volumetric method. A major problem (Jensen and Hansen 1859-1865) is that a layer of bleed water and entrapped air formed between the cement paste and the balloon surface; therefore, the volume measurements did not represent the change in the cement paste volume alone. The measured volume change was a combination of the bleed water, entrapped air and cement paste volume change. Another possible measurement error was caused by the pressure applied by the balloon to the cement paste.



Figure 12 : Volumetric Measurement of Autogenous Shrinkage

In the linear method a rigid mould was filled with a fresh cement paste in such a way that the mould caps were in contact with the cement paste. One of the disadvantages of this method was related to the time at which meaningful length measurements could be obtained, which was after final set. The cement paste before the final set was still in the plastic phase and the length measurements were then volume based rather than linear based. The second problem was due to the rigidity of the mould used for this method; the rigid mould was considered to provide a restraint to the fresh cement paste.

Currently ASTM C 1698 "Standard Test Method for Autogenous Strain of Cement Paste and Mortar" is most commonly followed for measuring the autogenous strain of cement pastes and mortars (ASTM International). This method uses a corrugated plastic tube, as shown in Figure 13, to compensate for the disadvantages of both the volumetric and linear methods. Jensen et al (Jensen and Hansen 1859-1865) explained that the corrugated shape of the tube "transforms the volumetric deformation into a linear deformation, and after set a normal, well-defined linear deformation is measured. In this way it is possible to commence linear measurements directly after casting". Kovler et al (Kovler and Jensen 5-13) stated that a portion of the measured deformation under sealed conditions was due to thermal deformations, and that therefore autogenous deformation should not be considered as the only cause of early age deformation under sealed conditions in a cement paste. However, the small diameter of the ASTM C 1698 corrugated tube permits one to ignore the shrinkage in the cement paste or mortar due to thermal deformations.



Figure 13: Corrugated Tube used in ASTM C 1698

2.3 Internal Curing

Concrete is cured for two reasons: to allow the hydration of the cement to continue; and to prevent the concrete from undergoing drying shrinkage (Kovler and Jensen 1-3). However, the dense structure of high strength concrete makes it essentially impermeable after a few days; therefore, internal curing is considered to be a useful (if not essential) curing method for this type of concrete. The ACI initially defined internal curing as "the process by which the hydration of cement occurs because of the availability of additional internal water that is not part of the mixing water" (Bentz, Lura, and Roberts 35-40); it later changed the definition to "supplying water throughout a freshly placed cementitious mixture using reservoirs, via pre-wetted light weight aggregates, that readily release water as needed for hydration or to replace moisture lost through evaporation or selfdesiccation" (Bentz and Weiss). The two methods of internal curing as shown in Figure 14 are: internal sealing and internal water curing. Kovler et al (Kovler and Jensen 5-13) defined internal sealing as "introduction to the concrete mixture of a curing agent, which is intended to delay or prevent loss of water present in the system". The internal water curing method, which is the focus of this study, is explained in detail below.



Figure 14: Internal Curing Classification (Kovler and Jensen 5-13)

Concrete can be externally cured by means of water ponding, fogging and sprinkling, evaporation retarders, impervious paper or plastic sheets (Kosmatka, S. H., Kerkhoff, B., Panarese, W. C.,MacLeod, N.F., McGrath ,R.J. 214-218). However, internal curing differs from external curing in that it provides small reservoirs of water within more or less uniformly distributed the cement paste. In practice, to internally cure high strength concrete, some of the fine aggregate is replaced by a material that is added to the concrete or mortar mixture to serve as an internal curing (IC) agent (Kovler and Jensen 5-13). IC agents may be added to a concrete or mortar mix as dry or pre-wetted materials. If the IC agent is strong enough to withstand the mixing process, it is added as a dry material. Otherwise it is added as a pre-wetted material.

In internally cured concrete the capillary pressure first draws the water necessary for the cement hydration out from the pores of a pre-wetted IC agent, instead of from the capillary pores. The order of water available to the cement paste depends on the pore size of the IC agent; as long as the IC agent pores are larger than the pore size of cement paste, the internal curing water will be used first (Henkensiefken et al. 22-30). The characteristics of an IC agent, such as its total porosity, compatibility with the cementitious system, ability to both absorb and hold water within its structure and then easily to desorb water at high relative humidity at a desirable rate are important in determining a suitable internal curing agent. The bond that water forms with an IC agent is as important as the factors mentioned above, because it determines the thermodynamic and kinetic availability of the internal curing water. Thermodynamic availability of the internal curing water ensures that the cement paste maintains a relative humidity of close to 100%; while the kinetic availability ensures that internal curing water transfers from the IC agent pores to the capillary pores of the cement paste (Jensen, Lura, and Kovler 15-27).

For the IC water to be easily available to hydrating cement, an ideal internal curing agent must hold water in the form of capillary or physically adsorbed water. Capillary water refers to either free water in the large pores (>50 nm) or water in the smaller pore sizes in the 5-50 (nm) range. Physically adsorbed water refers to the water molecules that are attached to the solid surfaces (Mehta 321-326) Of the different methods that are shown in Figure 14 for internal water curing, super absorbent polymers (SAP) and pre-soaked lightweight aggregates (LWA) are considered ideal and are the most popular internal curing agents, due to the form in which they hold water. Water molecules form a secondary bond with the super absorbent polymer structure. In a light weight aggregate water is physically absorbed in the LWA pore structure. This means that the only force that keeps the absorbed water in the pores of a LWA is capillary force. Therefore, the water in both SAP and LWA can be assumed as bulk (capillary) water which makes it more accessible to the cement grains during the hydration process. Previous studies have shown that during the drying phase in the cement paste, capillary suction causes the water to transfer from the coarser pores to the finer pores (Bentz and Weiss), since the capillary force is directly related to the pore diameter. Hence, a minimum pore size of 100 (nm) is necessary to allow the water to flow from an internal curing agent pore to the capillary pores of a cement paste (Jensen and Lura 817-825).

Between the two "ideal" IC agents, the concrete industry currently prefers SAP over LWA. Henkensiefkena et al (Henkensiefken et al. 49-54)considered SAP a better choice because the addition of LWA in a concrete mix makes controlling the rheological properties of the concrete difficult. Also, the dense pore system of LWA causes the performance of internally cured HSC in freeze-thaw cycles to be less than satisfactory. Super absorbent polymers, however, create a more controlled microstructure and the empty pores of SAP may be considered as an effective pore system for freeze thaw cycles. Jensen et al (Jensen and Hansen 647-654) have stated that factors such as difficulty in controlling fresh concrete consistency, and reduction in strength and modulus of elasticity are a few of the reasons that the concrete industry prefers SAP over LWA. In addition, SAP can be added to a mix in dry form while LWA cannot. When dry LWA is added to a concrete mixture, there is a chance that the LWA will absorb some of the water available for the hydration of cement, which further reduces the available water the cement hydration. Also, there is a probability that some hydration products will penetrate into the empty pores of the LWA and start to fill or block them. Although this could lead to a better bond between the LWA and the cement paste, it also lengthens the water transfer path from the internal curing agent pores to the capillary pores of the cement paste, which is not desirable (Jensen and Lura 817-825).

However, despite the advantages of SAP, pre-wetted LWA is still often used as an internal curing agent. Other main concern in using pre-wetted LWA, in addition to the ones mentioned above, is that a high replacement percentage of LWA could significantly reduce the compressive strength. Kovler et al (Kovler and Cusson 71-104) chose pumice as an IC agent due to "its excellent mechanical properties and high open to closed porosity ratio". These properties of pumice allowed them to reduce the percentage of sand replacement to 6% of the weight of cement, which resulted in complete elimination of autogenous shrinkage. Therefore, using LWA with excellent mechanical properties and a high open to

closed porosity ratio is preferred, since it results in a significant reduction in the required amount of LWA and minimizes the effect on the mechanical properties of concrete. In another study, (Kovler and Cusson 71-104) pre-wetted perlite was used due to its high water absorption capacity. However, many perlite particles were broken during mixing because they were not strong enough; this resulted in high slump mixtures, segregation and bleeding. This study confirmed that "excellent mechanical properties" are one of the important characteristics for a suitable IC agent.

Bentz et al (Bentz and Weiss) studied under-and over-curing problems associated with internal curing. They suggested that the required amount of internal curing agent must be determined for each mixture design in order to avoid these problems. (This concept is explained in detail in Section 2.4.1). Under-curing is caused by either a low desorption capacity of the LWA, loss of IC water through evaporation, or non-uniform distribution of the LWA in the concrete. It results in higher tensile stresses in the cement paste because the required water for continuation of the cement hydration is obtained from the bulk water, which undermines the purpose of internal curing in high strength concrete. Over-curing can cause problem when the high strength concrete is exposed to freezing conditions. When the concrete is over-cured, more water than necessary is available for cement hydration. This means that some of the internal curing water will be left in the pores of the internal curing agents, where it can freeze, leading to a lack of freeze-thaw durability (Bentz and Weiss)Jensen et al (Jensen and Lura 817-825) used Power's model to estimate the amount of internal curing water required to avoid selfdesiccation in a high strength cement paste. By assuming a w/c ratio of 0.36 and a total

coarse aggregate volume of 50%, the amount of internal curing water needed per cubic meter of concrete was estimated at about $50(Kg/m^3)$.

Although internal curing may impact long term shrinkage, as discussed in Section 2.3, the main purpose of internal curing is to mitigate early age plastic and autogenous shrinkage in high strength concrete. Henkensiefken et al (Henkensiefken et al. 49-54) used ASTM C 1579-06 to study the influence of pre-wetted LWA on plastic shrinkage cracking in concrete specimens, in which sand was replaced by up to 18% of pre-wetted LWA. The result of this study (Figure 15) shows that the average crack width in concrete specimens was reduced as the percentage of pre-wetted LWA was increased.



Figure 15: Cumulative Probability Distribution of Crack Width in Concrete Specimens with Varying Percentages of Pre-wetted LWA (Henkensiefken et al. 49-54)

The effects of varying percentages of pre-wetted LWA on reducing plastic shrinkage, average crack width and crack reduction ratio (CRR) on concrete specimens from the same study by Henkensiefken et al (Henkensiefken et al. 22-30) are shown in Table 4. The addition of pre-wetted LWA reduced plastic shrinkage, average crack width and CRR in concrete; when 18% of the sand was replaced by pre-wetted LWA, plastic shrinkage was fully eliminated. It must be noted again that the internal curing water that is used to compensate for the water that is lost from the concrete will not be available later on to reduce autogenous and drying shrinkage.

Table 4: Average Crack Width and CRR for Different LWA Replacements (Henkensiefken et al. 49-54)

| Volume of LWA | Average Crack Width (mm) | CRR |
|---------------|--------------------------|-----|
| 0.0% | 0.49 | 0 |
| 6.0% | 0.20 | 65 |
| 10.0% | 0.05 | 89 |
| 18.0% | 0.00 | 100 |

The effect of light weight aggregate on free and restrained autogenous shrinkage was studied by Bentur et al (Bentur, Igarashi, and Kovler 1587-1591). Normal weight concrete (NWC) was compared with light weight concrete (LWC) and with a normal weight concrete in which 25% of the sand was replaced by either air-dried (AD) or saturated surface dry (SSD) light weight aggregates. The results of free autogenous shrinkage tests are presented in Figure 16. NWC expanded for the first few hours and then started to shrink. The LWC-AD and LWC-SSD mixes both showed a rapid expansion rate for the first few hours. But, while the LWC-AD after 5 days began to shrink, the rate of expansion of the LWC-SSD slowed down to an almost constant rate; this mix did not undergo autogenous shrinkage. Therefore, it can be concluded that one way of eliminating autogenous shrinkage in NWC is to replace its fine aggregate by SSD-LWA.



Figure 16: Free Autogenous Shrinkage Comparison between NWC and LWC with SSD and AD Light Weight Aggregate (Bentur, Igarashi, and Kovler 1587-1591)

The results of free autogenous shrinkage studies on NWC and concrete specimens in which 25% of the sand was replaced by AD and SSD light weight aggregate are shown in Figure 17. The NWC, LWC with 25% AD-LWA and LWC with 25% SSD-LWA specimens all showed rapid expansion for the first few hours; in the NWC and LWC with 25% AD-LWA specimens, after 24 hours, autogenous shrinkage had occurred; while in the LWC with 25% SSD-LWA specimens the rate of expansion after the first few hours continued to increase with an almost constant rate and no autogenous shrinkage was measured. Both NWC and LWC with 25% AD-LWA curves crossed the x-axis on the graph shown in Figure 17, which

indicates shrinkage. From a comparison of Figure 16 and Figure 17, the LWC-AD showed more desirable results in terms of controlling autogenous shrinkage as opposed to the normal weight concrete in which sand was partially replaced by air-dried light weight aggregates.



Figure 17: Free Autogenous Shrinkage in NWC and NWC with Partial Fine Aggregate Replacement (Bentur, Igarashi, and Kovler 1587-1591)

Figure 18 shows the induced restrained autogenous shrinkage stresses in the NWC, LWC-AD and LWC-SSD specimens. In the NWC specimens tensile stresses caused by autogenous shrinkage were measured. These induced tensile stresses caused failure in the specimens at 3 MPa. In LWC-AD and LWC-SSD specimens compressive stresses were found up to 36 hours. However, after 36 hours in the LWC-AD specimens tensile stresses appeared; in the LWC-SSD specimens no autogenous tensile stress was induced.



Figure 18: Restrained Autogenous Shrinkage in NWC, LWC-AD and LWC-SSD (Bentur, Igarashi, and Kovler 1587-1591)

The results of restrained autogenous shrinkage studies on NWC, LWC with 25% AD and 25% SSD are shown in Figure 19. LWC with 25% AD and 25% SSD refer to the LWC specimens in which 25% of the sand was replaced by AD or SSD light weight aggregates. In NWC and LWC with 25% AD specimens, significant tensile stresses were induced after a few hours and continued to increase until failure occurred. The LWC with 25% SSD specimens, unlike the LWC-SSD, showed only a low value of tensile stress after 6 days. In general it can be concluded that pre-wetted LWA is preferred over air-dried LWA because of its ability to better control autogenous shrinkage in concrete or mortar specimens.



Figure 19: Restrained Autogenous Shrinkage in NWC and NWC with Partial Fine Aggregate Replacement (Bentur, Igarashi, and Kovler 1587-1591)

2.3.1 Absorption and Desorption Rate of LWA

Internal curing is effective when the internal curing agent releases the water held in its pore system at a high relative humidity (e.g. >93%). In addition, for an internal curing agent to be efficient both water absorption and water desorption rates are important. Currently there is no specific method to determine the saturated surface dry condition of coarse or fine light weight aggregates. The two common ASTM standards that are now used by the industry to determine the absorption rate of light weight aggregates are ASTM C 127-07 and ASTM C 128-07, but neither of these standards was designed for coarse or fine light weight aggregates. ASTM C 127-07 is the standard test method for the density, relative density and absorption of coarse aggregate and ASTM C 128-07 is that for fine aggregates (ASTM International; ASTM International). However, ASTM C 1498-04a "Standard Test Method for Hygroscopic Sorption Isotherms of Building Materials" can be used to determine both absorption and desorption isotherms of any hygroscopic

construction materials (Bentz and Weiss). Absorption and desorption isotherms in ASTM C 1498-04a are defined as, respectively, a "sorption isotherm that describes the wetting process of the material from the oven-dry condition" and a "sorption isotherm that describes the drying process of a material from the state of absolute saturation with water" (ASTM International).

Bentz et al (Bentz, Lura, and Roberts 35-40) used the ASTM C 1498-04a desorption isotherm section to determine the water absorption capacity of expanded shale. The prewetted expanded shale was placed in a relative humidity chamber with a known RH and a constant temperature. ASTM C 1498 04a suggests the use of different saturated salts to obtain an equilibrium relative humidity. In these tests potassium sulfate (K₂SO₄) with RH of 97% and potassium nitrate (KNO₃) with RH of 92% were used. Although the exposure of expanded shale to cement paste is different from its exposure to air, collecting desorption rate data at a high relative humidity (97%) and a lower RH (92%) for the cement paste would be the best representation of expanded shale desorption behavior when placed in an environment with similar relative humidities. Expanded shale was found to release 95% of its withheld water at a relative humidity of 92%. (Another method for determining the desorption rate of expanded shale is by determining its absorption rate at specific time intervals and then assuming desorption and absorption rates are the same).

Bentz (Bentz and Weiss) developed an equation to calculate the required mass of light weight aggregate as an internal curing agent in order for the cement to be completely hydrated. Equation 9 was developed based on equating the required amount of water for the complete hydration of cement to the water supplied by the light weight aggregate. By using this equation the amount of water is required to eliminate autogenous shrinkage and to avoid over-and-under curing concrete under sealed condition can be calculated. In "real life", no concrete structure can be completely sealed; therefore, autogenous and drying shrinkage occur simultaneously. Hence, in order to benefit from the internal curing intended to eliminate or control both autogenous and drying shrinkage, extra internal curing water must be added, either by increasing the mass of the calculated internal curing agent or by increasing the IC agent exposure time to water in order to increase the amount of absorbed water in its pores. The unknown parameters in Equation 9 are CS (cement chemical shrinkage) and \emptyset_{LWA} . Chemical shrinkage of cement can be calculated as explained in Section 2.2.3.

$$M_{LWA} = \frac{C_{f} * CS * \alpha_{max}}{S * \phi_{LWA}}$$
 Equation 9

Where:

- M_{LWA} =mass of dry fine LWA needed per unit volume of concrete
- C_f = Cement factor of the concrete mixture
- *CS* = Chemical shrinkage of cement
- α_{max} = maximum expected degree of hydration of cement (When α is greater than or equal to 0.36 its value is taken as 1 ; otherwise , its value is $\left(\frac{w/c}{0.36}\right)$)
- S= degree of saturation of LWA
- $Ø_{LWA}$ = Absorption of lightweight aggregate

The assumption made by Bentz et al (Bentz, Lura, and Roberts 35-40) in Equation 9 is that the degree of saturation is 100%. Consequently, the desorption rate that was assumed for the light weight aggregate was for the complete saturation (S=1) condition. Castro et al (Castro et al. 1001-1008) modified Equation 9 in order to account for both efficient and non-efficient internal curing agents and to consider the water absorption of light weight aggregate as a function of time, because in practice the saturation of light weight aggregate depends on its exposure time to water. They determined the water absorption capacity of commonly used LWA's in the U.S.A (clay, shale and slate) at specific time intervals for 48 hours. The rate of water absorption depends directly on the LWA's particle and pore size distribution, all of the tested samples showed a high water absorption capacity during the first few hours, though this rate of water absorption slowed down significantly over the period of 48 hours. Normalization of the water absorption data from the tested LWAs showed that the water absorption curves followed the behavior predicted by Equation 10, in which "S" is the relative absorption; "A" is the fitted constant and "t" represents the time. Equation 10 was developed from the data collected over 48 hours and showed the direct relationship between the amount of absorbed water and time. Therefore, based on the estimation calculated from Equation 9 for the required amount of internal curing water, the water exposure time must be adjusted accordingly. This means that when the water exposure time is less than 24 hours, the amount of LWA should be increased to compensate for the lesser amount of water absorbed by the LWA.

 $S = t^A$

Equation 10

Considering the reasoning behind Equation 10, Equation 11 (Castro et al. 1001-1008) can be considered as an improvement of Equation 9. In Equation 11, the value of "S" from Equation 10 was replaced by t^A to account for the time dependency of water absorption in LWA; the ψ factor in Equation 11 accounts for the amount of water released from the light weight aggregate at high RH.

$$M_{LWA} = \frac{C_{f} * CS * \alpha_{max}}{t^{A} * \phi_{LWA 24 h} * \psi}$$
 Equation 11

2.4 Fiber Reinforced Concrete

The concept of using fibers as reinforcement goes back to the early 1900s when asbestos fibers began to be used in concrete. Concrete tensile strength is about 10% of its compressive strength; therefore fibers can be used as a reinforcement to increase the tensile strength of concrete. Concrete which contains randomly dispersed short, discrete fibers is called fiber reinforced concrete (FRC). FRC is thus a composite material that contains two separate phases: fibers and concrete. These phases are chemically different and are separated by an interface. Fibers are not added to mortar or concrete to increase compressive strength; they are added primarily to increase the strain at failure. This means that the toughness and energy absorption capacity are increased, because the fibers can bridge the cracks that develop from loading or drying. Therefore, stress can be transferred across the cracks. This helps to create a more even distribution of induced tensile stresses in the specimen. This concept is shown in Figure 20 which compares the stress-strain curves for HSC, NSC and FRC.



Figure 20: Stress-Strain Curves for Plain Concrete and FRC

Fibers can also increase the tensile capacity of the matrix, depending on how well the fibers are bonded to the matrix. The ideal situation occurs when the fibers are not fractured but rather pulled out of the matrix slowly at a stress just below the tensile strength of the fibers (Banthia).

Polypropylene fibers are a man-made plastic material that is derived from oil. They are made through the melting of polypropylene chips within an extruder that then forces the melted plastic through spinning plates containing thousands of tiny holes. The resulting strands of molten plastic are cooled, stretched, and cut into fibers (NONWOVEN Team).

Sanjuan et al (Sanjuan and Moragues 655-660) studied the effect of polypropylene fibers on reducing plastic shrinkage in mortars. Their study showed that the most important factor in controlling plastic shrinkage in plain mortar is the sand/ cement ratio; however, in many cases this ratio cannot be changed. In those cases, the addition of polypropylene fibers was found to help reduce the plastic shrinkage.

In a study performed by Ma et al (Ma et al. 92-95) polypropylene fibers were added to mortar mixes to study the effect of different volume fractions of polypropylene fibers on reducing plastic shrinkage. The results of this study (Table 6) showed that by increasing the volume fraction of polypropylene fibers from 0.05% to 0.15%, the total weighted crack values compare to the plain mortar specimens were reduced from 44.7% to 87.7%.

"Crack widths were divided into 4 categories, each of which was assigned a weighted value (Ai) as shown in Table 2. For each category of crack, the weighted value was multiplied by the crack length in centimeters (li) to get the weighted cracking value (Wi). The sum of the weighted values of all cracks in a panel was called the "total weighted cracking value cracking value (W) "' for the panel. This value represented the cracking potential of the panel" (Ma et al. 92-95).

| Crack Width d (mm) | Weighted Value (Ai) |
|--------------------|---------------------|
| Large (d3) | 3 |
| Medium (3d2) | 2 |
| Small (2d1) | 1 |
| Hairline (1d) | 0.5 |

Table 5: Cracking Widths Weighted Values (Ma et al. 92-95)

| Fiber Volume Fraction (%) | 0.05 | 0.1 | 0.15 |
|--|-------|-------|-------|
| Percentage Reduction in Total Weighted Crack Value | 44.7% | 64.8% | 87.7% |

Table 6: Percentage Reduction in Total Weighted Crack Value (cm) (Ma et al. 92-95)

Ma et al (Ma et al. 92-95) also studied the impact of various volume fractions of polypropylene fibers on crack width. As shown in Figure 21, they divided the crack widths into three categories: 2 to 3 (mm), 1 to 2 (mm) and less than 1(mm). As shown in this figure, when 0.1% polypropylene fibers were added to the mix, all crack widths were less than 2 mm, with majority of them having a width less than 1 mm. The effect of polypropylene fibers on delaying the time of appearance of first crack is shown in Table 7.



Figure 21: Impact of Polypropylene Fiber on Crack Width (Ma et al. 92-95)

| Fiber Volume Fraction % | 0.05 | 0.1 | 0.15 |
|-------------------------------|-------|-------|-------|
| Increased Time of First Crack | 34.7% | 55.8% | 47.4% |

Table 7 : Time of First Crack (min) (Ma et al. 92-95)

Li et al (Li et al. 487-490) studied the effect of modified polypropylene fiber on the plastic shrinkage cracking and mechanical properties of concrete. The addition of 1.0% modified polypropylene fibers to plain concrete with a 28 day compressive strength of 58 MPa reduced the values of plastic shrinkage crack area, maximum crack width and average crack width by 97%, 71% and 66%, respectively. The addition of polypropylene fibers did not affect the 7 or 28 day compressive strength of the concrete but increased the splitting tensile strength and flexural strength of the concrete by 26% and 15%, respectively.

Chapter Three: Materials

The properties of the materials used for this study are described in the following sections.

3.1 Cement

General purpose cement², Type I, was used in all of the mix designs. The bulk density of the cement was $3150 \left(\frac{Kg}{m^3}\right)$. Table 8 shows the potential chemical composition of this Type I cement.

Table 8: Potential Composition of Type I Cement

| Type of Portland Cement | C ₃ S | C ₂ S | C ₃ A | C4AF | Blaine Fineness (m ² /Kg) |
|-------------------------|------------------|------------------|------------------|------|--------------------------------------|
| Type I | 54% | 18% | 10% | 8% | 369 |

3.2 Chemical Admixtures

ASTM C 125 defines a chemical admixture as "a material other than water, aggregates, hydraulic cement, and fiber reinforcement, used as an ingredient of concrete or mortar and added to the batch immediately before or during mixing". The chemical admixture used in all of the mixes was ADVA 190, which is a polycarboxylate based high range water reducer³ (Grace).

3.3 Supplementary Cementing Materials

CSA A23.5 defines a supplementary cementing material (SCM) as "a material that, when used in conjunction with Portland cement, contributes to the properties of the hardened concrete through hydraulic or pozzolanic activity or both" (Banthia). The two types of SCM

² The cement was donated to UBC by Lafarge Canada Inc.

³ADVA 190 superplacticizer was purchased from Grace Construction Products

used here were silica fume and class C fly ash⁴. Supplementary cementing materials, depending upon the percentage of lime (CaO) and silica (SiO₂) can show pozzolanic or cementitious behavior. As shown in Table 9, Class C fly ash has both pozzolanic and cementitious characteristics while silica fume only has pozzolanic characteristics.

Table 9: SCM Typical Chemical Composition (Mindess, Young, and Darwin 617)

| Type of SCM ⁵ | CaO | SiO ₂ | Al ₂ O ₃ | Fe ₂ O ₃ | Behavior in Contact with Water |
|--------------------------|-------|------------------|--------------------------------|--------------------------------|--------------------------------|
| Class C Fly Ash (%) | 20-30 | >30 | 15-25 | <10 | Pozzolanic and Cementitious |
| Silica Fume (%) | 0 | 85-98 | <2 | <10 | Highly Pozzolanic |

The zone between coarse aggregate surface and the bulk of cement paste is called interfacial transition zone (ITZ) which has a thickness of $20_40 \mu m$. The structure of ITZ is different from that of bulk cement paste; some of these differences are:

- The ITZ is porous
- It has higher CH concentration and lower C-S-H concentration compare to the rest of the cement paste
- It has higher concentration of ettringite

ITZ affects the mechanical properties of concrete, specifically the compressive strength, because the strength of concrete depends on strength of the matrix paste and pasteaggregate bond. In order to increase the strength of concrete both of these factors need to be increased. Reducing w/c ratio increases the matrix paste; however, improving the paste-

⁴Silica fume and Class C fly ash were provided to UBC by Lafarge Canada Inc.

⁵The abbreviations full name can be found in the Chemical Abbreviation Section of the report

aggregate bond is more difficult. The ITZ is considered as a weak link in the paste as most fractures usually occur 10_20 μ m in the thickness of ITZ. It is found that addition of 10_15% silica fume improves paste-aggregate bond due to silica fume particle size and pozzolanic capacity which allows it to act as filler and reduce the pores in ITZ, also transfer CH to C-S-H which improves the strength of ITZ.

Pozzolanic reactions of SCM's in cement paste use the CH in cement paste to produce C-S-H, as indicated by Equations 12 to 14. Aside from pozzolanic reactions that use porous CH in the cement paste, the addition of silica fume densifies ITZ; while fly ash works as filler.

$$C_3S + H \rightarrow C - S - H + CH$$
 Equation 12

$$C_2S + H \rightarrow C - S - H + CH$$
 Equation 13

$$Pozzolan + CH + H \rightarrow C - S - H$$
 Equation 14

The physical and chemical features of fly ash affect both the fresh and hardened properties of concrete. Fly ash particles, unlike cement grains, have a spherical shape which reduces the water demand for a concrete mix; at the same time its "ball-bearing" effect helps to improve workability of the mix while the mix is still in its plastic phase. The concrete mix therefore requires less water and yet shows better workability. The pozzolanic reaction of fly ash produces a somewhat denser concrete with a higher compressive strength. Figure 22 shows the inverse relationship between the porosity and compressive strength in concrete at later ages. Also, a reduction in porosity reduces the permeability of the concrete; this is important for increasing the durability and service life. One of the disadvantages of fly ash is low early strength and slower setting (Arnie Rosenberg). The percentage of fly ash added to the concrete and mortar here was kept constant at 7%, in order to keep the total pozzolanic content at 15%. Although the percentage of fly ash is somewhat lower than what it is used normally, it was appropriate for this content.



Figure 22: Compressive Strength vs. Porosity Relationship (Banthia)

Silica fume, due to its pozzolanic reactivity and filler effect, is used in concrete to achieve ultra-high strength. Silica fume, unlike fly ash, provides a balance between short and long term strength gain. The percentage of silica fume added to the mixes was kept constant at 8% by weight of cement. Due to its very fine particle size ($\sim 1\mu$ m), silica fume increases the specific surface area of the cementitious materials which leads to a significant workability loss in fresh concrete. This problem can be overcome by the addition of superplasticizers. Figure 23 shows the compressive strength for three mixes with three different percentages of silica fume at various w/c ratios. Although, the mix with 16% silica

fume shows the highest 28-day compressive strength, this high percentage of silica fume makes the concrete very unworkable, which is not desirable.



Figure 23: Influence of w/c Ratio and Silica Fume Percentage on Compressive Strength (Banthia)

Silica fume percentage of 8% was chosen because in addition, it provides sufficient strengthening without much adverse effect on workability, as shown in Figure 24, silica fume addition results in a drop in PH, which could cause rebar corrosion in reinforced concrete.



Figure 24: Silica Fume % and Concrete PH Relationship (Banthia)

3.4 Aggregates

The coarse and fine aggregates were purchased from a local supplier. The bulk density of both coarse and fine aggregate was 1600 (${}^{Kg}/{m^3}$). The coarse aggregate maximum particle size was 9 mm, with irregularly shaped particles. The fineness modulus of the fine aggregate was 3.0. Both coarse and fine aggregate were air dried for 24 hours prior to use.

3.5 Water

Tap water was used for all mixes, at a temperature of $\sim 20^{0}$ C.

3.6 Perlite

Perlite is an amorphous siliceous volcanic rock. As shown in Figure 25, perlite can be found in rock form, crushed, or expanded. Expanded perlite is produced by heating the crushed perlite to about 871° C. The perlite structure contains approximately 2 to 6

percent water; therefore, when heated this water forms steam and causes the perlite to "pop", which creates bubbles. This explains perlite's very light weight ("<u>www.perlite.net</u>").The expanded perlite is available in the market in coarse, medium or fine grades. The perlite used as an internal curing agent here was un-coated with a particle size of 1.5 (mm)⁶. Table 10 to Table 12 were obtained from the product sheet that was provided by the producer. Table 10 gives the sieve analysis for 1.5 mm perlite; while Table 11 and Table 12 give perlite's physical and chemical compositions.



Figure 25: Different Forms of Perlite ("<u>www.perlite.net</u>")

⁶The perlite used in all of the mixes was purchased from Imasco Minerals Inc.

| U.S. No. | Microns | Weight Retained (%) |
|----------|---------|---------------------|
| 10 | 2000 | |
| 12 | 1680 | |
| 14 | 1410 | 1.9 |
| 16 | 1190 | 28.8 |
| 18 | 1000 | 32.3 |
| 20 | 850 | 25.9 |
| 30 | 600 | 8.5 |
| 50 | 300 | |
| Pan | - | 2.5 |

Table 10: 1.5 (mm) Perlite Particle Size Distribution

Table 11: 1.5 (mm) Perlite Physical Properties

| Specific Gravity(Kg/m ³) | Loose Bulk Density (Kg/m³) | Particle Shape |
|--------------------------------------|----------------------------|-----------------|
| 490 | 270 | Round/Irregular |

Table 12: Typical Perlite Chemical Composition

| SiO2 | Al ₂ O ₃ | Fe ₂ O ₃ | Na ₂ O | K20 | Other |
|-------|--------------------------------|--------------------------------|-------------------|------|-------|
| 75.4% | 13.5% | 85.0% | 3.9% | 4.7% | 1.7% |

3.7 Water Absorption Capacity of Perlite

An important factor in using pre-wetted LWA is its water absorption capacity. Studies have shown that due to the low density of LWA, fully immersing it in water is a difficult task. Therefore, researchers such as Lura (Lura) have suggested soaking light weight aggregate in boiling or very hot water to obtain a higher degree of saturation. However, for the purpose of this study the procedure which was suggested by Castro et al (Castro et al. 1001-1008) was followed. This is an adaptation of the ASTM C 128-07 "Standard Test Method for Density, Relative Density (Specific Gravity), and Absorption of Fine Aggregate". The rates of water absorption and desorption were assumed to be the same for the perlite used here. Originally the water absorption capacity for three different perlite sizes⁷ was determined; however, due to unavailability of 1 and 2 (mm) perlites only the 1.5 mm perlite was used in this study.

One hundred grams of each perlite size was placed in the oven at a temperature of $105\pm$ 2 ^o C for 24 hours. The perlite was then air cooled for another 24 hours. A 1(L) volumetric flask was filled up to 80% of its volume with de-ionized water. Air-dried perlite was placed in a nylon stocking and then placed in the volumetric flask. Since the perlite density is much less than 1.0, if perlite will float; therefore, with the help of a smaller volumetric flask made of glass, the stocking filled with perlite was completely immersed in the de-ionized water. The 1L volumetric flask was agitated manually to get rid of the air, and then filled with de-ionized water to the calibrated level. After each reading, additional water was added to the volumetric flask to keep the water level at the calibrated level (Castro et al. 1001-1008). Table 13 shows the water absorption data for each perlite size up to 48 hours. The data show that the absorption capacity is directly proportional to the perlite particle size. The 2 mm perlite displayed the highest water absorption capacity. As shown in Table 13, the rate of water absorption during the first few hours for the three sizes of perlite was rapid; however, after about 6 hours there was no further absorption.

⁷Originally three different perlite sizes of 1, 1.5 and 2 (mm) were provided to us by Imasco Minerals Inc.

| Time (hr) | 1 mm (100.0g) | 1.5 mm (100.0g) | 2 mm (100.0g) | |
|-----------|---------------|-----------------|---------------|--|
| 0.17 | 0 | 3 | 8 | |
| 0.33 | 2.5 | 3 | 8 | |
| 0.50 | 2.5 | 3.5 | 9 | |
| 1 | 6.5 | 5.5 | 11.5 | |
| 2 | 7.5 | 10.5 | 16 | |
| 3 | 8.5 | 10.5 | 16.5 | |
| 4 | 10.5 | 16.5 | 23.5 | |
| 5 | 17.5 | 22.5 | 33.5 | |
| 6 | 20 | 22.5 | 38.5 | |
| 24 | 20 22.5 | | 38.5 | |
| 36 | 20 | 22.5 | 38.5 | |
| 48 | 20 | 22.5 | 38.5 | |

Table 13: Perlite Water Absorption Capacity

3.8 Polypropylene Fibers

Polypropylene fibers are synthetic fibers which are produced in either fibrillated or monofilament forms. Apart from their low cost, these fibers have hydrophobic characteristics, which allow them to be used in concrete without impacting the cement hydration reaction or the w/c ratio; they are also compatible with various chemical admixtures (Laning 87-90). Monofilament polypropylene fibers were used here; Table 14 gives their physical properties.

| Table 1 | 14: | Pol | vpropy | lene | Fiber | Physical | Properties |
|---------|-----|-----|--------|------|-------|----------|------------|
| | | | | | | | 1 |

| Fiber Type | Length (mm) | Bulk loose Density (Kg/m³) |
|---------------|-------------|----------------------------|
| Polypropylene | 25 | 900 |

3.9 Mix Design

This section covers the mix designs that were used in this study and the perlite replacement levels. The mix designs are categorized based on the type of shrinkage that was measured: plastic or autogenous. In order to be able to compare the results of internally cured mixes with plain mixes, the w/c ratio in for mixes was kept constant at 0.36. The reason for maintaining a constant w/c ratio was, first, to avoid any reduction in strength or stiffness that the additional water in the internal curing agent might cause, and second to create a condition in the cement paste to make it more vulnerable to autogenous shrinkage (Kovler and Cusson 71-104). In order to maintain the w/c ratio constant at 0.36 for internally cured mixes, the required amount of perlite was added to the original calculated amount of water. For example, Table 15 shows the mix design for the "Plain" mix without an internal curing agent. In order to prepare the pre-wetted perlite for this mix, first the required amount of perlite is calculated as shown in Appendix A-1, and then this amount of perlite was soaked in 5.49 Kg of water for 24 hours prior to casting.

| Sand | Cement | Water(w/c=0.36) |
|-------|--------|-----------------|
| 30.52 | 15.26 | 5.49 |

3.9.1 Perlite Replacement Level

In this study the amount of fine aggregate that was replaced by pre-wetted perlite was kept constant at 25%. The results of a study by Kovler et al (Kovler and Cusson 71-104) on the effect of the replacement level of NWA with pre-soaked LWA are shown in Figure 26. This study showed that even 25% replacement did not completely eliminate autogenous shrinkage; the higher the replacement level, the more the autogenous shrinkage was reduced. They reported that in order to reduce autogenous shrinkage significantly a percentage replacement in the range of 30% to 70% were required, which would greatly impair the mechanical properties of high strength concrete and was thus not practical.



Figure 26: Autogenous Shrinkage of Different Mixes with Various Replacement Percentage of LWA (Kovler and Cusson 71-104)

Kovler et al (Kovler and Cusson 71-104) referred to the results of a study by Geiker et al (2004) regarding the effect of replacement level on reducing autogenous shrinkage and on the 28-day compressive strength of mortar specimens. In that study, four different mortar mixes with silica fume were tested. These four mixes differed in the quantity and type of IC agent that was used. Aside from the control (reference) mix, in two of the mixes NWA was replaced by LWA with replacement levels of 8 and 20 %; in the last mix NWA was replaced by 0.04% SAP by mass of the cement. The results of this study (Geiker et al) are illustrated in Figure 27. They showed that higher percentage additions of the pre-soaked LWA exhibited better results compared to the mix containing SAP. Also, comparing the LWA08 and LWA20 results showed that when NWA is replaced by 20% of pre-soaked LWA, autogenous shrinkage was mitigated and an improvement in the 28-day compressive strength of the mortar was obtained.



Figure 27: Autogenous Shrinkage and Compressive Strength of Different Mortar Mixes with SAP and Various Replacement Levels of LWA (Kovler and Cusson 71-104)

3.9.2 Plastic Shrinkage

The tendency for plastic shrinkage to occur in both concrete and mortar mixes was tested. To distinguish between different mortar mixes, the abbreviations used are shown in Table 16. Table 17 to Table 20 show the mix designs that were used for studying early age
plastic shrinkage of mortar⁸. These tables are categorized based on the addition of prewetted perlite, polypropylene fibers or a combination of the two. The volume of fiber additions was kept constant at 0.15% by volume fraction of the mortar. For the mix designs with perlite, 25% of the sand was replaced with an equal volume of pre-wetted perlite. An example of how the amounts of polypropylene fibers and perlite were calculated is shown in Appendices A-1 and A-2.

| Mix Name | Mix Composition |
|------------|---|
| Control | No SCM |
| FA | Plain with 7% Fly Ash |
| SF | Plain with 8% Silica Fume |
| FA+SF | Plain with 7% Fly Ash +8% Silica Fume |
| +Fiber | The mix has polypropylene Fiber |
| +IC | The mix has pre-wetted perlite |
| +Fiber+ IC | The mix has both polypropylene fiber and pre-wetted perlite |

Table 16: Mortar Mix Abbreviations

Table 17: Reference Mortar Mix Design for Plastic Shrinkage

| | Control | FA | SF | FA+SF |
|-----------------------|---------|------|-------|-------|
| Sand (Kg) | 27.38 | 29 | 27.38 | 27.38 |
| Cement (Kg) | 13.69 | 14.5 | 13.69 | 13.69 |
| Fly Ash (Kg) | | 1.01 | | 0.96 |
| Silica Fume (Kg) | | | 1.1 | 1.1 |
| Superplasticizer (ml) | 43 | 49 | 47 | 50 |

⁸ "Early Age Shrinkage Test for Mortar" method is described in Section 4.4.2

| | Control+ IC | FA+IC | SF+IC | FA+SF+IC |
|-----------------------|-------------|-------|-------|----------|
| Sand (Kg) | 26.22 | 26.24 | 26.22 | 26.22 |
| Cement (Kg) | 13.69 | 14.5 | 13.69 | 13.69 |
| Fly Ash (Kg) | | 1.01 | | 0.96 |
| Silica Fume (Kg) | | | 1.1 | 1.1 |
| Perlite (Kg) | 1.16 | 1.14 | 1.16 | 1.16 |
| Superplasticizer (ml) | 43 | 49 | 47 | 50 |

Table 18: Mortar Plastic Shrinkage Mix Design with Pre-wetted Perlite

Table 19: Mortar Plastic Shrinkage Mix Design with Polypropylene Fiber

| | Control+ Fiber +IC | FA+ Fiber + IC | SF+ Fiber+ IC | FA+SF+ Fiber+ IC |
|-----------------------|--------------------|----------------|---------------|------------------|
| Sand (Kg) | 27.38 | 29 | 27.38 | 27.38 |
| Cement (Kg) | 13.69 | 14.5 | 13.69 | 13.69 |
| Fly Ash (Kg) | | 1.01 | | 0.96 |
| Silica Fume (Kg) | | | 1.1 | 1.1 |
| Perlite (Kg) | 1.16 | 1.14 | 1.16 | 1.16 |
| Polypropylene (g) | 24.17 | 22.82 | 22.82 | 22.82 |
| Superplasticizer (ml) | 43 | 49 | 47 | 50 |

| | Control+ Fiber | FA+ Fiber | SF+ Fiber | FA+SF+ Fiber |
|-----------------------|----------------|-----------|-----------|--------------|
| Sand (Kg) | 27.38 | 29 | 27.38 | 27.38 |
| Cement (Kg) | 13.69 | 14.5 | 13.69 | 13.69 |
| Fly Ash (Kg) | | 1.01 | | 0.96 |
| Silica Fume (Kg) | | | 1.1 | 1.1 |
| Polypropylene (g) | 24.17 | 22.82 | 22.82 | 22.82 |
| Superplasticizer (ml) | 43 | 49 | 47 | 50 |

Table 20: Mortar Plastic Shrinkage Mix Design with Pre-wetted Perlite & Polypropylene Fiber

The abbreviations that are shown in Table 21 were used to distinguish between the different concrete mixes shown in Table 21.

| Table 21: | Concrete | Mix A | bbrev | iations |
|-----------|----------|-------|-------|---------|
| | | | | |

| Mix Name | Mix Composition |
|----------------|---|
| Control | No Fiber or Perlite |
| Fiber | The mix has polypropylene fibers |
| Perlite | The mix has pre-wetted perlite |
| Fiber+ Perlite | The mix has both pre-wetted perlite and polypropylene fiber |

| | Control | Fiber | Perlite | Fiber+ Perlite |
|-------------------------|---------|-------|---------|----------------|
| Coarse Aggregate (Kg) | 35 | 35 | 31.44 | 31.44 |
| Sand (Kg) | 25.32 | 25.32 | 25.32 | 25.32 |
| Cement (Kg) | 15.32 | 15.32 | 15.32 | 15.32 |
| Fly Ash (Kg) | 1.07 | 1.07 | 1.07 | 1.07 |
| Polypropylene Fiber (g) | | 45.87 | | 45.87 |
| Perlite (Kg) | | | 3.56 | 3.56 |
| Superplasticizer (ml) | 52 | 52 | 52 | 52 |

Table 22: Concrete Mix Design for Plastic Shrinkage

3.9.3 Autogenous Shrinkage

The autogenous shrinkage was measured only on the mortar samples. To distinguish between the different mortar mixes, the abbreviations used are shown in Table 23. Table 24 shows the control mix designs which do not contain pre-wetted perlite. The internally cured mixes are shown in Table 25. The w/c ratio for the mortar mixes was kept constant at 0.36 and 25% of the sand in the internally cured mixes was replaced by pre-wetted perlite.

| Mix Name | Mix Composition |
|----------|---------------------------------------|
| Control | No SCM |
| FA | Plain with 7% Fly Ash |
| SF | Plain with 8% Silica Fume |
| FA+SF | Plain with 7% Fly Ash +8% Silica Fume |
| +IC | The mix has Pre-wetted perlite |

| | Control | FA | SF | FA+SF |
|-----------------------|---------|------|-------|-------|
| Sand(Kg) | 27.38 | 29 | 27.38 | 27.38 |
| Cement (Kg) | 13.69 | 14.5 | 13.69 | 13.69 |
| Fly Ash (Kg) | | 1.01 | | 0.96 |
| Silica Fume (Kg) | | | 1.1 | 1.1 |
| Superplasticizer (ml) | 43 | 49 | 47 | 50 |

Table 24: Mix Design for Autogenous Shrinkage without Perlite

Table 25: Mix Design for Autogenous Shrinkage with Pre-Wetted Perlite

| | Control+ IC | FA+IC | SF+IC | FA+SF+IC |
|-----------------------|-------------|-------|-------|----------|
| Sand (Kg) | 26.22 | 26.24 | 26.22 | 26.22 |
| Cement (Kg) | 13.69 | 14.5 | 13.69 | 13.69 |
| Fly Ash (Kg) | | 1.01 | | 0.96 |
| Silica Fume (Kg) | | | 1.1 | 1.1 |
| Perlite (Kg) | 1.16 | 1.14 | 1.16 | 1.16 |
| Superplasticizer (ml) | 43 | 49 | 47 | 50 |

Chapter Four: Experimental Procedures

In this study plastic shrinkage was measured on both concrete and mortar specimens, while autogenous shrinkage was measured only on mortar specimens. The detailed procedure for studying plastic shrinkage is described in Section 4.4; the procedure for measuring autogenous shrinkage is explained in Section 4.5. A review of the mixing and placing procedures, and the methods and procedures that were used for interpreting the data is also given in this chapter.

4.1 Mixing, Placing, Compression Tests and Time of Setting

For making concrete, a pan mixer with a capacity of 75 L was used. For the mortar mixes, depending on the casting volume, either an OMNI mixer (30L) or a HOBART mixer (1L) was used. The concrete specimens and the compressive strength cylinders were mixed in the pan mixer; while the mortar mixes were mixed in either OMNI or HOBART mixers (the HOBART mixer was used in two occasions for preparing small quantity of mortar for autogenous shrinkage test). For concrete, the ASTM C 192-7 "Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory " and for mortar the ASTM C 305-11 "Mechanical Mixing of Hydraulic Cement Pastes and Mortars of Plastic Consistency" were followed. The order in which the ingredients were added to the mixer was according to either ASTM C 192-7 or ASTM C 305-11. However, for the internally cured mixtures, the last solid material that was added was the pre-wetted perlite before the addition of water and superplasticizer, to minimize breakage of the perlite particles during mixing.

Slump and air content tests were performed on the fresh concrete according to ASTM C 143 "Standard Test Method for Slump of Hydraulic-Cement Concrete" and ASTM C 231 "Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method". For each concrete batch, thirty 100×200 mm cylinders were cast for measurement of the 3, 7 and 28 day compressive strengths. ASTM C 39 "Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens" was followed to determine the compressive strength of the concrete cylinders. A FORNEY Digital Controller III EX 600 was used for the compressive strength tests.

In order to determine the effect of the material variables (pre-wetted perlite, polypropylene fibers, fly ash and silica fume) on initial and final setting times of the mortar mixtures, ASTM C 403 "Standard Test Method for Time of Setting of Concrete Mixtures by Penetration Resistance" was followed. For each mortar batch six 75×150 mm cylinders were cast and the penetration resistance of the mortar was measured hourly until the last needle could not penetrate into the mortar. A graph of mortar penetration resistance vs. elapsed time was plotted in which 500 and 4000 psi, respectively, represent the initial and final setting times of the mortar. In addition the 1, 3 and 7 day splitting tensile strengths of the mortars was determined using ASTM C 496 "Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens". The Forney compression machine was also used for the split tensile strength tests on 100×200 mm cylinders. A schematic of the split tensile strength test is shown in Figure 28. Photographs of a split tensile test cylinder after failure are shown in Figure 29.



Figure 28: Split Tensile Strength Set up According to ASTM C 496 (ASTM International)



Figure 29: Split Tensile Test

4.2 Crack Width Measurement with Digital Microscope

The crack measurements were performed on specimens that were still in their moulds, 24 hours after placement. Although some drying takes place from the time of initial set, 24 hours is considered sufficient time for plastic shrinkage crack stabilization, because all cracked occurred prior to initial set and the crack width did not change after that. The plastic shrinkage crack widths were measured digitally with the help of a Moticam camera. The Moticam camera is a high resolution live imaging microscopy camera which is compatible with almost any microscope and its software can be installed on both PC and Macintosh computers ("Products--North America Site-Moticam 2300."). A Moticam 2300 camera was attached to an optical microscope to turn it into a digital microscope.

As shown in Figure 30, the Moticam was connected by a USB cable to the computer. The Moticam software consists of both photo capturing and image analyzing sections. The first section uses the monitor screen as the Moticam LCD; therefore, by clicking on the capture option as shown in Figure 31, a picture of the crack is taken. On a separate window which opens up after taking the picture, the crack width can be measured by choosing the linear measuring tool among various measuring tools that this software offers. As illustrated in Figure 32, using the linear measuring tool a line is drawn from one side of the crack to the other. The software gives the length of the line in mm. The crack width measurements were taken every 1 cm along the length of the crack. However, to prevent possible edge effects the cracks were measured at least 25 mm from the boundaries of the moulds.



Figure 30: The Microscope Set Up



Figure 31: Picture of the Specimen Taken with the Moticam Software



Figure 32 : Measuring Crack Width with the Moticam Software

4.3 Ultrasonic Pulse Velocity

The ultrasonic pulse velocity test (UPV) is a non-destructive test (NDT) which relates the speed of an ultrasonic pulse (v) passing through concrete to the elastic modulus of concrete. In this test, two transducers make contact with the concrete across a fixed distance. The device measures the time it takes for vibrations generated by a piezo-electric crystal in one transducer to be detected by the second transducer (Neville 844)

In this study, the elastic modulus of 1, 3 and 7 day specimens was determined using Equation 18 ($E = \rho v^2$). The cylinders were weighed to determine their densities. To determine the ultrasonic pulse velocity in the cylinders, aquasonic transmission gel was applied on the UPV transducers before placing them on the opposite ends of a cylinder. The length of the cylinder, 200 mm, was then entered into the equipment. The start button was pressed and the ultrasonic pulse velocity was recorded from the UPV device. The elastic modulus data are the average of three replicates for each specific mix design.

Contact surface smoothness and moisture content are two important factors which affect pulse velocity. Contact surface smoothness was ensured by grinding both ends of the cylinders. The use of the ultrasonic gel ensured full contact between the concrete and transducers. The moisture content was assumed to be constant between different cylinders and each specimen. Cylinders were brought out of the curing tank a few hours before testing to allow the cylinder's surfaces to dry equally in order to maintain a consistent moisture content. No special measures were required to keep other factors such as path length and temperature consistent (Mindess, Young, and Darwin 617)

4.4 Plastic Shrinkage

This section covers the procedures that were used for studying plastic shrinkage on the concrete and mortar specimens. In addition the environmental chamber setup proposed in ASTM C 1579 "Standard Test Method for Evaluating Plastic Shrinkage Cracking of Restrained Fiber Reinforced Concrete" and the chamber setup that was used for this study are explained in detail.

4.4.1 Plastic Shrinkage Environmental Chamber

The environmental chamber described here was used to study the plastic shrinkage potential on concrete and mortar specimens. Figure 33 illustrates the chamber set up proposed by ASTM C 1579 which, according to this standard, creates harsh environmental conditions as shown in Table 26. The harsh conditions proposed by ASTM C 1579 are assumed to create a water evaporation rate of 1.0 (Kg/m^2) or higher. According to CSA A23.1, for mix designs without silica fume, a water evaporation rate of 0.5 (Kg/m^2) is required; for mixes containing silica fume, a rate of 0.2 (Kg/m^2) is required. ASTM C 1579 does not provide information about the dimensions of the chamber, or the type and strength of the heater, fan and de-humidifier (Figure 33). Therefore, assumptions were made for these factors as shown in Figure 34.



ELEVATION

Figure 33: Environmental Chamber Set up Proposed by ASTM C 1579 -06

Table 26: Environmental Chamber Condition Proposed by ASTM C 1579-06

| | Values Proposed by ASTM C 1579 |
|-------------------|--------------------------------|
| Relative Humidity | 30 ±10 % |
| Temperature | 36 ±3 °C |
| Wind Velocity | 4.7 (m/s) |

The modified chamber setup had three heaters, one fan and no de-humidifier; its dimensions were $304 \times 141 \times 91$ cm. An Americ VAF-3000 industrial ventilator (Figure 40) was used as the fan in the chamber. The environmental conditions created by this set-up are given in Table 27. The wind velocity obtained with one fan was 2.7 (*m/s*) on average, which is lower than the 4.7 (*m/s*) value proposed in ASTM C 1579. However, preliminary tests confirmed that the chamber conditions satisfied the water evaporation requirements

based on CSA A23.1; also, using two ventilators in the chamber created a wind velocity of about 10 (m/s) which was too high; therefore, only one fan was used. No de-humidifier was used in the modified chamber because the combination of the low w/b mix, the temperature and the wind velocity created humidity in the range that was specified by ASTM C 1579.



Figure 34: The Modified Chamber Setup

| Table 27 : Modified Environmental Chamber Condition | on |
|---|----|
|---|----|

| | Modified Environmental Chamber |
|-------------------|--------------------------------|
| Relative Humidity | 18 % |
| Temperature | 36 ° C |
| Wind Velocity | 2.7 (m/s) |



Figure 35: Americ VAF-3000 Industrial Ventilator

5.1.1 4.4.1 Plastic Shrinkage Test, Based on ASTM C1579

ASTM C 1579 "Standard Test Method for Evaluating Plastic Shrinkage Cracking of Restrained Fiber Reinforced Concrete" was used to study the plastic shrinkage. The purpose of this test is to study the effect of evaporation rate, settlement and plastic shrinkage on fiber reinforced and plain concrete panels.

4.4.2.1 Mould

Figure 36 illustrates the mould that was used for studying plastic shrinkage on concrete panels based on ASTM C 1579-06. The mould itself was made out of durable and stiff plastic and the internal restraints and stress risers were constructed from one piece of steel.



Stress Riser Geometry

Figure 36: Side, Top View and Stress riser Geometry of ASTM C 1579 Mould (ASTM International)

4.4.2.2 Procedure

To ensure that the chamber temperature was in the range suggested by ASTM C 1579, the chamber heaters were turned on prior to concrete mixing. The moulds were filled with concrete in one layer and were vibrated for a few seconds to allow the air bubbles to escape and the concrete surface to level out. One concrete panel was placed in the chamber at a time. The fan was then turned on. It was turned off at the time of final set. The time of final set was determined for the mortar portion of the concrete mix design according to ASTM C 403. In designing the mortar mix to represent the concrete design, the w/c ratio and the concrete mix materials percentages were kept constant, except that the coarse aggregates were replaced by fine aggregates. As shown in Figure 34, the concrete panel was placed under the wind, temperature and humidity sensors which were connected to a data logger. A water pan was also placed in the chamber at the same elevation as the concrete mould to measure the rate of water evaporation in the chamber. The environmental conditions in the chamber and weight of the water pan were measured hourly up to 6 hours. Also during this time, the concrete panel was checked every hour in order to record the time of first crack. The heaters were turned off after 24 hours and, if the concrete panel was cracked, the crack widths were measured using the Moticam software as described in Section 4.2.

4.4.2 Early Age Shrinkage Test for Mortar

Dr. P. McGrath, an adjunct professor in the UBC Department of Civil Engineering, has proposed a method for studying early age plastic shrinkage in mortar. His method was used here because, unlike concrete, there is no ASTM standard for studying early age plastic shrinkage on mortar. The chamber set up and the curing conditions for this method are the same as those used for ASTM C 1579. The major difference between these two methods, (aside from the fact that one is used for concrete and the other one for mortar), is in the mould. The mould and the procedure used for this method are explained below.

4.4.3.1 Mould

Figure 37 shows the mould that was used for the "early age shrinkage test on mortar" method. The plywood mould dimensions are 270×80 ×100 mm. Plywood absorbs some water and represents the framework that is used in practice; therefore, the test results should be more applicable to "real life" cases. The mould has 7 stress risers, one of which is a blade which divides the mould in half, and 6 screws located evenly on each side of the blade. The mould thickness was chosen as 1 cm to reflect the target application thickness for specific repair applications; it also provides a large surface area to thickness ratio.



Figure 37: Top and Side View of Mortar Mould

4.4.3.2 Procedure

To ensure that the chamber temperature was in the range suggested by ASTM C 1579, the chamber heaters were turned on prior mortar mixing. Two people helped in the casting process to minimize the elapsed time between addition of water to the cementitious mixture and placement. While one person filled the plastic shrinkage moulds, the other person filled the time of set cylinders; the split tensile strength cylinders were filled afterwards. Five moulds were placed in the chamber at a time in the order shown in Figure 38. A water pan was also placed in the chamber to measure the rate of water evaporation in the chamber while the mortar specimens were curing. After placing the moulds in the chamber, the fan was turned on; it was turned off at the time of final set. The times of initial and final set were determined based on ASTM C 403. The chamber environmental conditions and the weight of the water pan were measured hourly up to final set. The heaters were turned off after 24 hours and, if the mortar specimens were cracked, the crack widths were measured using the Moticam software (Section 4.2).



Figure 38: Mould layout in the Environmental Chamber

4.5 Autogenous Shrinkage

ASTM C 1698-09 "Standard Test Method for Autogenous Strain of Cement Paste and Mortar" was followed for measuring the autogenous strain in the mortars. The dimensions of the corrugated mould and its end plugs used for this study are shown in Figure 39.



Figure 39: Outer Dimensions for Corrugated Mould and End Plug (ASTM International)

The eight mortar mix designs that were used for studying autogenous shrinkage are the same as those used for studying plastic shrinkage; therefore, to increase the accuracy of the measured data, the autogenous and plastic shrinkage specimens, the time of set cylinders and the split tensile test cylinders were all cast from the same batch.

Prior to mixing, the end plug lengths were measured and one end of the corrugated tube was sealed with a plug. During mixing, the time of water addition to the cementitious material was recorded. Due to the sensitivity of the test to the time between water addition to the cementitious material and placement, two people helped in filling the moulds.

The corrugated mould was placed in a support glass tube on a vibrating table. The mortar mixes studied here were not pourable; therefore, in order to fill the moulds the mortar was rolled into small sections and then dropped into the corrugated mould. The mould was filled up to 1.5 cm below its top end in four layers. Each layer was consolidated with a tamping rod in order to compact the mortar in the tube and allow the entrapped air to escape. They were then sealed with another end plug. The outer surface of the filled mould was washed with water to remove any excess mortar before weighing it. The moulds were then transported on a corrugated tray, shown in Figure 40, from the casting area to a measuring room with a constant temperature of $23 \pm 1^{\circ}$ C. The purpose of using a corrugated tray is to avoid any possible deformations that moving the moulds can cause. The specimens were left on the corrugated tray with a minimum distance of 5 cm between them to avoid the impact of heat of hydration from one specimen on the neighboring specimens. The change in length of the corrugated moulds was then measured at 1, 2, 3, 7 and 14 days, starting from the time of final set, using the dilatometer shown in Figure 41.

For each length measurement, the gauge was first set to zero by placing the reference bar on the dilatometer bench and setting it to zero. Then the reference bar was removed and a specimen was placed on the dilatometer bench. Three consecutive measurements were recorded for each reading from each specimen.



Figure 40: Corrugated Tray



Figure 41 : Autogenous Shrinkage Dilatometer, corrugated tube and Reference Bar

4.6 Data Interpretation

In this section the equations and calculation methods that were used for analyzing the data are explained.

4.6.1 Plastic Shrinkage Crack Analysis

According to ASTM C 1579, if the crack width average of the control sample is not at least 0.5 (mm), the plastic shrinkage test is not valid and the chamber conditions must be

changed so that this minimum crack width requirement is satisfied. The crack widths obtained for both the concrete and mortar control specimens was higher than 0.5 mm; therefore, the chamber conditions described in Section 4.4.1 were considered satisfactory.

For analyzing the plastic shrinkage cracking of concrete panels according to ASTM C 1579, the crack reduction ratio (CRR) was calculated using Equation 15. In addition, the crack width average and population standard deviation were determined for the concrete panels.

$$CRR = \left[1 - \frac{Average\ Crak\ Width\ of\ Tested\ Mixture}{AverageCrack\ Width\ of\ Control\ Mixture}\right]$$
Equation 15

For analyzing the plastic shrinkage cracking in the mortar specimens, the average crack width, crack area and the population standard deviation were determined. A trapezoidal approximation was used to determine the crack area (Figure 42). The height of the trapezoid was 1 cm because the crack widths were measured every 1 cm; b_1 and b_2 represents crack width "i-1" and "i".





Figure 42: Crack Area Calculation

4.6.2 Water Evaporation Rate

The rate of water evaporation in the environmental chamber was determined as the mass of water evaporated from the surface area of the pan per hour, see Equation 16.

$$Evaporation Rate = \frac{mass of water evaporated\left(\frac{kg}{hr}\right)}{surace area of pan(m^2)}$$
 Equation 16

The mass of evaporated water was measured every hour using a scale with a sensitivity of

0.1 g.

4.6.3 Autogenous Shrinkage Using ASTM C 1698

Autogenous shrinkage strain and the corrugated tube mass change at specific time intervals were measured following ASTM C 1698. Autogenous strain of the mortar

specimens was calculated using Equation 17. In Equation 17, $L(t_{fs})$ represents the length measurement at final set and L(t) represents the length of the mortar specimen at time (t), which is calculated from Equation 18.

$$\varepsilon_{autogenous} = \frac{L(t) - L(t_{fs})}{L(t_{fs})}$$
 Equation 17

In Equation 18, L_{ref} represents the length of the reference bar which was 424.903 (mm); R(t) represents the length that was read from the gauge in (mm) when the specimen was on the dilatometer bench. The length of the end plugs was 15 (mm).

$$L(t) = L_{ref} + R(t) - 2 * L_{plug}$$
 Equation 18

The change in mass of the corrugated tubes was calculated using Equation 19. The mass change was determined to ensure that moisture was not transferred to or from the tubes. In an ideal situation the mass change is zero. In Equation 19, ΔM represents the change in mass of the tube at specific time intervals and $M_s(t_e)$ and $M_s(t_{fs})$ represent the mass of the tube at each specific time interval and at the time of final set.

$$\Delta M = M_s(t_e) - M_s(t_{fs})$$
Equation 19

4.6.4 Split Tensile Strength Test Using ASTM C 496

Equation 20 was used to calculate the split tensile strength of 100×200 (mm) cylinders, where "P" represents the maximum load which the cylinder could withstand prior to failure in (N); "l" represents the length, "d" is the diameter of the cylinder and "T" is the split tensile strength in (MPa).

$$T = \frac{2P}{\pi l d}$$

According to ASTM C 496, the tensile strength determined from this method is greater than that from the direct tensile strength test, but is lower than the flexural strength.

4.6.5 Modulus of Elasticity Using UPV

The elastic modulus (E) of the cylinders was calculated from the pulse velocity and the density using Equation 21. This equation does not require the Poisson's ratio of the concrete for determining E (Mindess, Young, and Darwin 617).

$$E = \rho v^2$$
 Equation 21

E = Elastic Modulus

 ρ = Density

v = Pulse Velocity

Chapter Five: Results

In this study, the early age plastic and autogenous shrinkage of high strength concrete and mortar was studied. For analyzing the plastic shrinkage data on the mortar and concrete specimens respectively, "Early Age Shrinkage" and ASTM C 1579 procedures were used; for analyzing the autogenous shrinkage data, the ASTM C 1698 procedure was used. The Results Section is divided into two parts: concrete and mortar.

5.1 Concrete

Fresh properties, water evaporation rates, compressive strength and crack analysis data of the concrete panels are given in the following sections.

5.1.2 **Fre**sh Properties of concrete mixes and evaporation rates

The air content and slump values for the concrete mixes are shown in Table 28. The mix containing pre-wetted perlite was the most workable mix with the highest slump, 110 mm; the fiber mix had the lowest slump, 25mm. The results showed that the addition of pre-wetted perlite to the fiber mix ("Fiber+ IC" mix) increased the slump value by 50 mm. Although the percentage of superplasticizer was kept constant for all of the mixes, internally cured mixes were more workable. There was no significant difference in the air contents of the different concrete mixes.

| Type of Content | Air Content | Slump (mm) |
|-----------------|-------------|------------|
| Plain | 3.0% | 90 |
| Fiber | 3.2% | 25 |
| Fiber + IC | 4.5% | 75 |
| IC | 4.0% | 110 |

Table 28: Air Content and Slump of the Concrete Mixes

Table 29 shows the rate of water evaporation during the plastic shrinkage tests for concrete. The water evaporation rate for all of the concrete mixes was above $0.5 \binom{Kg}{m^2}$; hence, based on ACI 23.1, it was expected that all of the specimens would crack. However, only the control mix with no supplementary material to help the concrete in plastic shrinkage was cracked. Therefore, pre-wetted perlite, polypropylene fibers, and combinations of these materials were all able to completely eliminate plastic shrinkage cracking in the concrete.

| Type of Content | Evaporation rate |
|-----------------|-------------------------|
| Plain | 1 |
| Fiber | 0.9 |
| Fiber + IC | 1.0 |
| | |

1.1

Table 29: Evaporation Rate

5.1.3 7 and 28-Day Elastic Modulus and Compressive Strength

IC

Table 30 and Table 31 summarize the average values for compressive strength and elastic modulus for the concrete mixes at 1, 3, 7 and 28 days. It may be seen that the addition of pre-wetted perlite and polypropylene fibers reduced the compressive strength and elastic modulus in the concrete panels. The reduction in compressive strength and elastic modulus for the "Fiber + IC" and "IC" mixes was greater than those for the "Fiber" mix. That is, the addition of pre-wetted perlite reduced compressive strength and elastic

modulus more than did polypropylene fibers. A reduction in compressive strength of HSC is undesirable.

| | 1-Day | 3-Day | 7-Day | 28-Day |
|------------|-------|-------|-------|--------|
| Plain | 15.09 | 28.49 | 47.47 | 62.91 |
| Fiber | 10.17 | 28.37 | 40.16 | 58.47 |
| Fiber + IC | 10.04 | 25.61 | 29.25 | 42.06 |
| IC | 9.42 | 24.50 | 27.15 | 39.51 |

Table 30: Compressive Strength (MPa)

Table 31: Elastic Modulus (GPa)

| | 3-Day | 7-Day | 28-Day |
|------------|-------|-------|--------|
| Plain | 47.34 | 53.42 | 55.18 |
| Fiber | 47.18 | 53.38 | 54.95 |
| Fiber + IC | 44.88 | 51.60 | 52.09 |
| IC | 43.34 | 47.10 | 48.60 |

5.1.4 Crack Analysis

Among the different concrete specimens, only the control specimens were cracked. One crack was observed above the middle stress riser of the panel on the control specimens. The average crack width, total crack area and population standard deviation of crack widths are shown in Table 32.

Table 32: Average Crack Width and Standard Deviation

| | Average Crack Width (mm) | Crack Area (mm²) | Standard Deviation of Crack Width |
|-------|--------------------------|------------------|-----------------------------------|
| Plain | 0.35 | 9.35 | 0.131 |

5.2 Mortar

The two types of tests performed on the mortar specimens were plastic and autogenous shrinkage. The data obtained and their analyses are given in the following sections.

5.2.1 Plastic Shrinkage

Time of first crack, initial and final penetration time, elastic modulus, split tensile strength, crack width and crack area were determined for the mortar specimens. These data were then used to try to explain the reasons for the occurrence or non-occurrence of plastic shrinkage cracks on the mortar specimens.

These data are presented in several tables, based on the SCM that was added to the control mixture, in order to make it easier to analyze the effects of pre-wetted perlite, polypropylene fibers and the combination of the two materials on plastic shrinkage. The data presented here for the control mix and the mixes with fly ash are the average of twenty replicates; the data presented for the other mixes are the average of ten replicates.

5.2.1.1 Time of First Crack and Initial and Final Set

The time of first crack and the initial and final set times are given in Table 33 to Table 36. The following results were obtained:

• The time of first crack for all mixes was about one hour after placement, except for the "FA+ IC" and "FA+ Fiber" mixes, for which the time of first crack was about three hours after placement.

- The times of initial and final set were reduced in internally cured mixes in comparison with the reference mixes.
- "FA+ Fiber+ IC" was the only mix that did not crack.

The final set time, which represents the time that the penetration resistance of the mix is 4000 psi, was later used as " time zero" for the first autogenous shrinkage measurement.

Table 33: Time of First Crack and Initial & Final Set Times for Control Specimens

| Mixture Type | Time of First Crack (hr) | Initial Set (min) | Final Set (min) |
|------------------|--------------------------|-------------------|-----------------|
| Plain | 1 | 162 | 284 |
| Plain+ IC | 1 | 102 | 264 |
| Plain +Fiber | 1 | 156 | 329 |
| Plain+ IC+ Fiber | 1 | 133 | 296 |

Table 34: Time of First Crack and Initial & Final Set Times for Mixes with Fly Ash

| Mixture Type | Time of First Crack (hr) | Initial Set (min) | Final Set (min) |
|---------------|--------------------------|-------------------|-----------------|
| FA | 1 | 191 | 367 |
| FA +IC | 3 | 117 | 246 |
| FA +Fiber | 3 | 210 | 345 |
| FA +Fiber +IC | No Crack | 60 | 165 |

| Mixture Type | Time of First Crack (hr) | Initial Set (min) | Final Set (min) |
|---------------|--------------------------|-------------------|-----------------|
| SF | 1 | 140 | 236 |
| SF+IC | 1 | 104 | 230 |
| SF +Fiber | 1 | 112 | 216 |
| SF+ Fiber+ IC | 1 | 88 | 221 |

Table 35: Time of First Crack and Initial & Final Set Times for Mixes with Silica Fume

Table 36: Time of First Crack and Initial & Final Set Times for Mixes with Fly Ash and Silica Fume

| Mixture Type | Time of First Crack (hr) | Initial Set (min) | Final Set (min) |
|-------------------|--------------------------|-------------------|-----------------|
| FA+ SF | 1 | 132 | 264 |
| FA & SF+IC | 1 | 95 | 238 |
| FA & SF +Fiber | 1 | 112 | 229 |
| FA+ SF +Fiber +IC | 1 | 164 | 399 |

5.2.1.2 Elastic Modulus and Split Tensile Strength

Table 37 to Table 40 give the elastic moduli and split tensile strengths for the mortar specimens. The following conclusions may be made:

- Addition of pre-wetted perlite to the mortar mixes reduced the elastic modulus. However, the addition of pre-wetted perlite did not have a significant effect on the spilt tensile strength.
- Addition of polypropylene fibers did not have a significant effect on elastic modulus or split tensile strength of the mixes in comparison with the reference mix.

| | Elastic Modulus (GPa) | | | Split Tensile Strength (MPa) | | |
|------------------|-----------------------|-------|-------|------------------------------|-------|-------|
| Mixture Type | 1-Day | 3-Day | 7-Day | 1-Day | 3-Day | 7-Day |
| Plain | 31.59 | 38.31 | 40.97 | 3.32 | 4.28 | 4.63 |
| Plain+ IC | 28.75 | 33.24 | 36.71 | 3.15 | 4.09 | 4.29 |
| Plain +Fiber | 33.47 | 36.89 | 45.6 | 3.27 | 3.93 | 4.34 |
| Plain+ IC+ Fiber | 28.76 | 32.5 | 34.05 | 2.96 | 3.92 | 4.37 |

Table 37: Elastic Modulus and Split Tensile Strength for Control Mixtures

Table 38: Elastic Modulus and Split Tensile Strength for Mixtures with Fly Ash

| | Elastic Modulus (GPa) | | | Split Tensile Strength (MPa) | | |
|---------------|-----------------------|-------|-------|------------------------------|-------|-------|
| Mixture Type | 1-Day | 3-Day | 7-Day | 1-Day | 3-Day | 7-Day |
| FA | 37.18 | 41.74 | 44.95 | 3.95 | 4.17 | 5.03 |
| FA +IC | 27.74 | 33.70 | 36.82 | 3.36 | 3.97 | 4.23 |
| FA +Fiber | 31.48 | 36.71 | 40.97 | 3.38 | 4.1 | 4.57 |
| FA +Fiber +IC | 23.37 | 31.18 | 34.21 | 3.23 | 3.84 | 3.99 |

Table 39: Elastic Modulus and Split Tensile Strength for Mixtures with Silica Fume

| | Elastic Modulus (GPa) | | | Split Tensile Strength (MPa) | | |
|---------------|-----------------------|-------|-------|------------------------------|-------|-------|
| Mixture Type | 1-Day | 3-Day | 7-Day | 1-Day | 3-Day | 7-Day |
| SF | 31.21 | 35.29 | 39.42 | 3.54 | 4.23 | 4.69 |
| SF+IC | 28.70 | 33.83 | 38.15 | 3.36 | 3.97 | 4.23 |
| SF +Fiber | 31.21 | 37.08 | 41.72 | 2.97 | 3.46 | 4.24 |
| SF+ Fiber+ IC | 27.58 | 30.27 | 32.36 | 3.21 | 3.41 | 4.52 |
| | Elastic Modulus (GPa) | | | Split T | ensile Strength | (MPa) |
|-------------------|-----------------------|-------|-------|---------|-----------------|-------|
| Mixture Type | 1-Day | 3-Day | 7-Day | 1-Day | 3-Day | 7-Day |
| FA+ SF | 33.55 | 37.88 | 39.61 | 3.37 | 3.52 | 4.48 |
| FA & SF+IC | 28.32 | 34.16 | 36.23 | 3.39 | 3.84 | 4.55 |
| FA & SF +Fiber | 30.72 | 36.8 | 39.99 | 3.33 | 3.5 | 4.49 |
| FA+ SF +Fiber +IC | 28.30 | 34.64 | 35.22 | 3.84 | 4.18 | 4.45 |

Table 40: Elastic Modulus and Split Tensile Strength for Mixtures with Fly Ash & Silica Fume

5.2.1.3 Crack Width and Crack Area Results

The average crack width, crack area and population standard deviation of crack widths are shown in Table 41 to Table 44. The following conclusions can be drawn from these data:

- The reference mixes showed the highest values of average crack width and crack area.
- The average crack width and crack area were reduced in the internally cured mixes.
- The combination of pre-wetted perlite and polypropylene fiber leads to a better reduction of plastic shrinkage cracking than either of these materials alone; however, polypropylene fibers reduced the plastic shrinkage potential in the mortar specimens more than the pre-wetted perlite.

| Mixture Type | Average Crack Width (mm) | Average Crack Area (mm ²) | Standard deviation of Crack Width |
|------------------|--------------------------|---------------------------------------|-----------------------------------|
| Plain | 0.539 | 4.325 | 0.2 |
| Plain+ IC | 0.535 | 4.266 | 0.2 |
| Plain +Fiber | 0.336 | 2.675 | 0.1 |
| Plain+ IC+ Fiber | 0.287 | 2.264 | 0.2 |

Table 41: Crack Width and Area for Plain Mix

Table 42: Crack Width and Area for Fly Ash Mix

| Mixture Type | Average Crack Width (mm) | Average Crack Area (mm ²) | Standard deviation of Crack Width |
|---------------|--------------------------|---------------------------------------|-----------------------------------|
| FA | 0.971 | 5.854 | 0.3 |
| FA +IC | 0.48 | 5.402 | 0.3 |
| FA +Fiber | 0.584 | 4.083 | 0.2 |
| FA +Fiber +IC | - | - | - |

Table 43: Crack Width and Area for Silica Fume Mix

| Mixture Type | Average Crack Width (mm) | Average Crack Area (mm ²) | Standard deviation of Crack Width |
|---------------|--------------------------|---------------------------------------|-----------------------------------|
| SF | 0.763 | 6.14 | 0.2 |
| SF+IC | 0.668 | 5.402 | 0.2 |
| SF +Fiber | 0.691 | 3.844 | 0.3 |
| SF+ Fiber+ IC | 0.461 | 3.697 | 0.2 |

| Mixture Type | Average Crack Width (mm) | Average Crack Area (mm ²) | Standard deviation of Crack Width |
|-------------------|--------------------------|---------------------------------------|-----------------------------------|
| FA+ SF | 0.795 | 6.323 | 0.2 |
| FA & SF+IC | 0.638 | 4.768 | 0.2 |
| FA & SF +Fiber | 0.577 | 3.844 | 0.2 |
| FA+ SF +Fiber +IC | 0.389 | 3.154 | 0.2 |

Table 44: Crack Width and Area for Fly Ash and Silica Fume Mix

5.2.2 Autogenous Shrinkage

The second series of tests performed on the mortar mixes was autogenous shrinkage. The data obtained from these tests are shown below.

5.2.2.1 Autogenous Shrinkage Results

Autogenous shrinkage strain and the change in mass of the corrugated tubes were measured based on ASTM C 1698. The mass of the corrugated tubes did not change, which is an indication that no moisture was lost from or transferred to the specimens. The autogenous shrinkage strain results are shown in Table 45 an Table 46, which show the results for the mixes without and with pre-wetted perlite. The shrinkage strain for mixes that were not internally cured was negative, which is an indication of contraction in these specimens. The shrinkage strain for the internally cured mixes was positive, which indicates that these specimens underwent swelling.

| | Plain | FA | SF | FA + SF |
|---------|----------|---------|----------------|---------|
| 1- Day | -229.67 | -63.73 | -439.82 | -322.41 |
| 2- Day | -614.43 | -60.47 | -487.12 | -718.17 |
| 3- Day | -850.91 | -83.09 | <u>-434.57</u> | -760.86 |
| 7- Day | -1358.05 | -161.27 | -500.52 | -780.69 |
| 14 -Day | -1627.57 | -201.50 | -718.20 | -809.46 |

Table 45: Autogenous Shrinkage Strain for mixes without Pre-Wetted Perlite

Table 46: Autogenous Shrinkage Strain for mixes with Pre-Wetted Perlite

| | Plain + IC | FA + IC | SF + IC | FA + SF + IC |
|---------|------------|---------------|---------------|---------------|
| 1- Day | 176.45 | 249.17 | 313.58 | 396.29 |
| 2- Day | 246.49 | 607.54 | 511.20 | 610.46 |
| 3- Day | 312.74 | 929.00 | 517.01 | 616.90 |
| 7- Day | 461.41 | <u>686.52</u> | <u>476.63</u> | <u>541.46</u> |
| 14 -Day | 643.50 | 1393.50 | <u>423.31</u> | <u>586.19</u> |

Both shrinkage and swelling data for specimens with and without internal curing showed a continuous pattern of increase except for the few cases underlined in Table 45 and Table 46. This change in shrinkage or swelling pattern could be due to measurement errors. Two examples of this type of error are shown in Figure 43 and Figure 44. In some of the corrugated moulds, voids such as the one shown in Figure 43 were observed. The low workability of the mortar mixes, particularly the mixes with silica fume, made placing and compacting the mortar in the moulds very difficult. In addition, some of the corrugated moulds after final set started to curve (Figure 44); therefore, some error must be considered in the measured length.



Figure 43: Void in Corrugated Mould



Figure 44: Curvature of Corrugated Tube

In order to better describe the effect of each SCM addition on autogenous shrinkage strain and its rate of occurrence, compared to the plain mix, the data from and Table 46 are plotted in Figure 45 and Figure 46.



Figure 45: Autogenous Shrinkage Strain for Not Internally Cured Mortar Mixes The results obtained for mixes that were not internally cured indicated that:

- The control mix underwent the maximum amount of autogenous shrinkage.
- The "FA" mix underwent the least amount of shrinkage; the rate of shrinkage for this mix stayed almost constant.
- The "SF" mix shrinkage rate was slow up to day seven; afterwards, the rate of shrinkage increased.
- The "FA+ SF" mix shrinkage rate was rapid up to day two; it then slowed down to an almost constant rate subsequently.



Figure 46: Autogenous Shrinkage Strain for Internally Cured Mortar Mixes The results for the internally cured mixes indicated that:

- Addition of pre-wetted perlite to the plain mix resulted in a constant swelling.
- Addition of pre-wetted perlite impacted the "FA+ IC" mix the most. This was expected as the "FA" mix⁹ only underwent a mild autogenous shrinkage; therefore, the addition of pre-wetted perlite caused this mix to swell significantly compared to the other mixes, until 3 days after final set. The rate of swelling from 3 to 7 days decreased rapidly, and then experienced a rapid rate of swelling after 7 days.
- "SF+ IC" and "FA+ SF+ IC" mixes exhibited the same rate of swelling until 7 days after final set. The autogenous shrinkage strain curves for these two mixes were parallel up to this time. After 7 days, the rate of expansion for the "SF+ IC" mix started to decrease, while for the "FA+ SF+ IC" mix it started to increase.

⁹ "FA" mix is the control mix for "FA+IC" mix

Chapter Six: Discussion and Data Analysis

In this chapter, the plastic and autogenous shrinkage results are discussed.

6.1 Early Age Crack Analysis

Early age cracking is due both to the interaction of concrete with its surrounding environment, and internal physical and chemical changes in the cement paste. According to Mihashi et al (Mihashi and Leite 141-154) strength development, the progress of autogenous and drying shrinkage, creep and relaxation are all factors that affect early age cracking, as indicated by Equation 22.

$\varepsilon_T = \varepsilon_{elastic} + \varepsilon_{creep} + \varepsilon_{shrinkage} + \varepsilon_{thermal}$

Equation 22

In this study the role of plastic shrinkage on early age cracking was examined, and the effects of mixture proportioning, internal curing and fiber reinforcement on reducing plastic shrinkage cracking were studied on concrete and mortar. These strategies completely eliminated plastic shrinkage cracking in the concrete panels. Table 47 shows the relative effects of these methods on reducing plastic shrinkage cracking in the mortar specimens. The results show that the maximum crack area reduction was due to a combination of pre-wetted perlite and polypropylene fibers, followed by polypropylene fibers and then pre-wetted perlite alone.

| | IC | Fiber | IC+ Fiber |
|--------|------|-------|-----------|
| Plain | -1% | -38% | -48% |
| FA | -8% | -30% | -100% |
| SF | -12% | -37% | -40% |
| FA+ SF | -26% | -39% | -50% |

Table 47: Crack Area Percentage Difference Compare to the Reference Mix

The one day elastic modulus for the mortar specimens is shown Table 48. Although the mixes with pre-wetted perlite had a lower one day elastic modulus, the crack area for these mixes was higher than for the mixes with polypropylene fibers. The mixes with a combination of polypropylene fiber and pre-wetted perlite had the lowest one day elastic modulus and crack area. Therefore, fibers play an important role in reducing plastic shrinkage cracking in mortar. Pre-wetted perlite, by providing additional water to a selfdesiccating paste, reduces the capillary pressure. Fibers are more effective than pre-wetted perlite because by bridging cracks they transfer stress from one side of the crack to the other.

Control FA SF FA+ SF Plain 31.59 37.18 31.21 33.55 IC 28.75 27.74 28.7 28.32 Fibre 33.47 31.48 31.21 30.72 IC+ Fibre 23.37 27.58 28.76 28.30

Table 48: 1-Day Elastic Modulus for Mortar Mixes

6.2 Autogenous Shrinkage Data Analysis

As mentioned earlier, the reason for the occurrence of autogenous shrinkage prior to final set is chemical shrinkage. Before final set, the capillary pressure in the capillary pores due to loss of water creates a strain in the cement paste. This strain is the autogenous shrinkage due to the chemical shrinkage. This type of autogenous shrinkage, which is also referred to as early age autogenous shrinkage, is highly dependent on the cement paste microstructure, Figure 47. Parts (a) and (b) of this figure, respectively, represent the spaces between cement grains in the NSC and HSC cement paste microstructure. As mentioned earlier, capillary pressure has an indirect relationship to the space between the cement grains; therefore, the dense microstructure of HSC results in high capillary pressure in the cement paste, leading to early age autogenous shrinkage in HSC (Holt 446).



Figure 47: NSC and HSC Cement Paste Microstructure (Holt 446)

After final set, during the hardening phase a skeleton is formed in the cement paste which allows it to resist the stresses due to the chemical shrinkage; therefore, as shown in **Error! Reference source not found.**, the autogenous and chemical shrinkage curves are o longer parallel. Autogenous shrinkage after final set, which is also referred to as long term autogenous shrinkage, is due to self-desiccation. In this phase, the water required for hydration of the cement is supplied by moisture from the air voids in the cement paste (Holt 446).

The autogenous shrinkage strains obtained by following ASTM C 1698 are long term autogenous shrinkage, as the first measurement is done at the time of final set. From the data obtained it can be concluded that replacing 25% of the fine aggregates with prewetted perlite can completely eliminate autogenous shrinkage in the mortar mixes. Therefore, these saturated perlite particles were able to provide enough water for the cement paste to prevent self-desiccation.

Comparing Figure 45 and Figure 46, it can be seen that the response of the IC mixes to autogenous shrinkage was opposite to that of mixes without IC. This concept is illustrated in Table 49.

| Mix Not Internally Cure (Shrinkage) | | Internally Cured (Swelling) | |
|---------------------------------------|--------------------------------|-------------------------------|--|
| Plain | Highest Shrinkage | Lowest Swelling | |
| FA | Lowest Shrinkage | Highest Swelling | |
| FA+ SF | Higher shrinkage than "SF" mix | Lower swelling than "SF" mix | |

Table 49: Autogenous Shrinkage behavior of mixes with and without pre-wetted perlite

Mixes with SCM addition underwent less autogenous shrinkage than the plain mix. The percentage differences between the "FA", "SF" and "FA+SF" mixes and the "plain" mix are shown in Table 50. The obtained data showed that, the addition of fly ash to the plain mix on average reduced autogenous shrinkage by 86%. The addition of silica fume, however, increased the amount of shrinkage in the first day though, after that time it reduced the amount of shrinkage compared to the plain mix. However, since during the first 24 hours concrete does not have enough tensile strength to resist the induced internal stresses, this could result in worse internal micro-cracking in the silica fume mixes compare to the plain mix. In the "FA+ SF" mix, the addition of fly ash to the silica fume mix reduced the first day autogenous shrinkage compared to the "SF" mix, but there was still an increase in its percentage difference compared to the plain mix.

Table 50: Percentage Difference of autogenous shrinkage in the SCM included Mixes compare toPlain Mix

| | FA | SF | FA+SF |
|---------|-----|-----|-------|
| 1- Day | -72 | 92 | 40 |
| 2- Day | -90 | -21 | 17 |
| 3- Day | -90 | -49 | -11 |
| 7- Day | -88 | -63 | -43 |
| 14 -Day | -88 | -56 | -50 |

6.2.1 Theoretical Amount of Internal Curing Water Required

As discussed earlier, both under-and over-curing problems associated with the internal curing method are due to lack of knowledge about the amount of internal curing water required to completely eliminate autogenous shrinkage. For this study, the percentage of sand replacement by with pre-wetted perlite was chosen based on assumptions from previous studies. However, in practice, where time and financial concerns do not allow a trial and error approach, it is desirable to be able to determine theoretically the required IC water.

As discussed in Section 2.2.1.1, a hydrating cement paste consists of: pores due to chemical shrinkage, capillary water, gel water, gel solid and hydrated and unhydrated cement. Power's model provides theoretical formulae, to determine the relative volumes of these phases in a hydrating and air free cement paste (Equations 23 to 27), in which "p" and " α " respectively represents initial porosity and degree of hydration (Jensen and Hansen 647-654). These equations are later used to calculate the required amount of internal curing water.

$$p = \frac{\left(\overline{c}\right)}{\left(\frac{w}{c} + \frac{\rho_{w}}{\rho_{cement}}\right)} \text{ [Initial Porosity]}$$
Equation 23

$$V_{cs} = 0.20 (1 - p)\alpha \text{ (Chemical Shrinkage)}$$
Equation 24

$$V_{cw} = p - 1.32(1 - p)\alpha \text{ (Capillary Water)}$$
Equation 25

$$V_{gw} = 0.60(1 - p)\alpha \text{ (Gel Water)}$$
Equation 26

$$V_{gs} = 1.52(1 - p)\alpha \text{ (Solid Gel)}$$
Equation 27

$$V_{uc} = (1 - p)(1 - \alpha) \text{ (Unhydrated Cement)}$$
Equation 28

(W)

Jensen et al (Jensen and Hansen 647-654)considered internally cured concrete as a type of "water-entrained" concrete in which the extra water allows the cement to reach a higher degree of hydration in low w/c ratio mixes. As discussed in Section 2.2.1.1, the internal curing water which is considered as an external source of water for the hydrating cement paste must fill the 8% pores in the cement paste due to chemical contraction, which

causes self-desiccation, shown in Figure 48. Hence, comparing Figure 48 with Figure 49 shows that in an internally cured mix, if the pores due to chemical contraction are filled by internal curing water, self-desiccation can be prevented.



Figure 48: Schematic Hydration of Mix with w/c =0.42 (Mindess and Aïtcin 186)





The amount of IC agent added to a high strength mix is of great importance, because compressive strength and durability are decreased with an increase in the porosity. Jensen et al (Jensen and Hansen 647-654) based on Power's model, developed a set of equations to obtain the required amount of internal curing water. Equation 29 is based on the assumption that the maximum degree of hydration is achieved. Hence

$$V_{gw} + V_{gs} + V_{cw} = 1$$
 Equation 29

By substituting Equations 25 to 27 into Equation 29, the maximum degree of hydration for a low w/c ratio mix is obtained, Equation 30.

$$\alpha_{max} = \frac{p}{1.12 \ (1-p)}$$
 Equation 30

The initial volume of internal curing water should be the same as the volume of empty pores in the cement paste at the maximum degree of hydration, Figure 49. Therefore, by substituting Equation 30 into Equation 24, a relationship between the internal curing water volume and the capillary water volume is obtained for mixes with w/c ratio below 0.36, Equation 31.

$$(w/c)_{IC water} = 0.18 (w/c)$$
 for $\frac{w}{c} < 0.36$ Equation 31

Equation 32 implies that hydrating cement paste with a w/c ratio in the range of 0.36 to 0.42 can achieve full hydration; however, when the w/c ratio of a mix is below 0.42, the difference in the amount of water added to the mix and w/c=0.42 must be provided by an external source of water which, in the case of internally cured mixes, is the internal curing water.

$$(w/c) + (w/c)_{IC water} = 0.42$$
 Equation 32

$$(w/c)_{IC water} = 0.42 - (w/c)$$
 for $0.36 \le \frac{w}{c} \le 0.42$ Equation 33

The equations developed by Jensen et al (Jensen and Hansen 647-654) were applied to the mixes used in this study to calculate the theoretical amount of internal curing needed to eliminate the autogenous shrinkage. Since the w/c ratio of mixes used for this study was 0.36, Equation 33 was applicable. The required amount of internal curing water needed per gram of cement was found to be 0.06 g, Equation 34. Figure 50 shows the minimum amount of IC water needed to eliminate autogenous shrinkage based on Equations 31 and 33.

$$(w/c)_{IC water} = 0.42 - 0.36 = 0.06 \text{ g}$$
 Equation 34



Figure 50: IC water needed for low w/c ratio mixes (Jensen and Hansen 647-654)

An example calculation used to determine the theoretical amount of water for the "Control+ IC" mix is shown in Equations 35. This calculated value was then compared with the amount of IC water that was added to the mix, Equation 36. Table 51 illustrates the "Control+ IC" mix design.

| | Control+ IC |
|----------------------------------|-------------|
| Sand | 25.7 |
| Cement (Kg/m ³) | 13.42 |
| Fly Ash (Kg/m ³) | |
| Silica Fume (Kg/m ³) | |
| Perlite | 1.13 |
| Superplasticizer (ml) | 43 |

Table 51: "Control+ IC" Mix Design

$$\frac{0.06 \ g \ IC \ water}{1 \ g \ cement} = \frac{x}{13420 \ g \ cement} \Rightarrow x = 805.2 \ g \ IC \ water$$
Equation 35
$$\frac{22.5 \ g \ water}{100 \ g \ perlite} = \frac{x}{1130 \ g} \Rightarrow x = 254.25 \ g \ IC \ water$$
Equation 36

The above calculations showed that theoretically 805.2 g of IC water was needed to eliminate autogenous shrinkage in the "Control+ IC" mix. However, addition of 25% of prewetted perlite to the mix provided 254.25 g of IC water for the mix; which is 68% less than the theoretical amount of IC water. Based on the theoretical calculation, in order to eliminate autogenous shrinkage in the "Control+ IC" mix, 83% of the sand would have had to be replaced by pre-wetted perlite¹⁰.

6.3 Recommendations

Plastic and autogenous shrinkage procedures that were used here followed ASTM C 1579 and ASTM C 1698, respectively. The problems that were faced while using these two standards are given below.

- ASTM C 1579 requires some modifications as it does not provide information for dimension of the chamber or the type of fan that is required to create the proposed harsh curing conditions by it.
- The gauge of the autogenous shrinkage equipment based on ASTM C 1698 is very sensitive to the orientation of the corrugated moulds. Any misalignment can result in a different reading from the previous one. This leads to imprecise determination of length change. The errors associated with orientation of samples on the

¹⁰ The calculations are shown in Appendix B

equipment can be reduced if instead of removing them from the equipment after each reading; the samples are placed on the equipment and the change in length are measured via a digital data logger at specific time intervals.

6.4 Future Studies

A few topics could be elaborated in any future studies on internal curing method in high strength concrete.

- The rate of water desorption from an IC agent at various RH levels should be measured. Also, the length that this IC water can travels in the paste.
- Another study on internally cured concrete which was retrieved from the tidal zone of a marine exposure with silica fume and w/c ratio of 0.33 showed good chloride ion penetration resistance in comparison with the plain normal strength concrete which is usually used in a marina (Kovler and Zhutosky). Further studies on chloride ion penetration of internally cured high strength concrete are needed.
- Different percentages of pre-wetted perlite must be tested in order to find an optimum percentage replacement for the amount of sand to be replaced.

Chapter Seven: Conclusions

In this study the effect of mix proportioning, fiber reinforcement, and internal curing on reducing or eliminating plastic shrinkage of high strength concrete and mortar specimens was studied. In addition, the effect of pre-wetted perlite on reducing or eliminating autogenous shrinkage was studied on mortar.

To study the effects of pre-wetted perlite and polypropylene fibers on reducing plastic shrinkage in high strength concrete, four mix designs were used to compare and contrast the effectiveness of these two materials. A control mix was used for baseline comparison of subsequent results. Fiber effects were studied in a second mix containing 0.15% polypropylene fibers by volume. The effect of pre-wetted perlite was studied in a third mix containing 25% perlite by volume of fines. A fourth and final mix contained both 0.15% fibers by volume of concrete and 25% perlite by volume of fines. These mixes were all cast into panels (ASTM C 1579) and numerous 10×20 cm cylinders. Crack analysis and ultrasonic pulse velocity (UPV) tests were conducted, as well as compression tests at 1, 3, 7 and 28 days. The cylinder densities and pulse velocity measurements were used to analytically calculate the elastic modulus of the specimens.

The result showed that both fiber reinforcement and internal curing were effective materials in mitigating plastic shrinkage as both these materials resulted in fully eliminating plastic shrinkage cracking on the concrete panels. The "Early age shrinkage test" was followed for studying plastic shrinkage on mortar specimens. The "FA¹¹" mix

¹¹ FA: Plain mix with 7% Fly Ash

showed maximum crack width compare to "plain", "SF¹²" and "FA+SF¹³" mixes. Internal curing was most effective in the "FA+SF" mix by reducing the crack area by 26% compared to the plain mix. Fiber reinforcement reduced the crack area in "plain¹⁴", "FA", "SF" and "FA+SF" mixes approximately by 30%. The most effective method in reducing plastic shrinkage cracking in mortar specimens was the addition of both pre-wetted perlite, as the internal curing agent, and polypropylene fiber. The combination of these two materials completely eliminated plastic shrinkage in the "FA" mix specimens; also, in "Plain", "SF" and "FA+SF" mixes it reduced the crack area by approximately 50%.

The result showed that addition of fibers and pre-wetted perlite decreases the compressive strength of concrete mixes. The percentage difference of 28 days compressive strength of "Fiber¹⁵", "Fiber +Perlite"¹⁶ and "Perlite"¹⁷ concrete mixes with the control concrete mix was, respectively 7%, 33% and 39%. Elastic modulus of concrete mixes, aside from IC mix which showed a 12% decrease, was almost the same.

To study the impact of internal curing on reducing autogenous shrinkage in mortar, pre-wetted perlite as an internal curing agent was used. Eight different mixes were tested for autogenous shrinkage. The first set of four did not contain pre-wetted perlite; in the second set 25% by volume of fine aggregates was replaced by pre-wetted perlite of 1.5 mm particle size. One mixture contained fly ash, one contained silica fume and one mixture

¹² SF: Plain mix with 8% silica fume

¹³ FA+SF: Plain mix with 7% fly ash and 8% silica fume

¹⁴ Plain: Mix with no cementitious materials

¹⁵ Fiber: The concrete mix with polypropylene fibers

¹⁶ Fiber+ Perlite: The concrete mix with polypropylene fibers and pre-wetted perlite

¹⁷ Perlite: The concrete mix with pre-wetted perlite

contained a combination of both of these supplementary cementing materials. The addition rates of fly ash and silica fume were kept constant, respectively at 7% and 8%. These mixtures were cast into corrugated plastic tubes (ASTM C 1698) and 100 by 200 mm cylinders. The autogenous shrinkage strain of the mortar in the corrugated plastic tubes was measured at final set, and at 1, 2, 3, 7 and 14 days. Ultra-sonic pulse velocity tests and split cylinder tests were performed to determine the elastic modulus and split tensile strength of the mortar mixtures at 1, 3 and 7days.

The results showed that replacing 25% of the fine aggregates with pre-wetted perlite by volume completely eliminated autogenous shrinkage in the mortar mixes. The "FA" mix without IC underwent the lowest amount of autogenous shrinkage; also, the highest amount of swelling was related to the "FA" mix with IC agent. While the result showed that replacing 25% of fine aggregates with pre-wetted perlite is sufficient for eliminating autogenous shrinkage, the theoretical calculations showed that about 80% of fine aggregates were required to be replaced by pre-wetted perlite in order to eliminate autogenous shrinkage in the mixes. This difference between the practical and theoretical values for the required amount of IC agent for a mix could be due to the fact that theoretical formulae do not consider factors, such as rate of water desorption from an IC agent, pore size and high open to close porosity ratio. In addition, the elastic modulus of fiber and mixes containing pre-wetted perlite showed lower elastic modulus in comparison with the control or the mortar mix included fibers and pre-wetted perlite. The split tensile strength stayed almost constant among the eight different mortar mixes.

Appendix A: Mix Design Calculation

A-1 Concrete Fiber-Perlite Mix Design

Calculation for Mass of Fiber:

$$X m^{3} fibre = 0.15\% \times (Concrete or Mortar Volume m^{3}) = X m^{3}$$
$$Y kg fibre = (X m^{3}) \left(\frac{900 kg}{m^{3}} (Polypropylene Density)\right) = Y kg$$

Calculation for Mass of Perlite:

$$X m^{3} fines = (Sand kg) \left(\frac{1 m^{3}}{1600 kg(Sand Density)}\right) = X m^{3}$$
$$Y m^{3} perlite = 25\% \times (X m^{3}) = Y m^{3}$$
$$Z kg perlite = (Y m^{3}) \left(\frac{270 kg(Perlite Density)}{m^{3}}\right) = Z kg$$

<u>Calculation for Reduction of Fines (75% of Total Fines Volume):</u>

$$X m^{3} fines = 75\% \times \left[(\text{Sand } kg) \left(\frac{1 m^{3}}{1600 \ kg} \right) \right] = X m^{3}$$
$$Y kg fines = (X m^{3}) \left(\frac{1600 \ kg}{m^{3}} \right) = Y kg$$

A-2 Concrete Perlite Mix Design

Calculation for Mass of Perlite:

 $V_{sand} = \frac{M \ (known \ from \ the \ mixture \ porportion)}{\rho_{sand}}$

 $0.25 * V_{sand} = V_{perlite}$

 $M_{perlite} = V_{perlite} * \rho_{perlite}$

New Sand $Mass_{sand} = M_{sand} - M_{perlite}$

Appendix B: Theoretical IC Water Calculations

The following sets of calculations illustrate the percentage of sand that was needed to be replaced by pre-wetted perlite theoretically, in order to eliminate self-desiccation and therefore autogenous shrinkage in the "Control+ IC" mix. X and Y respectively represent mass of perlite and the replacement percentage of sand by pre-wetted perlite.

 $\frac{805 \text{ g IC water}}{x} = \frac{22.5 \text{ g IC water}}{100 \text{ g Perlite}} \Longrightarrow x = 3.58 \text{ Kg Perlite}$

 $\rho_{perlite} = \frac{m}{V} \Longrightarrow 270 \ {\binom{Kg}{m^3}} = \frac{3.58}{V} \Longrightarrow \ V_{Perlite} = \frac{3.58}{270} = 0.0133 \ (m^3)$

 $\rho_{Sand} = \frac{m}{V} \implies 1600 {\binom{Kg}{m^3}} = \frac{25.7}{V} \implies V_{Sand} = \frac{25.7}{1600} = 0.0161 (m^3)$

 $V_{Sand} \times Y = V_{Perlite} \implies Y = \frac{0.0133}{0.0161} * 100 = 83\%$

Appendix C: Concrete

Crack width Data and Crack Area Calculations

Table 52 exhibits the crack width and total crack area of the control concrete specimen. The crack area was calculated using the formula and the method that was explained in Section 4.6.1. Figure 51 illustrates few pictures that were taken by Moticam for measuring crack width from the control concrete specimen.

| Crack Width Number | mm | Crack Area (mm²) |
|--------------------|--------|------------------|
| 1 | 0.3305 | |
| 2 | 0.5101 | 0.4203 |
| 3 | 0.2801 | 0.3951 |
| 4 | 0.1816 | 0.23085 |
| 5 | 0.3709 | 0.27625 |
| 6 | 0.4141 | 0.3925 |
| 7 | 0.4001 | 0.4071 |
| 8 | 0.3461 | 0.3731 |
| 9 | 0.4467 | 0.3964 |
| 10 | 0.1677 | 0.3072 |
| 11 | 0.2354 | 0.20155 |
| 12 | 0.4781 | 0.35675 |
| 13 | 0.612 | 0.54505 |
| 14 | 0.4781 | 0.54505 |
| 15 | 0.593 | 0.53555 |
| 16 | 0.4608 | 0.5269 |
| 17 | 0.5101 | 0.48545 |
| 18 | 0.3469 | 0.4285 |
| 19 | 0.3293 | 0.3381 |
| 19 | 0.3289 | 0.3291 |
| 20 | 0.4115 | 0.3702 |
| 21 | 0.3458 | 0.37865 |
| 22 | 0.302 | 0.3239 |
| 23 | 0.2325 | 0.26725 |
| 24 | 0.1316 | 0.18205 |
| 25 | 0.2138 | 0.1727 |
| 26 | 0.1197 | 0.16675 |

Table 52: Crack Width and Area for concrete Specimen

| Average Crack Width 0.35 9.35 |
|-------------------------------|
|-------------------------------|

|--|



Figure 51: Concrete Specimen Crack Opening