

DURABILITY OF CONCRETE CONTAINING AGGREGATE PRODUCED FROM RECYCLED CONCRETE AFFECTED BY ALKALI-SILICA REACTION

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By

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ABSTRACT

DURABILITY OF CONCRETE CONTAINING AGGREGATE PRODUCED FROM RECYCLED CONCRETE AFFECTED BY ALKALI-SILICA REACTION

By

Chris Christidis
Master of Applied Science, 2006

Civil Engineering
Ryerson University

Minimal information is available on the durability of concrete containing demolished concrete as an aggregate. The purpose of this study was to: investigate and control the reactivity of recycled concrete aggregate (RCA) produced from concrete that was suffering from alkali silica reaction (ASR); and investigate other durability issues of concrete containing RCA with emphasis on salt scaling and freezing & thawing. It was deduced that the RCA produced from ASR-affected concrete caused expansion when used in new concrete as an aggregate. This high reactivity of the tested RCA was attributable to its relatively high alkalis and calcium contents. Furthermore, a greater amount of SCM was required to reduce expansion compared to concrete containing raw natural reactive aggregate. In terms of freezing and thawing and salt scaling, concrete containing coarse RCA at a 100% replacement level of coarse aggregate showed lower durability compared to concrete made with natural coarse aggregate.

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List of Abbreviations

AAR	Alkali Aggregate Reaction
ACR	Alkali Carbonate Reaction
ASR	Alkali Silica Reaction
ASSHTO	American Association of State Highway and Transportation Officials
ASTM	American Society for Testing Materials
CH	High Calcium
CI	Intermediate Calcium
CSA	Canadian Standards Association
C-S-H	Calcium Silica Hydrate
F	Low Calcium
FA	Fly Ash
GU	General Use
HA	High Alkali
LA	Low Alkali
MTO	Ontario Ministry of Transportation
PC	Portland Cement
RCA	Recycled Concrete Aggregate
RH	Relative Humidity
SCM	Supplementary Cementing Materials
SEM	Scanning Electron Microscope
SF	Silica Fume
SSD	Saturated Surface Dry
w/c	Water to Cementing Materials Ratio
Ca/Si	Calcium to Silica ratio

Chapter 1

Introduction

1.1 General

Environmental issues resulting from construction waste are a major concern. The catch phrase “reuse, reduce, and recycle” is gaining further attention and should be implemented at all construction phases. In order to implement these measures encouragement for sustainable development is required. Sustainable development ensures that the resources used today will not compromise the ability for future generations to meet their needs.

Each year 200 million tons of construction waste is continuously discarded in landfills (Gomez-Soberon, 2002). Most of the construction material used can be recycled, thus reducing the accumulation of waste directed towards landfills. These include steel and polymers which can be extracted and processed to form new products. Concrete is of great concern when considering wasted materials due to its contribution as the largest portion of the total debris after demolition, for example, when an apartment is demolished concrete amounts to approximately 30 to 40% of the wreckage (Park, 2003).

Deterioration is a major issue when dealing with concrete. The environment and chemicals constantly attack concrete structures. For example, in Canada freezing

and thawing cycles occur several times in a year, during these cycles the addition of de-icer salts applied to pavements to remove ice causes damage if proper precautions are ignored. Sometimes, the constituents of concrete can produce a damaging reaction without severe environmental conditions or exposure to chemicals. For instance, alkalis inside the concrete react with some forms of silica in aggregates producing expansion. In severe cases repairs are more difficult to implement and removal may be the only solution.

With time structures deteriorate to a point where repairing the structure is not cost effective, and as a result the structure will need to be replaced. Structures are also demolished and replaced by other structures, such as an apartment building being replaced by a housing development. The debris that remains after demolition must be removed and taken to landfills.

As landfills become crowded and become more heavily regulated, it makes sense to seek alternative means of disposal of these materials from construction and demolition operations. Furthermore, recycling concrete as aggregate is an alternative option, which would reduce the consumption of natural and non-renewable resources, such as stones. Therefore, recycling aggregates would be beneficial in all respects. Although the use of recycling aggregates is mostly used in road bases, the goal is to increase the use of recycled concrete aggregates in the

field of concrete either as full or partial replacement of the aggregates. The problem lies, however, in making sure that the produced concrete is durable.

1.2 Objective

The purpose of this study is to determine if deterioration continues in newly formed concrete containing RCA; at what extent does deterioration occur; and can the deterioration be controlled. This study focused on deterioration due to alkali silica reaction. This was accomplished by testing aggregates that were produced from concrete that was deteriorating from alkali silica reaction. Several preventative measures were incorporated. These included using various amounts of silica fume, fly ash, slag, ternary blends of silica fume and fly ash or silica fume and slag, low alkali cement, and pre-washing aggregates to remove excess alkalis. The use of high levels of supplementary cementing materials is prohibited by some organizations. Therefore, other issues included in this study are the effects of adding supplementary cementing materials to concrete containing recycled concrete aggregate exposed to salt scaling and freeze thaw conditions.

1.3 Scope

The scope of this study was to:

1. Control deterioration in concrete containing recycled concrete aggregates that were produced with alkali silica reactive concrete.
2. Investigate the effects of ASR preventative measures on salt scaling and freeze thaw resistance.

In this thesis Chapter 2 provides a literature review to gain a stronger background of the topics discussed in this thesis. Chapter 3 presents the materials and experimental program. Chapter 4 discusses the results obtained and possible reasons for the results. Chapter 5 examines the conclusions that can be obtained from the results and mentions possible recommendations for further studies. All graphs, tables, and numerical results not found in the main text of the thesis can be found in the applicable appendices.

Chapter 2

Literature Review

2.1 Recycled Concrete Aggregate (RCA)

2.1.1 General

Construction materials are constantly evaluated for their resistance to the environment. Eventually, it will be necessary for all construction materials to be replaced and disposed of due to deterioration. Recycling of concrete is becoming more important because it protects natural resources and eliminates the need for disposal.

The process of recycling concrete is easy to understand. It essentially involves demolishing an existing concrete structure, removing non-concrete materials, and then crushing the remaining concrete into aggregates with a specified size.

The quality of the concrete containing RCA greatly depends on the quality of the recycled material used. For example, concrete that was exposed to de-icer salts will most likely produce aggregates that contain salt ions (Prieur & Nikitine, 2006). Furthermore, concrete containing alkali silica deterioration could possibly retain the reactive gel when crushed into recycled aggregates. It is believed that the white substance seen in Figure 1 is alkali silica gel.

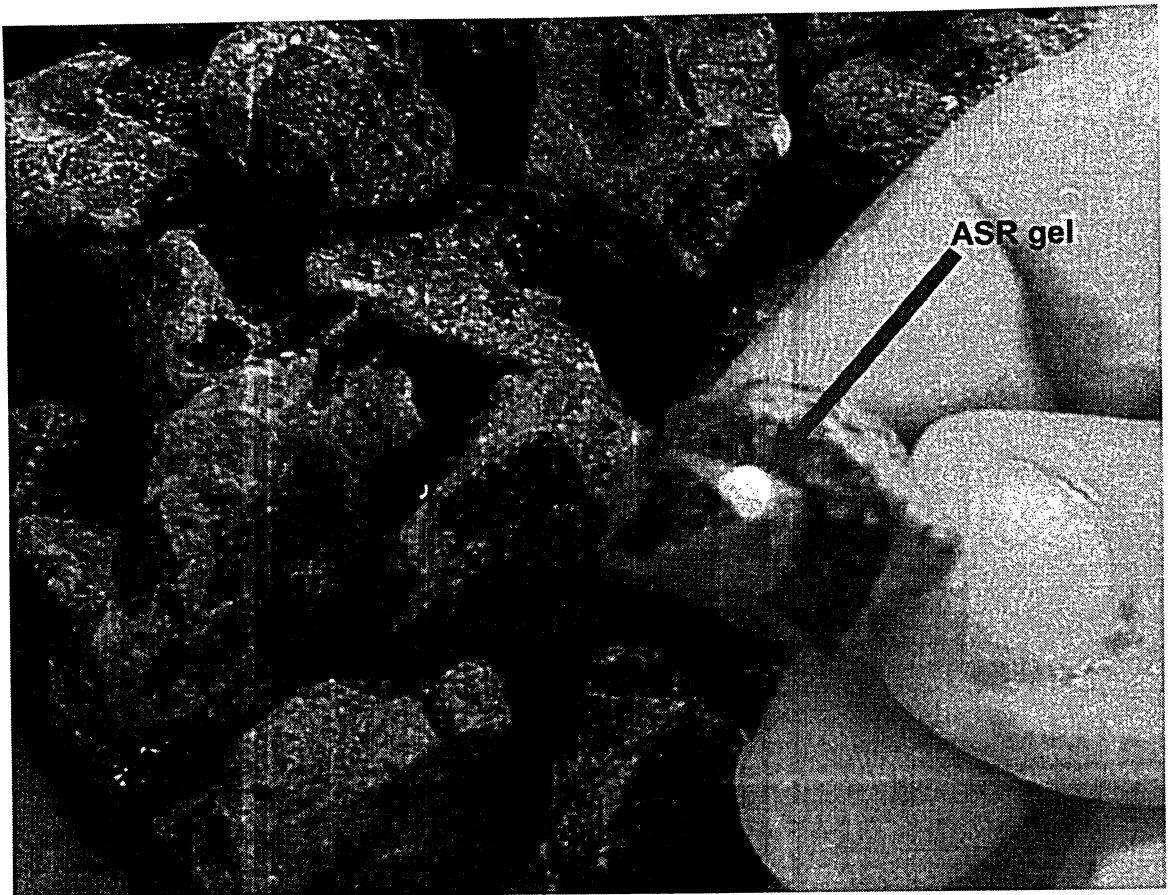


Figure 1: RCA examined in this study with white deposits believed to be ASR gel.

The removal of embedded material is necessary before using the recycled aggregates in fresh concrete, such as reinforcing steel. Great care must also be taken to prevent contamination by other materials, which can adversely affect concrete quality, such as asphalt, soil, glass, wood, and roofing materials (U.S. Army Corps of Engineers, 2004).

Recycling concrete from demolition projects can result in considerable savings. It can save the costs of transporting concrete to the landfill by as much as \$ 0.25 per

ton/mile and eliminate the cost of disposal by as high as \$100 per ton (Recycling Concrete, 2004). In addition, a profit can be made by selling these aggregates to a supplier.

For some years, extensive research has been conducted to determine the impact of using recycled building materials in concrete. Unfortunately, there is minimal data on the durability of concrete containing RCA. This is delaying the large-scale use of this economic and environmentally friendly material in the construction industry (Zaharieva et al., 2003).

2.1.2 Recycled Concrete Properties

An example of concrete containing recycled aggregate can be viewed in Figure 2. It is clear from the figure that when comparing a regular concrete mix with a concrete mix containing RCA, the concrete containing RCA has a reduced amount of natural aggregate present. This can be observed by comparing the dark aggregate with the lighter paste. However, it is difficult to distinguish between the old paste and new paste with this sample. It almost appears as if the sample shown in Figure 2 was batched with a reduced amount of aggregates and improper aggregate gradation.



Figure 2: A concrete prism cross section in this study containing RCA.

It should be restated that industrially produced recycled aggregates can contain various impurities. For example, concrete containing recycled concrete aggregates contaminated with wood could reduce the strength and durability of the concrete. Also, varying the properties of concrete that is recycled into RCA can influence the properties of concrete containing RCA (Zaharieva et al., 2003). Concrete containing RCA produced with air entrainment would generate different properties than concrete containing RCA produced with no air entrainment. The following are some typical examples of recycled concrete properties:

- Recycled coarse aggregate was found to have absorptions of 5% to 6%; whereas recycled fine aggregate absorptions was 9% to 10% (McGovern, 2002).
- The compressive strength of concrete decreases as the percentage of RCA in the concrete increases, as seen in Figure 3. At 100% RCA the reduction in strength could be as much as 80% compared to concrete without RCA (Topcu, 1997). Although concrete containing RCA has been demonstrated to have a lower compressive strength, it is possible to fabricate concrete containing RCA with equal to or greater compressive strength than the original recycled concrete. For example, concrete with a compressive strength of 60 MPa has been crushed and reused in new concrete to produce a concrete with 80 MPa (Ajdukiewicz & Kliszczewicz, 2002). However, this increased compressive strength tends to be only possible with the use of high strength recycled aggregates.
- According to Gomez-Soberon (2002) recycled aggregates have a lower density and higher porosity than natural aggregates. As the replacement level of the RCA increases, so does the porosity in the concrete containing these recycled aggregates (Gomez-Soberon, 2002). In addition, the increased porosity reduces the density of the RCA.

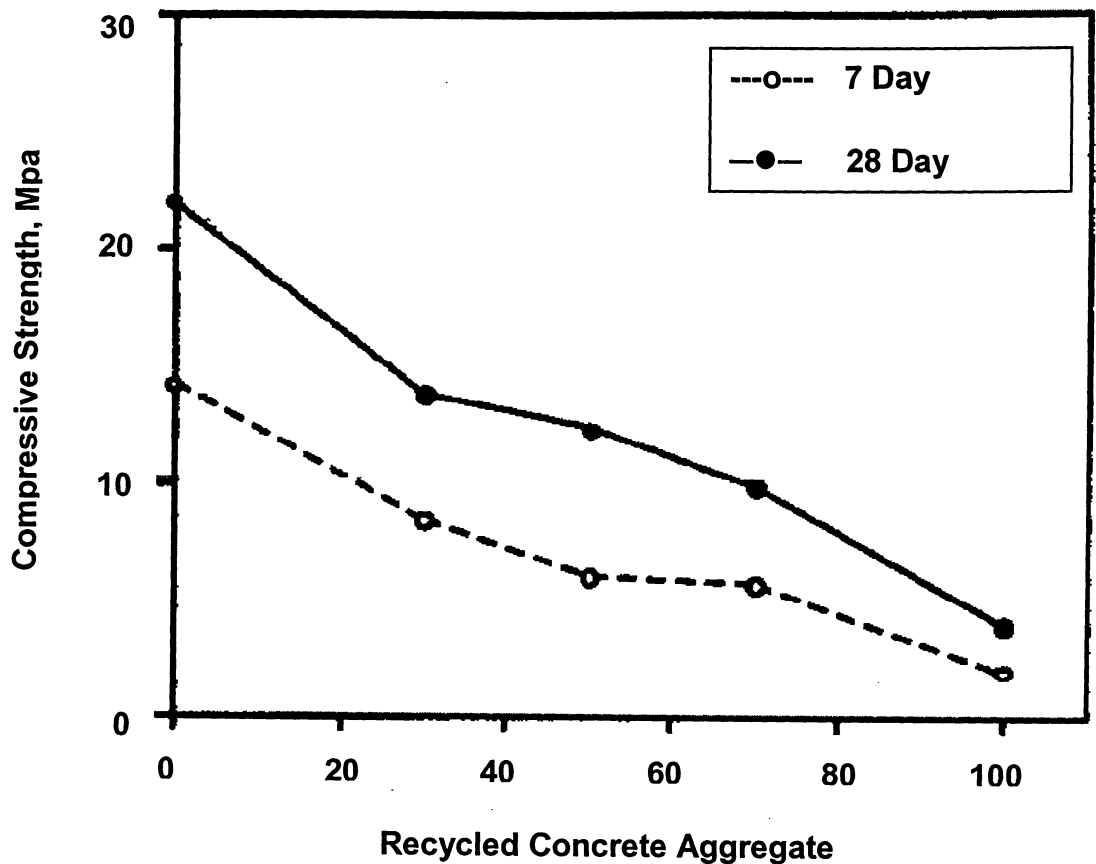


Figure 3: Changes in compressive strength due to varying replacement quantities of recycled aggregate (Topcu, 1997).

- The tensile strength is approximately 10% lower in concrete containing RCA than in concrete containing limestone aggregate (Ajdukiewicz & Kliszczewicz, 2002).
- With 100% recycled aggregate, the modulus of elasticity of concrete containing RCA was about 35% lower than the modulus of the reference concrete (McGovern, 2002).

- Drying shrinkage and creep of concrete are significantly increased by the use of recycled aggregate, particularly fine aggregate. After 2 years, the drying shrinkage of concrete made with 100% recycled aggregate was 60% to 100% greater than the reference concrete. Creep was up to 3.5 times greater (McGovern, 2002).
- When conducting the Chloride Penetration Depth Test on concrete containing RCA, the chloride penetration is similar to concrete with normal aggregates, as seen in Figure 4, except when using a 0.55 water to cement ratio (w/c) (Otsuki et al., 2003). In this case, the RCA is slightly greater in penetration depth. With carbonation penetration, the RCA is slightly greater at low and high w/c; however, with mid range w/c the carbonation depth of concrete containing RCA was almost double the normal aggregate, as seen in Figure 5 (Otsuki et al., 2003). In general the RCA concrete is less resistant to carbonation than the concrete with normal aggregates (Otsuki et al., 2003).

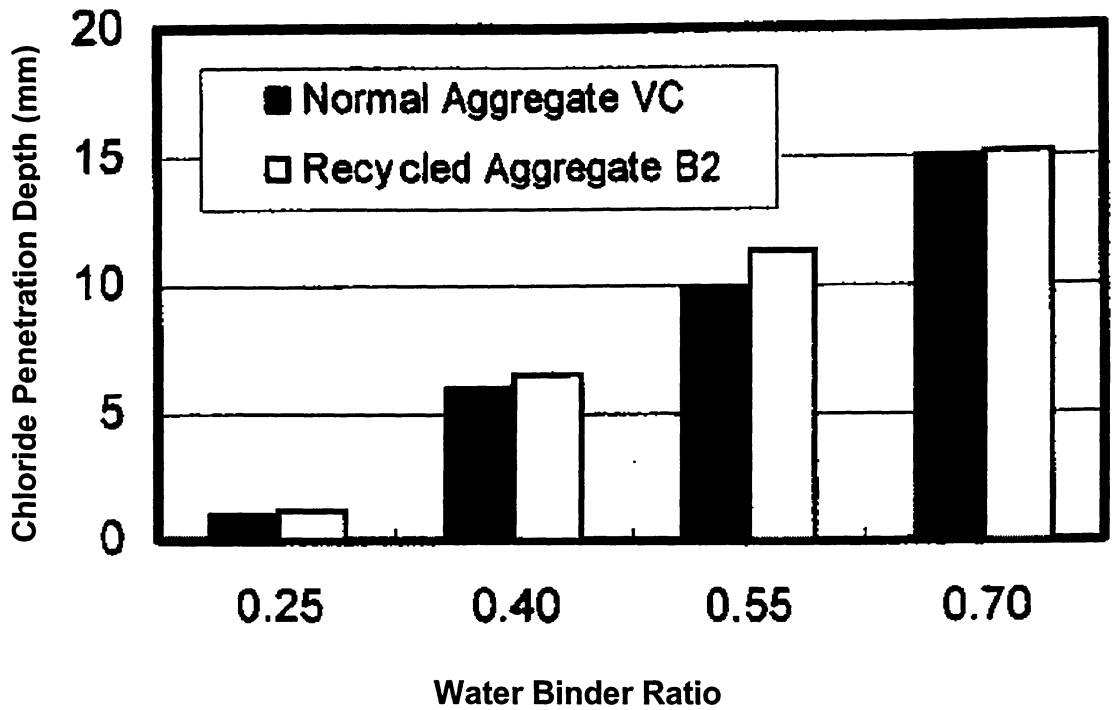


Figure 4: Chloride penetration depth of normal and recycled aggregate concretes (Otsuki et al., 2003).

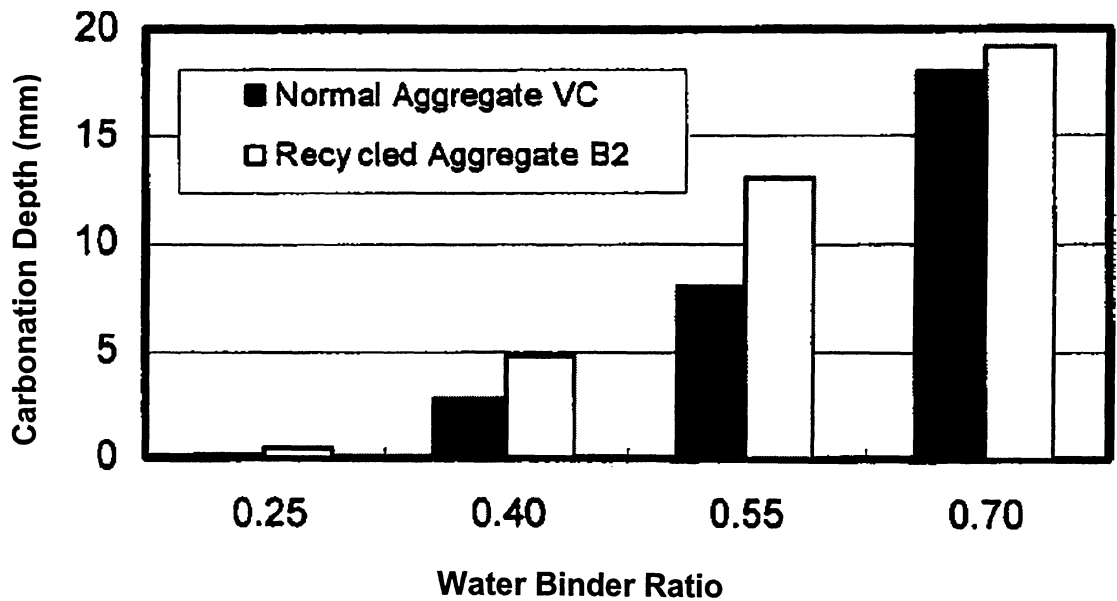


Figure 5: Carbonation depth of normal and recycled aggregate concretes (Otsuki et al., 2003).

- Recycled aggregate is less durable when considering the Los Angeles abrasion having a percentage loss ranging from 27% to 35%. Frost vulnerability and sulphate soundness have also shown that RCA produces a less durable concrete (Zaharieva et al., 2003).

The transition zone of the concrete that produces RCA could explain the disadvantages of using recycled aggregate, particularly in terms of strength. According to Otsuki et al. (2003), if the transition zone between the paste and aggregate in the RCA is stronger than the transition zone between the paste and RCA in concrete, then the strength of the concrete containing RCA is affected by the transition zone between the paste and RCA. However, if the strength of the transition zone between the paste and aggregate in the RCA is weaker than the strength of the concrete containing RCA is affected by the transition zone between the paste and aggregate in the RCA.

Some of the above properties pertain to tests done using laboratory produced recycled concrete aggregates. Laboratory produced aggregates are concrete samples that have been crushed with a small lab crusher; whereas, commercially produced RCA are crushed with a large crusher and stored in a stockpile. The storage of the RCA could result in further breakage of weaker aggregate particles causing smoother and more durable aggregates. Therefore, tests done using commercially produced recycled aggregates have shown to have slightly different

properties. For instance, the smoother RCA particles produced in a plant are slightly more workable than lab produced RCA. Also, tensile strength of commercially produced RCA has shown to have no significant reduction in strength during a period of 91 to 365 days (Sagoe-Crentsil et al., 2001).

2.1.3 Applications

As mentioned earlier, RCA is predominately used in road bases. According to a study by the Portland Cement Association (Concrete Technology, 2004), 38 states recycle concrete as an aggregate base; 11 recycle it into new Portland cement concrete (Concrete Technology, 2004). In general, RCA applications without any processing include:

- 1) Many types of general bulk fills
- 2) Bank protection
- 3) Base or fill for drainage structures
- 4) Road construction
- 5) Noise barriers and embankments

Removal of contaminants can be completed through selective demolition, screening, air separation, and size reduction into proper gradations. Crushed concrete can be used as:

- 1) New concrete for pavements,
- 2) Soil-cement pavement bases,
- 3) Lean-concrete or econo-crete bases, and
- 4) Bridge foundations

A trial study was conducted at the University of Central Florida's Circular Accelerated Test Track. The study was to test an asphalt base layer with RCA and a concrete pavement with RCA. Chini et al. (2001) discovered that using a base layer with RCA was within ASSHTO limits. The properties of RCA and concrete containing RCA obtained in the study were consistent and within the range of values obtained by other researchers. Despite having lower compressive strength than the concrete with natural stones, 100% RCA concrete had a 28-day compressive strength of 35 MPa, which was well above the target strength of 25 MPa.

2.2 Alkali Silica Reaction

2.2.1 General

Alkali aggregate reactions (AAR) are chemical reactions in concrete that can cause severe deterioration in concrete structures. The pore solution in concrete is a highly basic liquid. It consists of dissolved alkali hydroxides in the form of potassium and sodium, with minor amounts of other elements such as calcium and sulphates (Fournier & Berube, 2000). Some mineral phases within the coarse

and fine aggregates in concrete are chemically unstable. They react deleteriously when exposed to high pH environments. Two types of AAR are generally recognized: alkali carbonate reaction (ACR) and alkali silica reaction (ASR). The two types differ in the kind of mineral phases and mechanisms involved. ACR is caused by the dedolomitization of the aggregate and the only remedy available at the present time is to avoid using these aggregates (Neville, 1996). The cause of ASR is the reaction between amorphous silica and alkalis. This type of silica is found in some aggregates and the alkalis are provided, mainly, by the cement.

ASR can be considered to be a deterioration that affects the serviceability and the aesthetics of a structure, rather than its integrity. This is due to the crack width being between 0.1 mm to 10 mm and the crack depth being typically less than 25 mm (Neville, 1996). ASR can cause extreme deterioration problems when combined with other forms of deterioration, such as chloride attack (Neville, 1996). AAR related problems were first identified in Canada in the 1950s (Rogers et al., 2000). Since then, ASR has been recognized in more than 50 countries around the world. It is likely that the problems associated with ASR exist in a larger number of countries, but the deterioration of the concrete may have been attributed to other causes (Fournier & Berube, 2000). Extensive research has been carried out on ASR over the past six decades.

2.2.2 Alkali Silica Reaction Mechanisms

There have been many cases of ASR reported in Canada. The problem is fundamentally related to the increased solubility of amorphous, poorly micro or cryptocrystalline forms of silica in high pH solutions (Fournier & Berube, 2000). Two categories of ASR reactive rocks are recognized in Canada according to the silica form involved:

- 1) Rock types incorporating poorly crystalline or metastable silica minerals.

Concrete structures that incorporate even as little as 1 to 2% of these silica minerals have shown to greatly expand in ASR conditions (Fournier & Berube, 2000).

- 2) Rocks that contain very fine quartz grains or some variety of macro-granular quartz. This category is common in Eastern Canada. It is characterized by an initial delay of expansion, followed by deterioration cracks that can take 10 to even 25 years to become apparent in field conditions (Fournier & Berube, 2000).

In general, the mechanism for most alkali silica reactive Canadian aggregates can be explained by two processes. First, the reactive silica generally found in the aggregate combines with the alkali hydroxides found in the pore solution, which is predominately a product of the cement. This reaction produces alkali silica gel.

Secondly, the gel expands by absorbing moisture as water enters the concrete. The expansion that occurs is dependent on the characteristics of the gel. This process can also be explained as follow:

- Alkali hydroxide + reactive silica gel — — — ► reaction product (ASR gel)

- Gel reaction product + moisture — — — ► expansion

At first the gel is restrained from expanding into the cement paste. However, cracking occurs when the tensile stresses that are developed exceed the tensile strength of the cement paste as shown in Figure 6. As soon as adequate micro cracking has occurred, the gel spreads out freely through the cracks in the cement paste. It continues spreading, but gradually loses its expansive properties with the absorption of calcium. The calcium enters the system through an ion-exchange process with the hydrates of the paste (Fournier & Berube, 2000).



Figure 6: Illustration of expansion forces induced by expanding alkali-silica reactive siliceous limestone particles causing cracking of the particles and the surrounding cement paste (Fournier & Berube, 2000).

2.2.3 Conditions required for Alkali Silica Reaction

Three conditions are required in order to allow ASR to deteriorate concrete. They are:

- 1) **Presence of a reactive aggregate:** With the mixture proportions being constant, the reactivity level of alkali silica reactive aggregates will generally increase with an increased amount of reactive particles in the aggregate. The reactivity level will also increase as the aggregate particle size decreases, or in

other words increasing the surface area of the aggregate (Fournier & Berube, 2000).

Some fine and coarse aggregates with reactive particles of opal or cryptocrystalline quartz can display a pessimism effect (Fournier & Berube, 2000). For example, a maximum expansion can be obtained for a given proportion and size fraction of such reactive particles. It should be noted that all natural rocks react to some extent with the alkaline pore solution in concrete; however, the reaction does not necessarily produce deleterious expansion (Rogers et al., 2000).

- 2) **High concentration of alkali hydroxides in the concrete pore solution:** The alkali hydroxides found in the pore solution is normally related to the alkali content of the cement used. Aggregates generally expand in the concrete with an increased amount of total alkali content in the form of sodium hydroxides, as seen in Figure 7 (Shehata & Thomas, 2000). However, the amount of alkali content needed to initiate and sustain ASR varies from one aggregate to another (Fournier & Berube, 2000). Alkalies from sources other than the cement in concrete are:

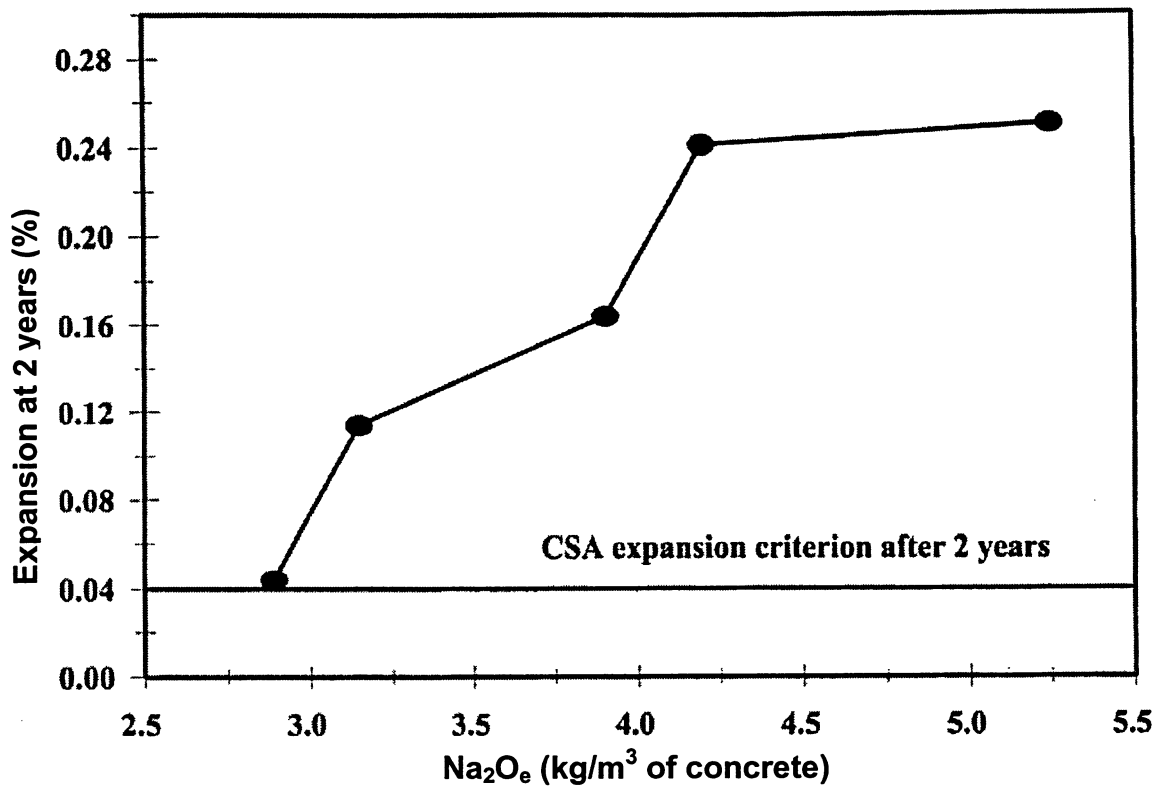


Figure 7: Effect of alkali content of concrete on the 2-year expansion of concrete prisms containing Spratt aggregate (Shehata & Thomas, 2000).

- **Aggregates:** alkalis contained in these aggregates can be expelled into the pore solution. This is particularly true with feldspar-rich aggregates (Berube et al., 2002).
- **Unwashed sea dredged sands:** these sands can contribute due to the salt content of these aggregates (Neville, 1996).
- **Chemical admixtures such as superplasticizers:** superplasticizers have been known to contain alkalis (Neville, 1996).

- **Mixture water:** again hard water can contain moderate levels of alkalis (Neville, 1996).
- **High alkali supplementary cementing materials:** Fly ashes (Shehata, & Thomas 2000), silica fume (Boddy et al., 2003), and Slag (Hooton, 2000) can be produced with high levels of alkalis, which contributes to the expansion of ASR.

These other sources can also contribute in raising the alkali hydroxides in the pore solution. Therefore, they can increase the chance of ASR deterioration in the presence of reactive aggregates. The Lady Evelyn Lake dam is an example of how low alkali concrete can counteract the deleterious affects of ASR. The dam was replaced in 1972 using an aggregate containing the same reactive rock type, but with low alkali cement ($\text{Na}_2\text{Oe} < 0.6$). After more than 25 years, there was no evidence of cracking in the concrete (Rogers et al., 2000).

- 3) **Presence of adequate amount of moisture** (Neville, 1996): Alkali-aggregate reactivity typically occurs in concretes that are exposed to internal relative humidity of approximately 80 to 85% (Neville, 1996). The alkali-silica gel needs water to swell, which causes expansive pressure on the concrete. Laboratory investigations have shown that partially dehydrated gel due to drying of the specimen can be rehydrated and expand if additional water is

supplied to the specimen (Fournier & Berube, 2000). However, dried and carbonated gels are unlikely to regain their expansive properties (Neville, 1996).

There are other factors that when combined with the conditions mentioned above can result in an increase or decrease in ASR expansion. These factors are:

- The type and degree of reactivity of the aggregates (Neville, 1996).
- The chemical properties of the components added to the concrete mix. These properties include: the type and composition of cement (Neville, 1996); concrete alkali content (Shehata & Thomas, 2000); water-to-cement ratio (w/c) (Neville, 1996); and the chemical composition of the supplementary cementing materials used (Shehata & Thomas, 2000).
- The temperature and humidity exposure conditions. With an increase in temperature there is an increase in the rate of reaction (Neville, 1996).
- Mechanical restraints (Haddad & Smadi, 2004). The use of fiber reinforcement can slightly retard the affect of ASR expansion (Haddad & Smadi, 2004). Polypropylene fibers were found to be more effective than brass coated steel fibers (Haddad & Smadi, 2004). The fibers reduce the

amount of cracks formed in the concrete, consequently reducing the progression of the gel product and its ability to spread through the cracks (Haddad & Smadi, 2004). However, it does not prevent the expansion.

2.2.4 Testing for alkali silica reactivity of aggregates

Concrete aggregates often must be evaluated within a very short period of time, thus requiring test methods that are rapid, reliable, simple, and reproducible.

2.2.4.1 Petrographic examination (ASTM C 295)

Essentially this is the first step in testing aggregates for ASR. An example of a sample tested for ASR can be seen in Figure 8. As seen in the figure, a white substance is present. This could be deduced to be ASR gel (Fournier & Berube, 2000). The white substance present is similar to the white substance noticed with the RCA used in this study, as seen in Figure 1. Microscopic examination in a thin section, possibly accompanied by other methods such as X-ray diffraction, infrared spectroscopy, Scanning Electron Microscope (SEM), microprobe analysis, image analysis, and bulk chemical analysis, is used to identify the presence and distribution of potentially reactive material in aggregates (Fournier & Berube, 2000). In some cases, a petrographer with strong ASR experience could recommend accepting an aggregate based only on the knowledge of its petrographic characteristics. However, most of the time additional testing on

mortar and concrete is required to confirm preliminary indications from petrography.

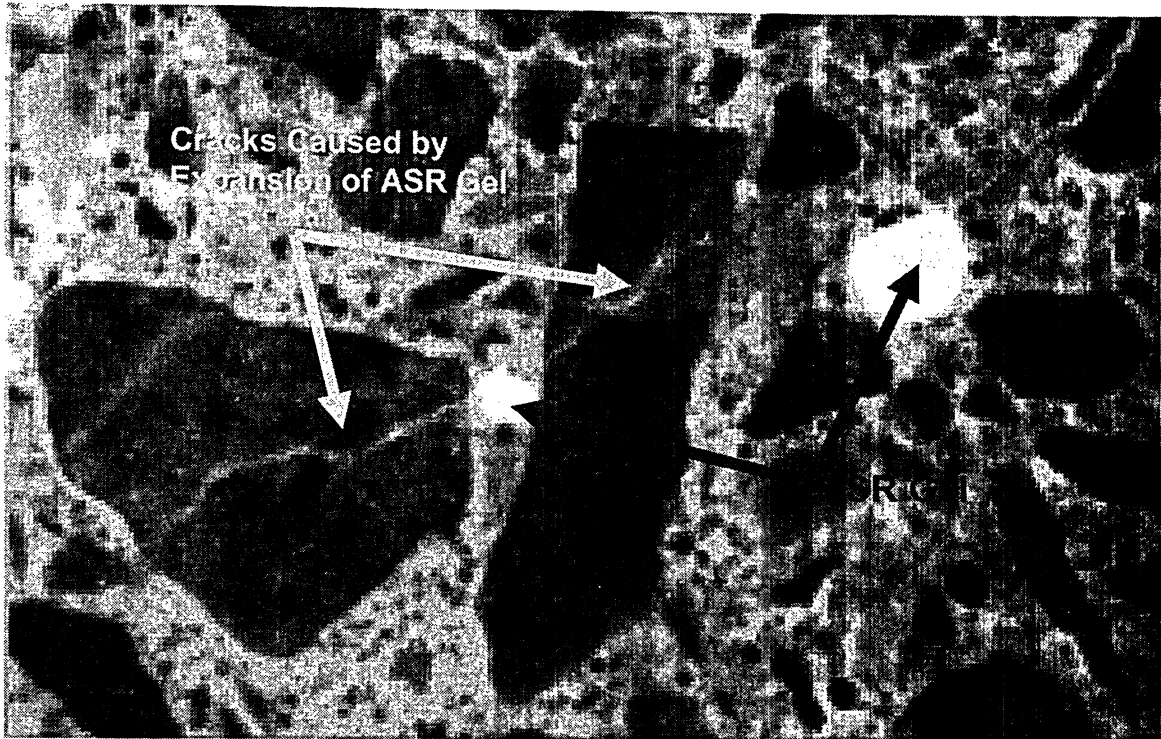


Figure 8: Petrographic analysis of ASR gel filling pores and cracks within the aggregate particles and the cement paste (Fournier & Berube, 2000).

2.2.4.2 Accelerated Mortar Bar Test (CSA A23.2-25A, ASTM C 1260)

This test can be used to evaluate the potential ASR of fine aggregates. Mortar is prepared using specific aggregate gradation, cement, and w/c. These ingredients are then mixed using a specific procedure. The mortar is placed in bars approximately 25 X 25 X 285 mm in size. After 24 hours of moist curing in their molds, the bars are placed in a sealed plastic container filled with water at 23°C, and the container is stored in an oven at 80°C. The next day, an initial length

measurement is taken, and the bars are transferred to a 1 N NaOH solution at 80°C. Measurements occur several times over a 2-week time period. Reactive aggregates are determined based on a 14-day expansion of approximately 0.10%. This test is typically an effective method to determine ASR aggregates within 2 weeks. However, it was found to be misleading for several aggregates with good field performance, but greatly expansive concrete prisms (Fournier & Berube, 2000). Therefore, the Accelerated Mortar Bar Test should not be used for rejecting aggregates. Further testing using the Concrete Prism Test is required.

2.2.4.3 Concrete Prism Test (CSA A23.2-14A, ASTM C 1293)

This test is used to evaluate the potential alkali-reactivity of coarse aggregates. Prisms are prepared using a specific quantity of cement, coarse-to-fine aggregate ratio, and w/c. The w/c should be 0.42 to 0.45, except when supplementary cementing materials are used. In this case, the CSA 2004 limits the w/c to 0.35 to 0.40. The prisms are kept in their molds for 24 h at 23°C, and then stored above water in a sealed plastic container lined with a damp material. After the one-day curing, the prisms are placed in a room controlled at 38°C. Length change measurements are made periodically up to 1 year with concrete containing no SCM and 2 years with concrete containing SCM. Aggregates can show different expansion rates in the Concrete Prism Test. Generally, the more rapid the onset of expansion and the development of micro-cracking due to ASR, the faster the leaching of alkalis out of the test prisms and the levelling off of the expansion

curve (Rogers et al., 2000). Reactive aggregates are determined based on an expansion of approximately 0.04%. There are a number of disadvantages to this test. One disadvantage is that the testing period is too long for most construction projects. Another disadvantage is that depending on the storage conditions, some alkali leaching from the prisms is evident which would slow down or stop the expansion (Rogers et al., 2000).

2.2.5 Preventive measures against ASR

2.2.5.1 Aggregate selection

The most effective measure against ASR is certainly the use of non-reactive aggregates. However, this solution is often not practical or economical because such aggregates may not be locally available or must be transported over long distances to a construction site.

2.2.5.2 Limiting the alkali content in the concrete

ASR can be considerably reduced or even prevented by the use of low-alkali cement or by limiting the total alkali content of concrete under a limiting value. There is a relationship between the alkali content of the concrete and the expansion that occurs as seen in Figure 7.

2.2.5.3 Use of supplementary cementing materials

An adequate amount of an effective SCM can significantly reduce and even control deleterious expansion due to ASR in concrete (Shehata & Thomas, 2002). The use of any SCM reduces the permeability of the concrete, which reduces the ability of the gel to absorb water and expand (Neville, 1996). Alkali dilution and consumption of Ca(OH)_2 through pozzolanic reaction are also thought to play a role in the beneficial effect of SCMs against ASR (Fournier & Berube, 2000). These affects are important but are considered secondary compared to others mentioned further on in this paper (Shehata & Thomas, 2002).

A disadvantage with the use of slag as a preventative measure is the amount required. High proportions are necessary to reduce expansion as shown in Figure 9 (Thomas & Innis, 1998), which may lead to other deleterious problems as seen in Figure 9 (Rogers et al., 2000). Salt scaling for example, was shown to be increased with levels of above 30% slag replacement (Bleszynski et al., 2002). As a consequence, the Ontario Ministry of Transportation has permitted a maximum of 25% slag cement substitution (Rogers et al., 2000). However, it has been determined that for some reactive aggregates levels as high as 50% are required to reduce the ASR expansion to expectable limits (Thomas & Innis, 1998). Thomas & Innis (1998) believed that slag helps to reduce the hydroxyl ion content of the pore solution. This is achieved by incorporating higher levels of alkalis into the C-S-H matrix compared to the C-S-H that is produced with Portland cement.

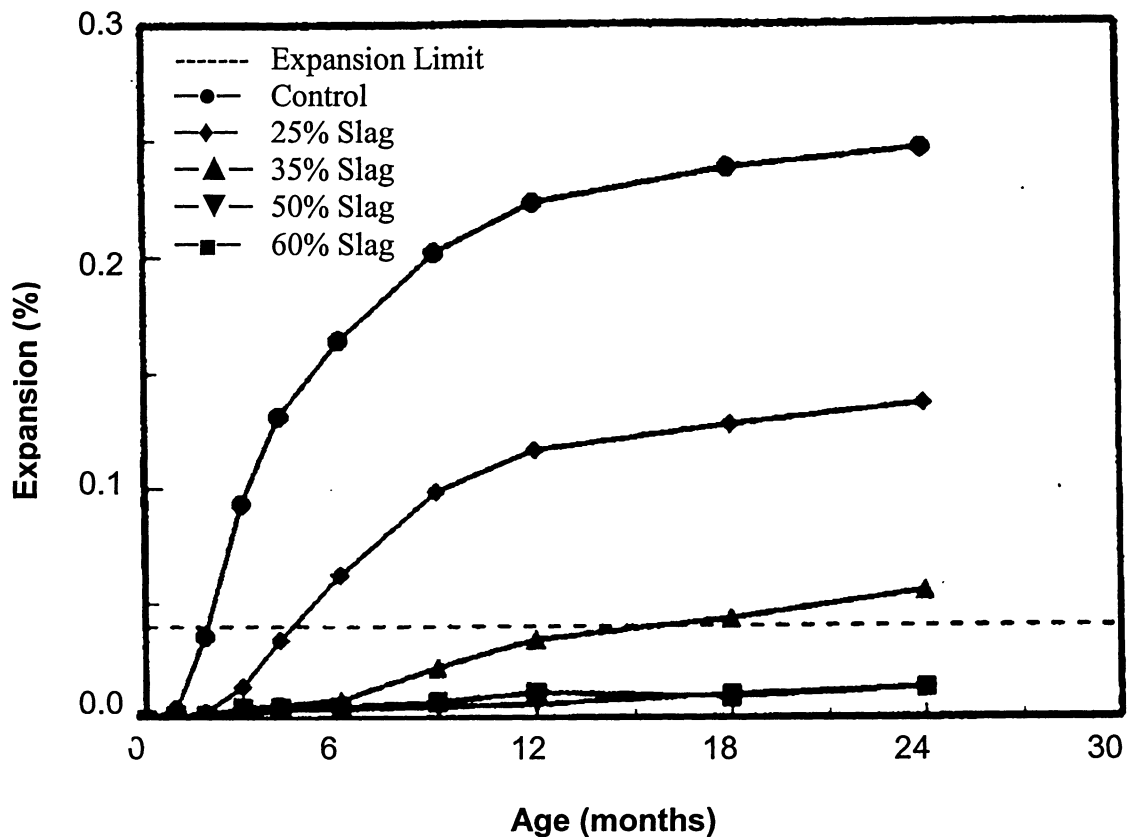


Figure 9: Expansion of concrete with Spratt aggregate alkalis = 1.25% Na_2O_e (by mass of cement) (Thomas & Innis, 1998).

Fly ash has proven to be an effective SCM in reducing ASR expansion. Shehata & Thomas (2000) determined that the amount of alkalis and calcium in the fly ash was important factor in the amount of reduction that occurred, as seen in Figure 10. Of course, an increased amount of alkalis in the fly ash increases the expansion that occurs. In addition, an increased amount of calcium in the fly ash increases the expansion that occurs. It should be stated that a good portion of the alkalis in the fly ash are considered to be insoluble and do not contribute to the alkalis in the

pore solution (Neville, 1996). A reduction in expansion is evident with concrete samples containing all types of fly ash when comparing against concrete with no SCM. It has been theorized that fly ash can be expected to have a beneficial impact on the expansion of concrete beyond the role of merely diluting the cement (Shehata & Thomas, 2000). Furthermore, Shehata & Thomas (2000) state that fly ash contributes further by reducing the availability of the alkalis in the system beyond diluting the cement. Certain physical properties can also improve fly ash contribution to reduce expansion. For example, the fineness of the ash plays a role as does the amount of glass present in the ash (Shehata & Thomas, 2000). It was also noted that the increase in total silica content of the fly ash increased the effectiveness of the fly ash to reduce expansion. The Lower Notch Dam in 1971 is an example of using fly ash as a preventive measure. The fly ash replacement level was 30% and the reactive constituents were an argillite aggregate (Rogers et al., 2000). It was the only known instance in Canada where the use of a known alkali reactive aggregate was permitted with high alkali cement combined with fly ash (Rogers et al., 2000). The dam has performed well for more than 30 years (Rogers et al., 2000).

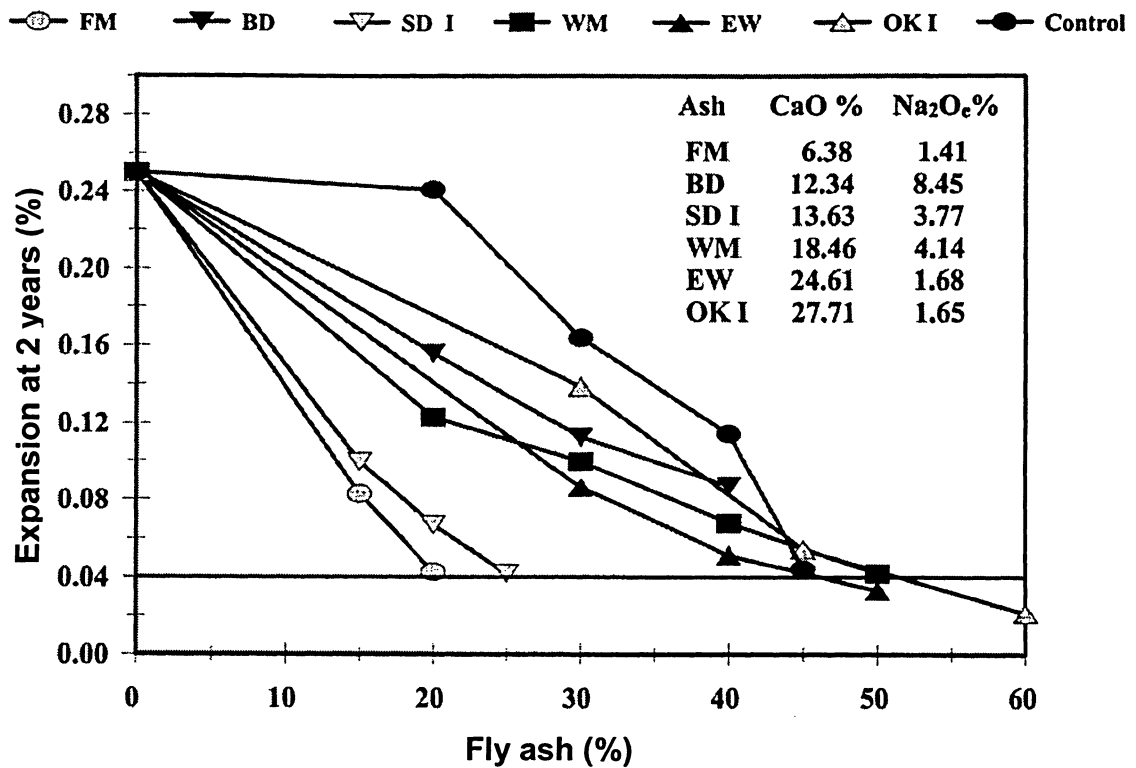


Figure 10: Effect of ash composition and replacement level on expansion due to ASR (Shehata & Thomas, 2000).

Unlike slag or fly ash, high levels of silica fume are not feasible to incorporate in concrete due to a reduced workability and increased drying shrinkage (Neville 1996). Some researches have found that levels of more than 8% are required to control expansion due to ASR (Boddy et al., 2003). At lower replacement levels and combined with extremely reactive aggregate, silica fume has shown to increase the expansion of concrete beyond the expansion of concrete with no SCM (Shehata & Thomas, 2002).

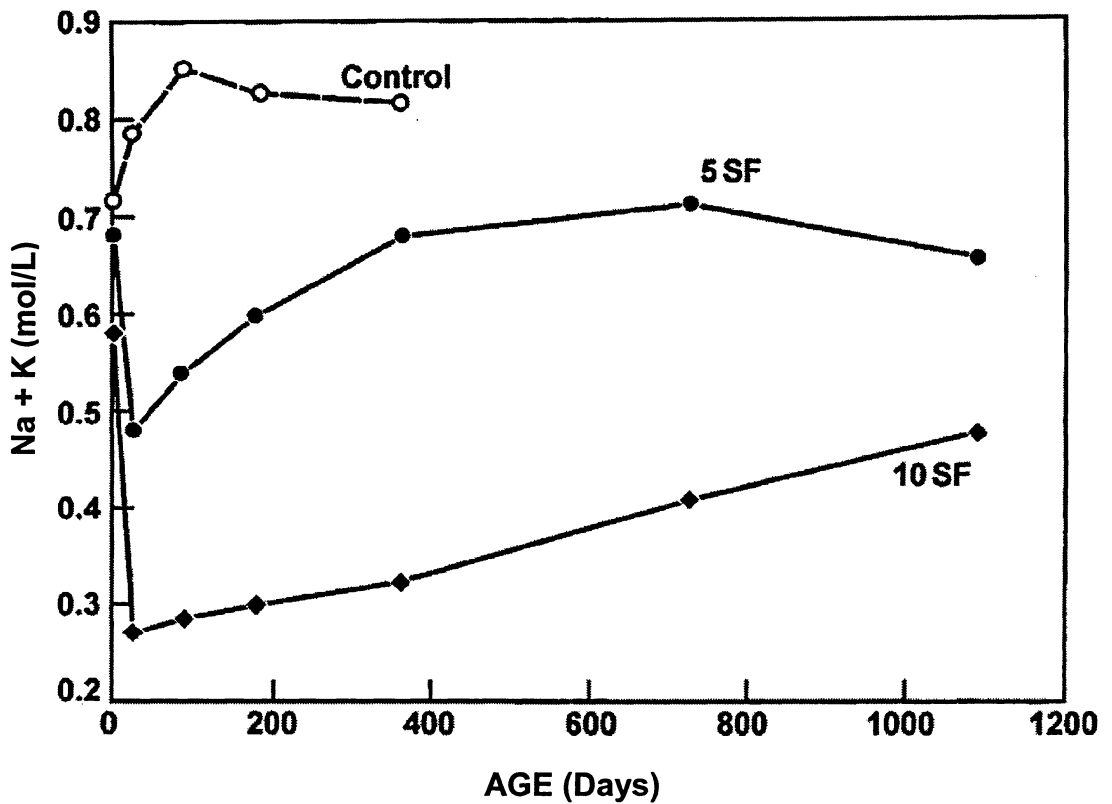


Figure 11: Effect of Silica Fume on the alkalinity of pore solution of pastes, represented by the sum of alkali cations in the pore solution (Shehata & Thomas, 2002).

Ternary blends, containing silica fume and another type of SCM have been recognized for reducing ASR expansion. Concrete samples containing 5% silica fume have shown to initially reduce expansion, then increase to approximately the same expansion as the control as seen in Figure 11. An analysis of the pore solution has shown that initially the addition of 5% silica fume greatly reduces the alkalis in the pore solution. This could be attributed to a reaction that is probably similar to ASR (Shehata & Thomas, 2002). However as time passes, the silica releases these alkalis back into the pore solution by replacing alkalis for calcium (Shehata & Thomas, 2002). An analysis of the pore solution with samples

containing fly ash indicates that the reduction of alkalis in the pore solution occurs gradually (Shehata & Thomas, 2002). When both fly ash and silica fume are added to concrete a “synergy” occurs. This synergy initially has a great reduction of alkalis in the pore solution and then slowly continues to reduce the alkalis as time progresses. In addition these ternary blends showed high capacity to retain alkalies in their hydration products as found by Shehata & Thomas, 2006 (Figure 12). Therefore, the cement replacement with a ternary blend has immense benefits in reducing ASR expansion.

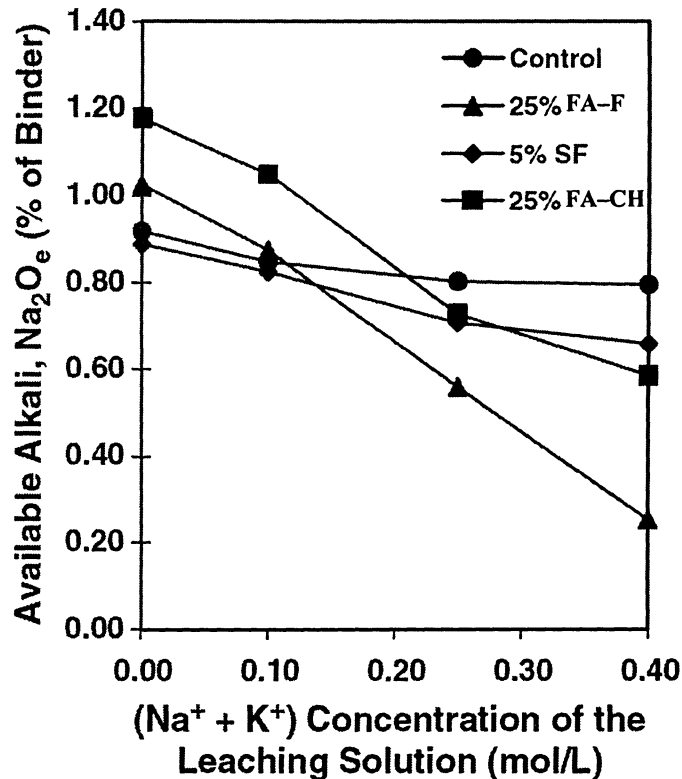


Figure 12: Available alkalis from control and binary blends in leaching solutions of different alkali concentrations expressed as % of mass of binder or cementing materials (Shehata & Thomas, 2006).

Duchesne & Berube (2001) have conducted a more than 10 year long term study on the effects of using SCM on concrete expansion due to ASR. In the study, it was determined that prolonged use of SCMs did not greatly expand beyond approximately two years of testing. Even concrete with 10% silica fume did not expand greatly after two years of testing, as seen in Figure 13 (Duchesne & Berube, 2001).

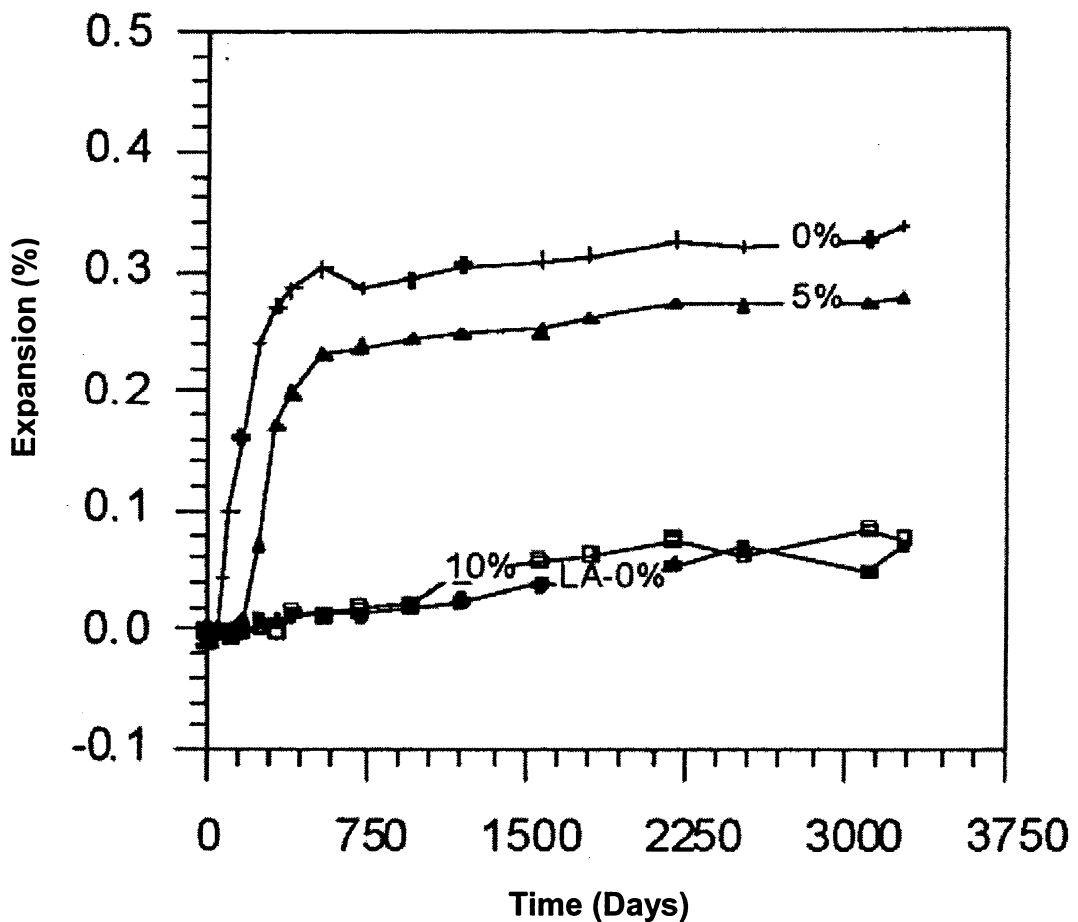


Figure 13: Expansion of concrete prisms containing Spratt and various amounts of Silica Fume with regular alkali cement and low alkali cement (LA) (Duchesne & Berube, 2001).

Metakaolin is a relatively new SCM and is manufactured rather than being a by-product from some other process. It has been used successfully to reduce expansion due to ASR (Ramlochan et al., 2000). A replacement level of 20%, metakaolin has shown to reduce the hydroxyl ion concentration by 4 times the control sample with no SCM as seen in Figure 14 (Ramlochan et al., 2000). The mechanism by which metakaolin reduces expansion due to ASR, seems to be the entrapment of alkalis by the supplementary hydrates formed in the concrete (Ramlochan et al., 2000). With 10% replacement a slight increase of alkalis at 7 days can be observed.

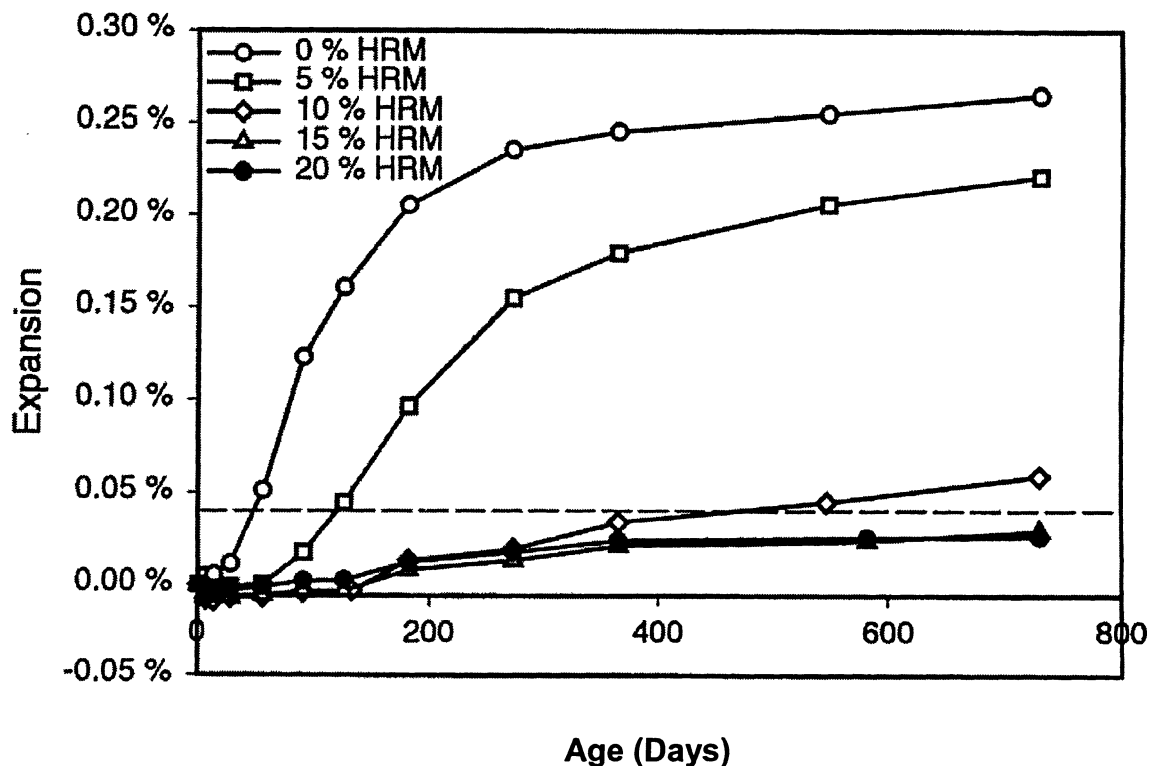


Figure 14: Expansion of concrete prisms containing highly reactive metakaolin with Spratt aggregate (Ramlochan et al., 2000).

2.2.5.4 Use of chemical additives

Lithium-based compounds were probably among the most effective chemical compounds against ASR that were tested by McCoy and Caldwell in 1951. The level of lithium required to control deleterious expansion due to ASR varies with the alkali content of the concrete, the nature of the reactive aggregate, and the type of lithium compound used (Diamond, 1998).

The long-term effects of using Lithium (LiOH) as a preventive measure to reduce expansion due to ASR have proven to be successful (Mo et al., 2003). Some researchers believe that the lithium ion Li^+ has both a smaller ionic radius and higher surface charge density than Na^+ or K^+ (Mo et al., 2003). This allows the Li^+ to enter the ASR product more readily than Na^+ or K^+ . Since the ASR product consists of lithium, this produces a crystalline and non-expansive gel (Mo et al., 2003). Others believe that lithium decreases the rate of silica dissolution (Shomglin et al., 2003). Although the mechanisms are still unclear, lithium has proven to be an effective measure.

The use of a LiOH as a lithium additive has been criticized by others (Diamond, 1998). Not as a preventative measure, but on the amount required to reduce expansion. Diamond (1998) states, that the improper proportions of LiOH can actually increase expansion. As seen in Figure 15 the addition of 0.4, and 0.8 molar equivalent increases the expansion due to ASR; whereas, 1.2 molar equivalent

reduces the expansion. This can be explained by the amount of alkalis that are released into the pore solution upon the absorption of Li^+ into the hydrating cement (Diamond, 1998). Therefore, Diamond (1998) recommended the use of LiNO_3 as a lithium preventative measure, rather than LiOH . Although the Li^+ is still absorbed into the hydration products, the release of NO_3 does not seem to have a negative effect in the concrete and also does not increase the amount of alkalis in the pore solution.

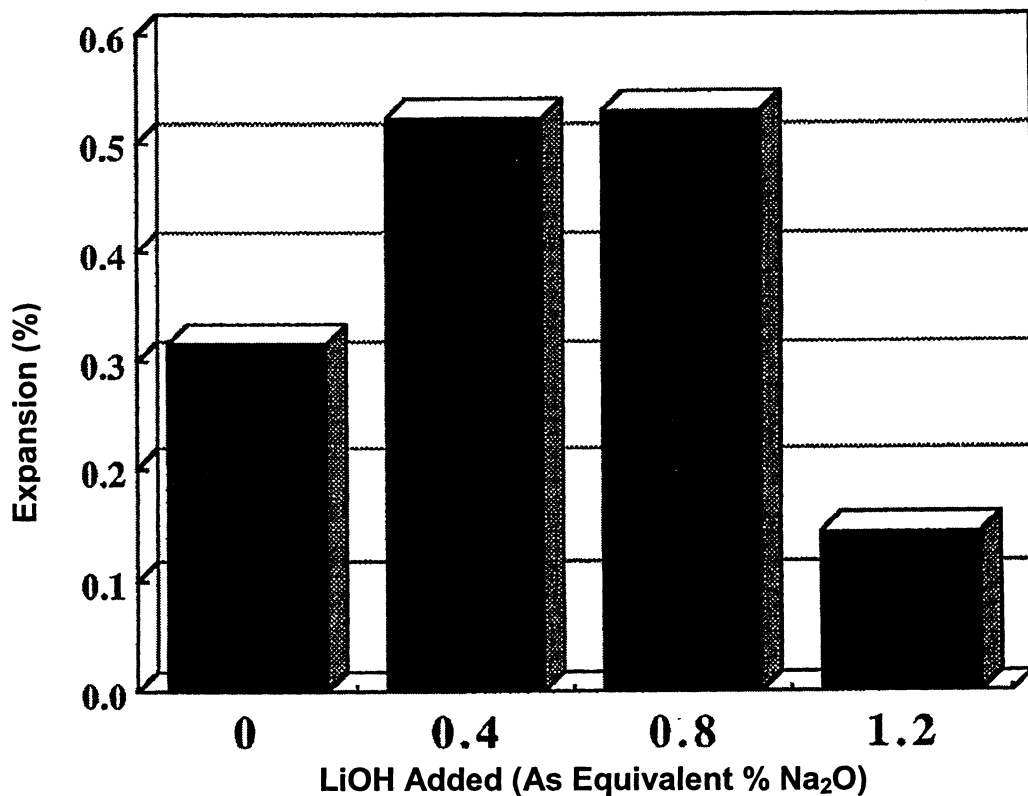


Figure 15: Expansion at 9 months of mortar bars containing Beltane opal as a function of dosage of LiOH concentration dissolved in the mix water (Diamond, 1998).

2.3 Freezing and Thawing

2.3.1 Mechanisms of Freezing and Thawing

Freezing and thawing is a process that is unavoidable in Canada; therefore, proper measures should be taken in order to avoid deterioration. The main mechanism that results in freezing and thawing deterioration is believed to be hydraulic pressure. This mechanism occurs when water in the fully saturated pores freezes causing expansion of around 9%. This expansion produces a pressure applied to the walls of the pore causing the concrete to crack (Neville, 1996).

2.3.2 Additives Influencing Freeze Thaw Resistance

2.3.2.1 Air Entrainment

It has been determined that a proper amount of air entrainment can positively improve the resistance to freezing and thawing in concrete (Chatterji, 2003). The air entrainment produces spherical air voids in concrete. These air voids act as a relief mechanism for the water that is expelled from the capillary pores. Since the air voids are considered to be closed and full of air, full saturation of these voids does not occur, although some water may exist (Chatterji, 2003). The pressure that is accumulated by the concrete's inability to expel the water is enough for the water to surpass the membrane of the air void. Therefore increasing the air entrainment can create enough air voids to prevent the deteriorating effect of freezing and thawing. Conversely, a great amount of air entrainment produces air voids with an increased volume and increased spacing between air voids

(Chatterji, 2003). An increase in size would imply that a greater amount of water can be accommodated. However, the increase in spacing results in a further distance for the water to travel before it can release the pressure. Therefore, the pressure is built up and cracks can occur in the concrete. An optimum amount of air entrainment has been determined to be between 5 to 9% (Neville, 1996). Other factors that can promote deterioration are:

- **Reduced strength:** Reduced strength has a negative affect. A reduction in strength would imply that the concrete has a reduced ability to resist the pressure that is applied by the water. Increasing the strength of concrete does not necessarily improve the resistance of freezing and thawing (Jacobsen et al., 1995). Some non-air entrained concrete samples with high strength have shown to deteriorate (Jacobsen et al., 1995). Different types of aggregates that are contained in high strength concrete produce different results. In a study by Jacobsen et al. (1995), high strength concrete with Granite and Leca 800 did not finish the 300 cycles required by ASTM C666. They deteriorated by less than 100 cycles. High strength concrete with Limestone, Quartzdiorite, and Macrolite can have varied results. The majority of concrete samples containing these stones can have durability factors of 40 to 100 (Jacobsen et al., 1995). The ASTM C666 standard was found to correlate with the amount of cracks that were observed in concrete samples though Fluorescent Liquid Replacement as seen in Figure 16 (Jacobsen et al., 1995).

- **The increase in w/c:** The increase in w/c can cause several effects. One is the increase in the capacity of the capillary pores (Neville, 1996). The more water the capillary pores can hold results in more water being expelled as the pores freeze. If the volume of the water that is expelled is greater than the volume of the surrounding air voids, then the air voids cannot accommodate the water and pressure will build up cracking the concrete (Neville, 1996). Second is the reduction of strength that occurs with increased w/c (Neville, 1996). Third effect is the increase in air void spacing caused by an increase in w/c (Neville, 1996).

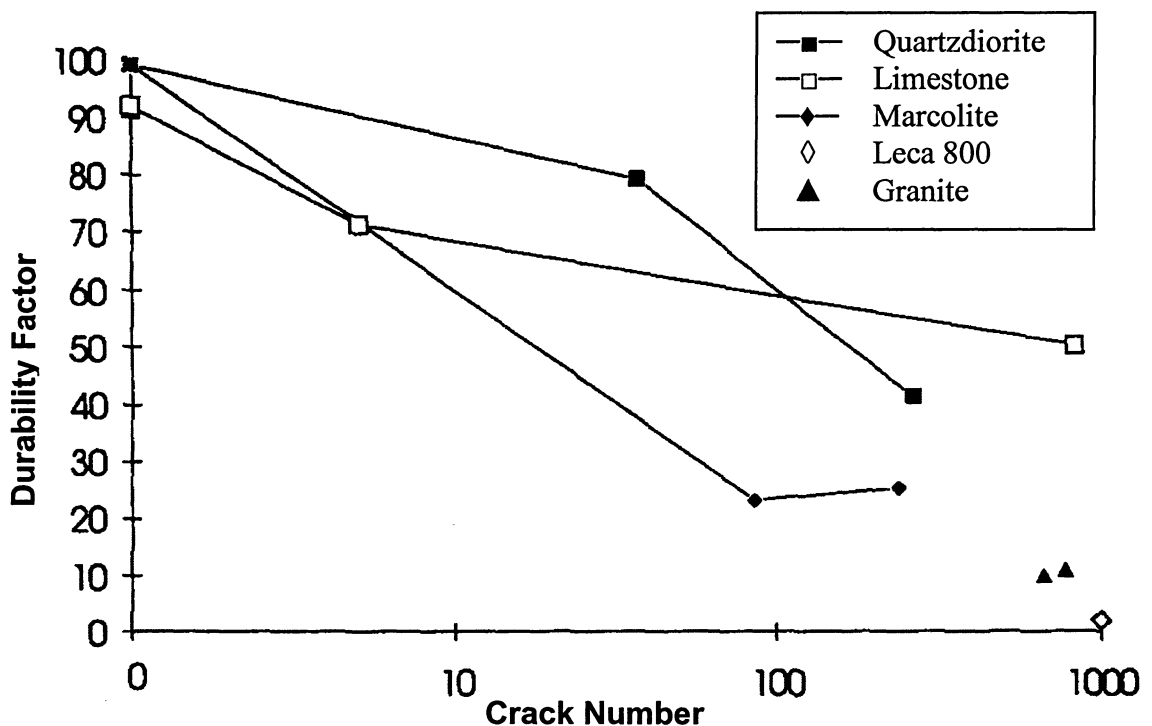


Figure 16: Durability Factor verses number of cracks in high strength concrete (Jacobsen et al., 1995).

- **The degree of saturation:** As mentioned earlier, saturation above 91% would not accommodate the increase of 9% that occurs when water freezes; therefore, a proper drying period before exposure to freezing would help prevent freeze thaw damage (Neville, 1996).
- **The time it takes for freezing and thawing to occur and the time between freeze thaw cycles:** If the concrete rapidly freezes then the water doesn't have enough time to be expelled into the pores. In addition, short thaw periods result in a greater chance for increased saturation of the concrete (Neville, 1996).
- **Proper curing before exposure to freezing and thawing:** With proper curing the concrete has a reduced permeability. This results in less water being absorbed during wet weather. Furthermore, proper curing can reduce the amount of freezable water in the paste (Neville, 1996).

2.3.2.2 Supplementary Cementing Materials

Other factors that have shown to reduce concrete's resistance to freezing and thawing are the addition of supplementary cementing material as seen in Figure 17 (Toutanji et al., 2004). Using the freezing and thawing test described by ASTM C666, Toutanji et al. (2004) demonstrated that SCMs, in general, reducing the durability factor (Figure 17).

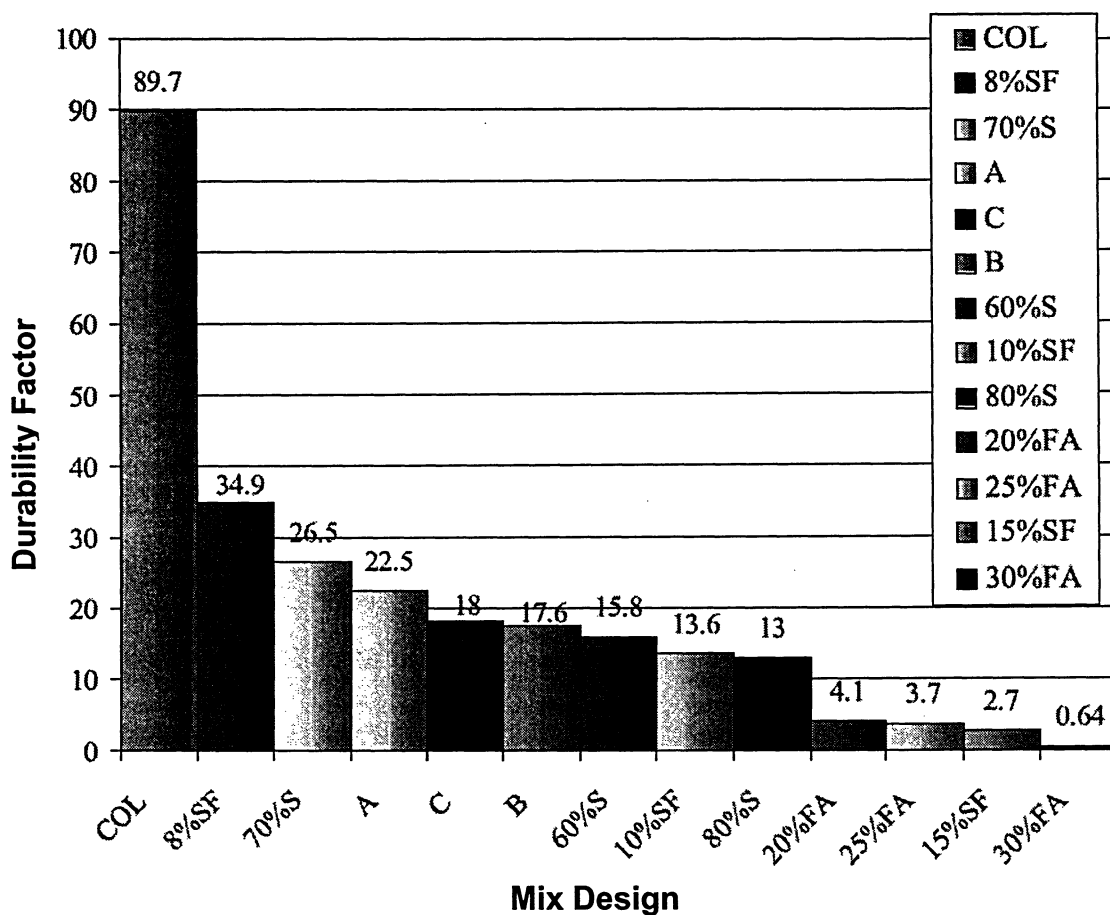


Figure 17: Durability factor for control and mixes with supplementary cementations materials (Toutanji et al., 2004).

2.3.2.3 Micro Fibers

Research involving the use of micro fibers in concrete has shown that a reduction in the deterioration can be observed; however, micro fibers cannot discontinue deterioration (Pigeon et al., 1996). Pigeon et al. (1996) concluded that the addition of 2 - 5% steel fibers or 2% carbon fibers resulted in reduced deterioration, whereas the concrete sample with no fibers completely deteriorated. However, the

addition of fibers does not prevent the formation of cracks; it just inhibits the rate of crack formation (Pigeon et al., 1996).

2.3.2.4 Recycled Concrete Aggregates

In terms of using recycled concrete aggregates, many conclusions have been derived. Some researches report that the use of recycled aggregates should be completely avoided in order to produce a freezing and thawing resistant concrete (Zaharieva et al., 2004). Others state that the use of RCA produces concrete that is within acceptable limits, depending on the properties of recycled concrete (Gokce et al., 2004). Gokce et al. (2004) found that RCA produced from concrete that originally had no air entrainment resulted in a concrete with reduced freezing and thawing resistance as seen in Figure 18. This type of concrete can reach a durability factor of below 60 in less than 90 cycles (Gokce et al., 2004). Greatly reduced freeze thaw resistance can occur in concrete containing as little as 12.5% non-air entrained recycled aggregate (Gokce et al., 2004). However, in some cases concrete containing air entrained recycled aggregate can outperform concrete containing natural aggregates (Gokce et al., 2004). Thus it can be concluded that the use of non-air entrained recycled aggregate should not be used. It should be noted that Gokce et al. (2004) allowed the samples to cure for 28 days before experimentation.

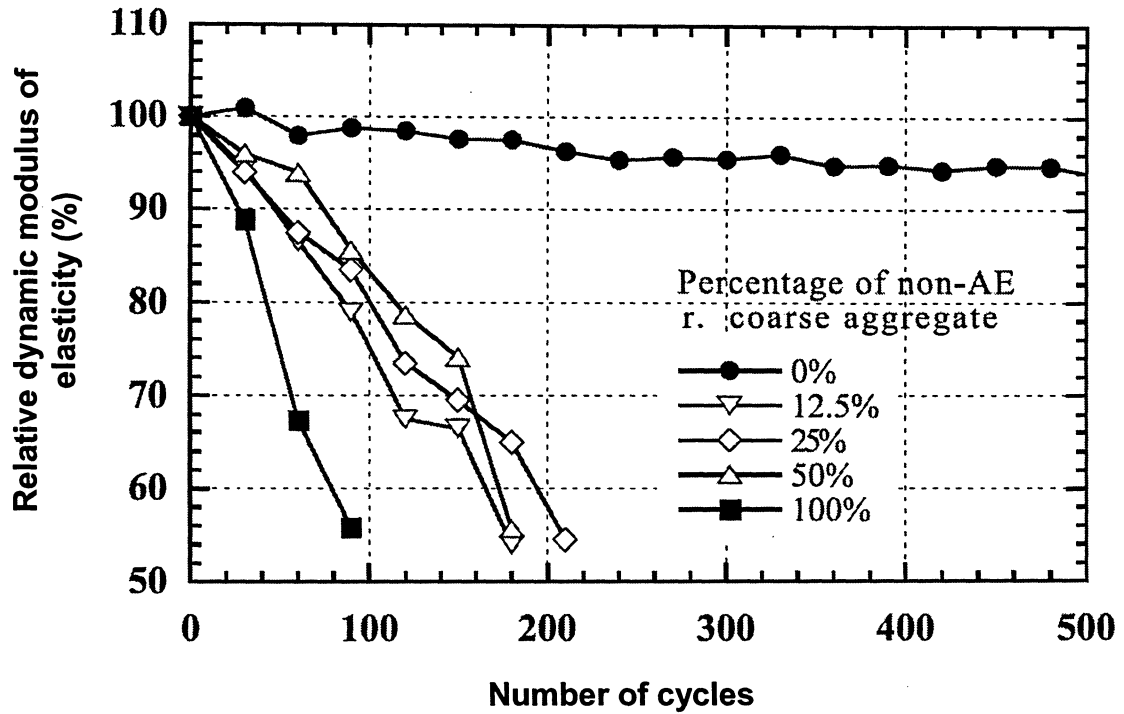


Figure 18: Relative dynamic modulus of elasticity versus number of frost cycles for the concretes incorporating pure type or blended recycled coarse aggregates (Gokce et al., 2004).

Zaharieva et al. (2004) states that the frost resistance of saturated recycled aggregate concrete is not satisfying, and their use in structures exposed to severe climate is not recommended. The contributing factor seems to be the high total w/c, which generates a higher porosity as well as the frost resistance of recycled aggregates themselves (Zaharieva et al., 2004). Decreased resistance could also be a factor of the presence of unsound particles, which would be deteriorated by the repeated action of the freezing and thawing cycles. Furthermore, recycled aggregates could contribute to the frost damage by expelling water into the

surrounding cement paste during the freezing period (Zaharieva et al., 2004). It should be noted that Zaharieva et al. (2004) used non-air entrained RCA.

2.4 Salt Scaling

2.4.1 General

The use of supplementary cementing materials (SCM's) to improve deterioration resistance of concrete has become well known. From reducing expansion due to ASR and sulphate attack to simply increasing workability and strength, SCM's have shown to be advantageous and are used in most mixes; however, there are situations where the use of SCM's actually promotes further deterioration. Some SCM's have shown to reduce the salt scaling resistance of concrete, mainly fly ash. It should be noted that lab results have been found to differ with in-situ results; this was attributed to the effect of sun and wind exposure on the bleeding of concrete (Bleszynski et al., 2002).

2.4.2 Mechanisms of Salt Scaling

In order to understand the effects of SCM's on salt scaling, an understanding of the mechanisms of salt scaling is required. The exact mechanism is not yet clear; however, it is probably the combination of a couple of processes (Mindess et al., 2003). It has been stated that the processes effecting scaling resistance are most likely physical rather than chemical (Neville, 1996). The physical processes are:

- **Osmotic Pressure:** water-containing salt fills the pores of a concrete structure.

As the water begins to freeze in the large pores the salt ions in the frozen water are squeezed out into the remaining water in the pore. This produces a difference in concentration in the remaining water. This difference causes the water in the smaller pores to move towards the larger pores due to osmosis. The pressure caused by the increase in water triggers the concrete to crack. Again this process is continued until the concrete surface is deteriorated (Neville, 1996).

- **Hydraulic Pressure:** water in the fully saturated pores freezes causing expansion of around 9%, thus there is a pressure applied to the walls of the pore causing cracking. This process can be considered a minimal effect compared to the other effects (Neville, 1996).
- **Differential Thermal Strain/Rapid freezing:** the consumption of heat required to melt ice when a de-icer is applied causes a rapid drop in temperature of the concrete just below the surface, which may cause damage (Mindess et al., 2003).

Whatever the process that causes salt scaling, it is evident that if the concrete has a low permeability, low w/c, and adequate air entrainment, it will also have a higher resistance to salt scaling (Mindess et al., 2003). It is also evident that

anything that affects the surface of the concrete will also play a role in the resistance of salt scaling; such as, over vibrating, trowelled too early and too long, subjected to plastic shrinkage, or where excessive bleeding has occurred (Mindess et al., 2003).

2.4.3 Factors Influencing Resistance to Salt Scaling

2.4.3.1 Trowelling

Several lab experiments have been performed to determine the effects of salt scaling. It has been determined that an extremely porous layer is created in the first millimetres of the trowelled laboratory concrete surface (Pigeon et al., 1996). This highly porous layer was found to reduce the resistance of salt scaling (Pigeon et al., 1996). Laboratory produced trowelled concrete samples were compared against sawed concrete samples. The comparison revealed that the trowelled samples initially deteriorated rapidly then, after approximately 25 cycles, the deterioration rate began to slow down (Pigeon et al., 1996); whereas, the sawed samples had great salt scaling resistance and had an approximate constant rate of deterioration (Pigeon et al., 1996). It was also shown that the addition of fly ash has a negative effect on scaling resistance on both trowelled and sawed samples and increasing the fly ash also decreases the scaling resistance as seen in Figure 19 (Pigeon et al., 1996).

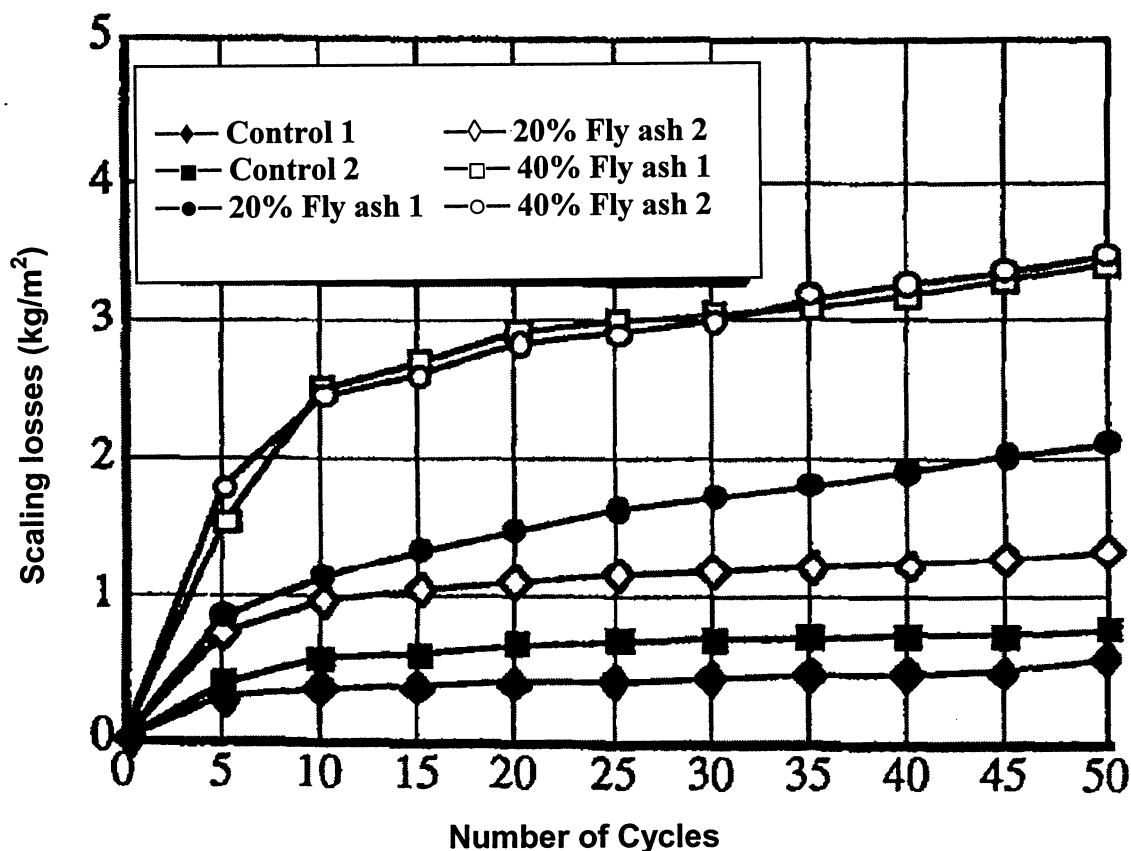


Figure 19: Results obtained using varying amounts of fly ash (Pigeon et al., 1996).

2.4.3.2 Fly ash

It should be noted that the delay of strength development of the fly ash mixes was not found to be a contributing factor to the reduced resistance (Talbot et al., 1996). Various types of cement containing fly ash were cured to 14 and 28 days and found to have a minimal difference in scaling resistance as seen in Figure 20 (Talbot et al., 1996). The same conclusion was discovered with slag as seen in Figure 21 (Talbot et al., 1996).

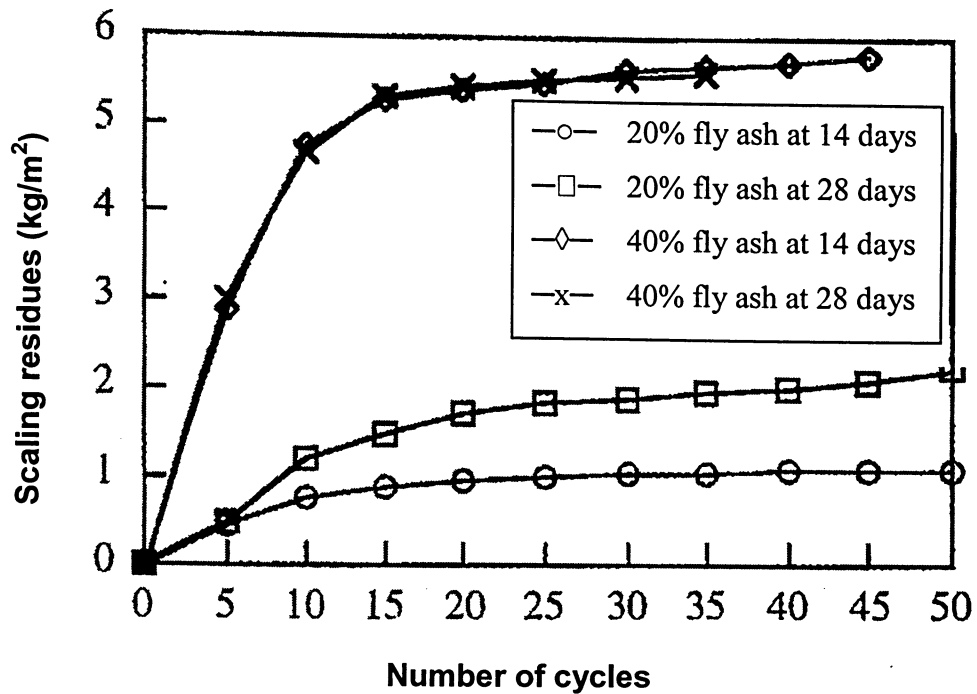


Figure 20: Salt scaling resistance of fly ash mixtures at different curing times (Talbot et al., 1996).

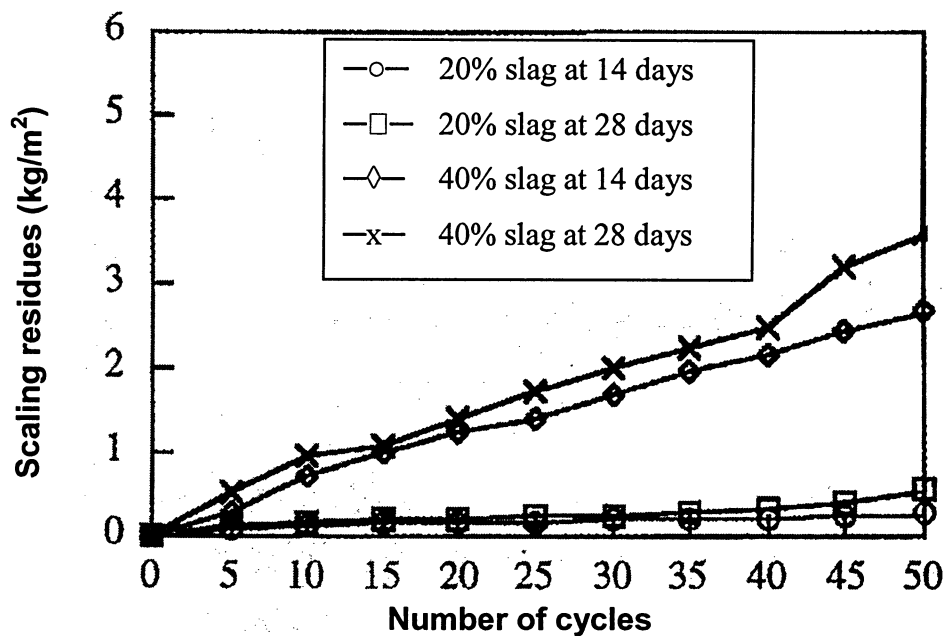


Figure 21: Salt scaling resistance of slag mixtures at different curing times (Talbot et al., 1996)

Different chemical compositions of fly ashes were also compared and found to not have an effect on the scaling resistance of concrete (Bilodeau et al., 1994). Eight different fly ashes were tested to determine if the calcium content or alkali content had an effect on the resistance of scaling. Two samples contained CaO percentages of 19.34% and 14.93%, one sample had a CaO percentage of 9.51% and the last five samples had CaO contents of less than 4.5% (Bilodeau et al., 1994). The samples also had varying alkali content from high to low. After 50 cycles the concretes were all found to have a visual rating of 5 and the amount of residue essentially were similar (Bilodeau et al., 1994).

Contrary to the above-mentioned papers, fly ash was found to benefit the resistance of concrete in one paper (Naik et al., 1998). Percentages of 35 to 40% were tested and found to have visual ratings of 1 to 3, which would indicate that the scaling was slight to moderate and the concrete would be suitable for highway use; however, residue accumulation was not recorded and visual inspections are not as reliable (Naik et al., 1998).

2.4.3.3 Slag

It was also found that the use of slag of 40% and above also reduces the scaling resistance (Talbot et al., 1996). Two samples were tested, one containing 20% and the other containing 40% slag replacement. The sample containing 20% slag had minimal deterioration while the 40% did not pass the MTO's standards of 0.8

Kg/m² (Talbot et al., 1996), while in another paper slag contents of 35 and 50% were tested (Bleszynski et al., 2002). The 35% passed the MTO's standard while the 50% did not pass (Bleszynski et al., 2002) as seen in Figure 22.

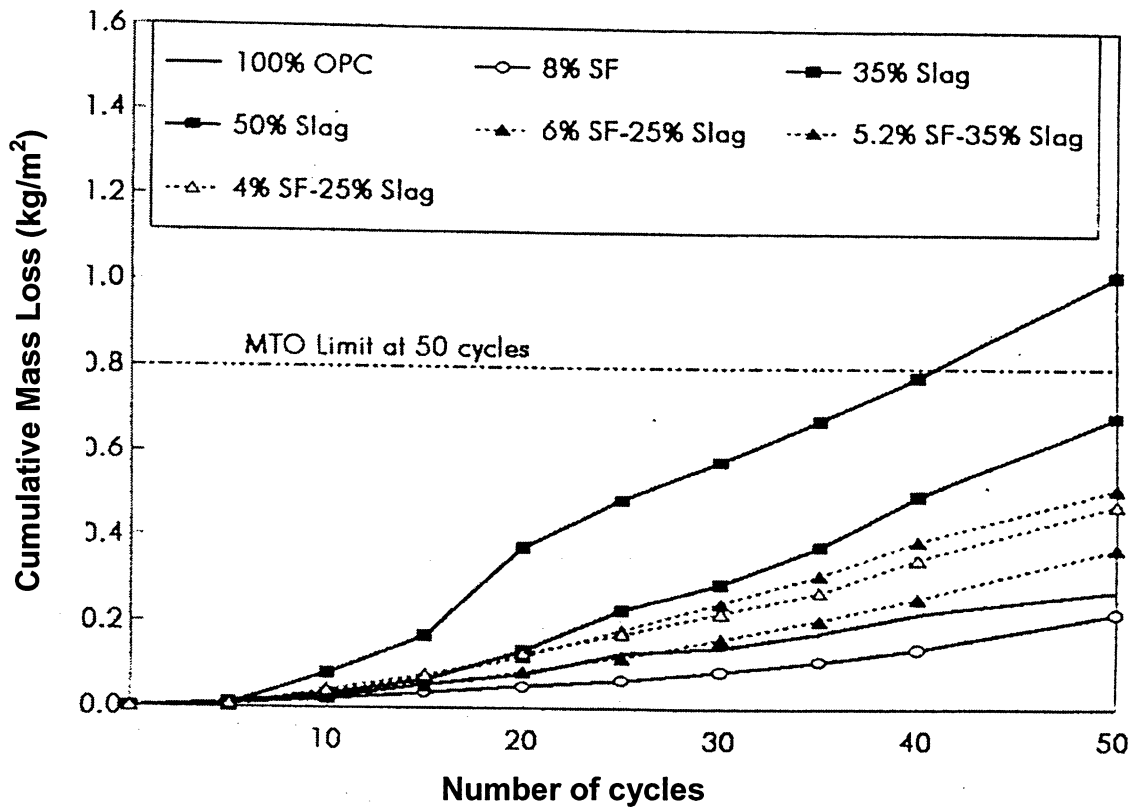


Figure 22: Salt scaling results slag and ternary slag mixes (Bleszynski et al., 2002).

2.4.3.4 Silica Fume & Ternary Blends of SF/Slag

Silica fume and ternary blends containing silica fume and slag were also tested. The sample containing 8% silica fume was found to improve the concretes resistance to scaling (Bleszynski et al., 2002). The ternary blends did not improve the scaling resistance compared to the sample with no SCM's (Control), but did

improve the samples with slag alone as seen in Figure 22 (Bleszynski et al., 2002). It was concluded that increasing the silica fume content increases the concretes scaling resistance (Bleszynski et al., 2002). It should be noted that the samples by Bleszynski et al. were tested on the formed surface instead of the trowelled surface. This modification to the standard was to eliminate any discrepancies caused by bleeding and finishing (Bleszynski et al., 2002). The samples that were trowelled were found to have differences of 11 to 50% when compared to duplicate samples (Bleszynski et al., 2002), thus the modification was necessary and this confirms that bleeding and finishing contribute greatly to the salt scaling resistance.

2.4.3.5 Salt Concentration

Although the concentration of salt that is pooled on top of the samples is kept identical, it should be noted that there is an optimal amount of salt that can be added, which will result in the most deterioration from occurring on the samples. The ASTM C672 requires a salt-water concentration of 4%; however, the most effective concentration percentage has been determined to be between 1.5 to 3% as seen in Figure 23 (Marchand et al., 1999). Marchand et al. (1999) also found that increasing the water/cement (W/C) ratio resulted in further deterioration. The W/C ratios tested in this study were 0.25, 0.35, and 0.45, with 0.45 having the greatest deterioration. Although the W/C ratios were varied, the optimal salt concentration remained consistent between the samples. The concentrations of salt tested were from 0 to 12%.

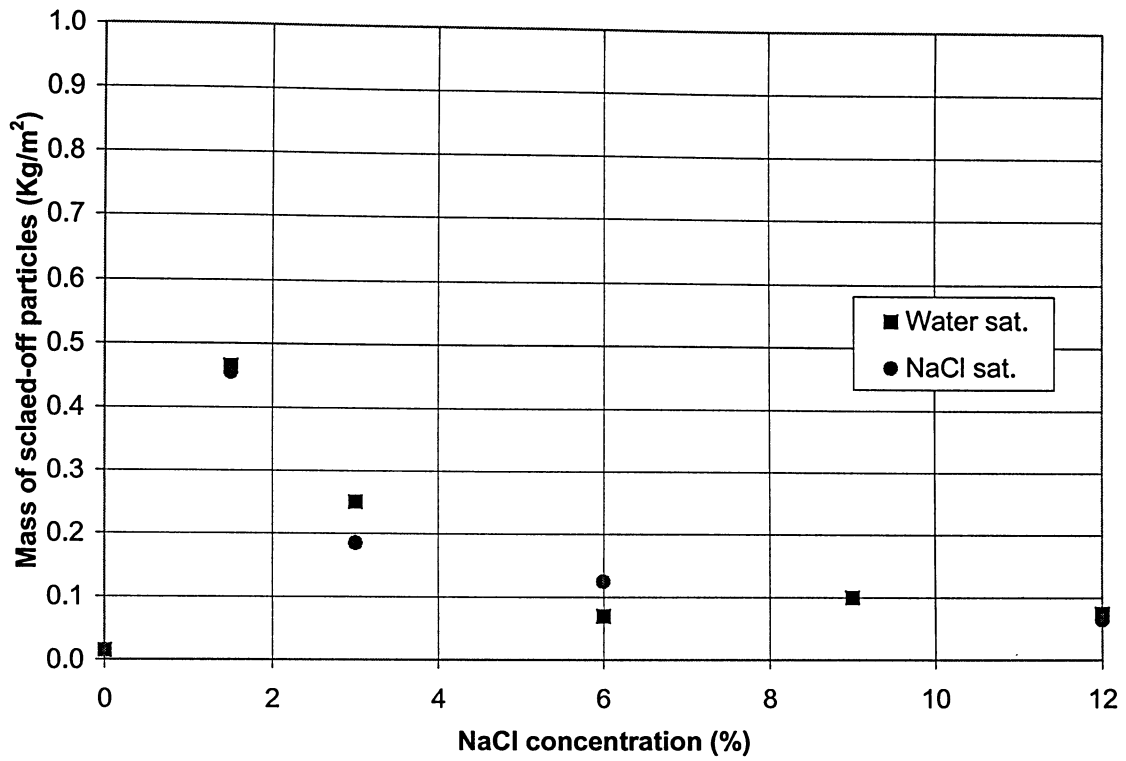


Figure 23: De-icer salt scaling test results for 0.35 w/c concrete mixes (after Marchand et al., 1999)

Chapter 3

Materials & Experimental Program

3.1 Materials

3.1.1 Aggregates

The properties of all the aggregates used in this study are shown in Table 1. There were four different types of coarse aggregates tested; two reactive and two non-reactive. The reactive aggregates were Spratt and ASR recycled concrete aggregate. The two non-reactive aggregates were Limestone and Non-ASR recycled concrete aggregate. The 3/4" coarse limestone was received from Acton, Ontario. The Spratt aggregates were obtained from Ottawa, Ontario. Through the help of the MTO, the ASR recycled concrete aggregates used in this study were obtained from the Ministry of Transportation trial site in Kingston, Ontario. The concrete was part of a sidewalk that was built with Spratt aggregate in 1991. The sidewalk was found to be deteriorating due to alkali silica reaction. The ASR affected concrete was removed and crushed into aggregate-size particles. As seen in Figure 1 the aggregates still contain some white deposits that are thought to be ASR gel. The non-reactive RCA was from Dufferin plant in Toronto, Ontario. The aggregates were part of a pile that mostly contained leftover concrete and some quantity of concrete collected from the demolition of old concrete. Most of the non-concrete debris, such as steel, was removed with the aid of magnets and other techniques.

There were also four different types of fine aggregate tested; three were reactive and one was non-reactive. The three that were reactive were natural sand, Jobe, and ASR recycled concrete aggregate that was primary crushed and secondary recrushed. The one non-reactive was Ottawa sand. The natural fine aggregate was from Orangeville Ontario. This sand was found to correspond with the gradation limits in ASTM C136. Jobe was from New Mexico and has been proven to be very reactive with respect to alkali silica reaction. ASR fine recycled concrete aggregate that was primary crushed was obtained from the original crushing of the deteriorated sidewalk and does contain the white deposits similar to the coarse aggregate. The secondary recrushed fine RCA was acquired from recrushing the ASR coarse aggregates, mentioned above, into the appropriate gradations. For all intended purposes, when the type of sand is not mentioned it should be assumed that the natural sand was used.

The RCA was separated in 3 sizes for testing of coarse material and 5 sizes for testing of fine material. The gradations were according to ASTM Standard C1293 and C1260, respectively. The coarse ASR RCA was not washed to avoid removing the pre-existing alkalis. Furthermore, heating the aggregates was avoided because it was assumed that the heat would destroy some of the ASR gel.

3.1.2 Cementing Materials

The cement and supplementary cementing materials used in this study were received by various suppliers. The chemical analysis for cement, slag, and silica fume is shown in Table 2. The chemical analysis for the fly ash can be seen in Table 3. Carbon content was measured with Leco SC444DR Carbon/Sulfur Analyzer. The chemical analysis was performed on a dry, ignited basis for the fly ash and on a dry basis for the slag, silica fume and cement using a Bruker S4 X-ray fluorescence spectrometer according to ASTM D4326.

Table 1: Aggregate Properties using ASTM Standards

Aggregate Type	Bulk Relative Density (Dry) (Kg/m³)	SSD Bulk Relative Density (Kg/m³)	Apparent Relative Density (Kg/m³)	Dry Rodded Density (Kg/m³)	Absorption
<i>Coarse Aggregate</i>					
Limestone	2680	2694	2718	-	0.51
Spratt	2665	2680	2706	-	0.58
ASR Recycled Concrete	2333	2441	2615	1385	5.07
Non-ASR Recycled Concrete	2249	2400	2647	-	6.69
<i>Fine Aggregate</i>					
Ottawa Sand	2417	2418	2419	-	0.00
Natural	2648	2660	2691	-	0.60
Jobe	2515	2534	2564	-	0.77
Primary ASR Recycled Concrete	2023	2203	2469	-	8.94
Secondary ASR Recycled Concrete	2107	2217	2367	-	5.21

Table 2: Chemical analysis of cement, slag, and silica fume

Sample Material	1.06 Alkali Cement	0.96 Alkali Cement	0.89 Alkali Cement	0.55 Alkali Cement	Slag	Silica Fume
Carbon Content	0.48	-	-	-	0.21	-
Silicon Dioxide, SiO ₂	19.51	19.58	19.92	20.12	34.40	96.19
Aluminum Oxide, Al ₂ O ₃	4.94	5.35	5.57	4.43	7.40	0.35
Iron Oxide, Fe ₂ O ₃	2.65	2.29	2.10	2.80	0.94	0.10
Sulfur Trioxide, SO ₃	4.24	4.10	3.86	3.60	0.83	-
Calcium Oxide, CaO	62.65	62.84	63.02	62.41	43.20	0.27
Sodium Oxide, Na ₂ O	0.31	0.21	0.20	0.27	0.57	0.11
Magnesium Oxide, MgO	2.56	2.43	2.55	3.01	9.30	0.23
Potassium Oxide, K ₂ O	1.13	1.13	1.05	0.44	0.58	0.51
Phosphorus Pentoxide, P ₂ O ₅	0.16	0.11	0.12	0.13	175ppm	-
Titanium Dioxide, TiO ₂	0.28	0.31	0.31	0.23	0.44	-
Chloride, Cl	786ppm	-	0.01	-	-	-
Manganese Oxide, Mn ₂ O ₃	0.07	0.07	0.05	0.13	-	-
Strontium Oxide, SrO	0.11	0.08	0.09	-	-	-
Na ₂ O _e	1.05	0.95	0.89	0.56	0.95	0.44

Note: All cements in study were Type 10 Portland cements
Silica Fume was not palletized
A dash (-) represents that no data was available

Table 3: Chemical analysis of fly ash

Sample Material	Fly ash F, LA	Fly ash F, HA	Fly ash Cl, LA	Fly ash CH, LA #1	Fly ash CH, LA #2	Fly ash CH, LA #3
Carbon Content	1.58	-	1.29	-	-	-
Silicon Dioxide, SiO ₂	55.74	61.29	40.15	33.26	35.92	31.39
Aluminum Oxide, Al ₂ O ₃	27.35	16.81	21.37	18.24	21.18	18.54
Iron Oxide, Fe ₂ O ₃	5.59	4.62	9.92	6.45	5.85	5.22
Sulfur Trioxide, SO ₃	0.26	1.12	2.46	2.59	1.29	2.62
Calcium Oxide, CaO	4.43	6.42	16.97	28.73	24.48	29.83
Sodium Oxide, Na ₂ O	0.44	3.68	1.36	1.94	1.70	2.10
Magnesium Oxide, MgO	1.56	2.15	4.23	5.32	4.46	5.22
Potassium Oxide, K ₂ O	2.29	0.98	1.04	0.33	0.48	0.31
Phosphorus Pentoxide, P ₂ O ₅	0.31	0.54	1.10	0.88	1.36	1.20
Titanium Dioxide, TiO ₂	0.67	0.95	1.39	1.45	1.68	1.60
Chloride, Cl	-	-	-	-	-	-
Manganese Oxide, Mn ₂ O ₃	-	-	-	-	0.03	0.11
Strontium Oxide, SrO	-	-	-	-	0.44	0.55
Na ₂ O _e	1.95	4.33	2.04	2.16	2.02	2.30

Note: F = Low Calcium < 8% CaO, CI = Intermediate Calcium 8-20% CaO, CH = High Calcium > 20% CaO

LA = Low Alkali, HA = High Alkali

A dash (-) represents that no data was available

3.2 ASR Prisms (CSA A23.2-14A, ASTM C 1293)

3.2.1 Mix Procedure

The water/cement ratio was kept between 0.42 and 0.45 as per ASTM C1293. Beginning with 0.42 and adding water up to 0.45 where mix required more water to make the concrete workable. The total cementing materials (Portland cement and SCM where applicable) content in the mixtures were 420 kg/m³. The aggregate proportions were a 60:40 coarse to fine ratio; however, the mix consistency was too coarse. Thus it was decided to use the mix design standard associated with light weight aggregate. This meant that the amount of coarse aggregate used in the mix was determined by using a 0.7 dry rodded density instead of 60:40 coarse to fine ratio. The necessity to use the dry rodded density was due to the RCA being more porous and slightly lighter than limestone aggregate. Using 0.7 dry rodded density of the coarse RCA was still not sufficient because the mix was slightly too coarse. Finally, a 0.69 dry rodded density of the coarse RCA was found to produce an acceptable consistency; hence, it was used.

In order to increase the total alkalinity of the concrete to 1.25% by mass of the cement, NaOH was dissolved in the total mixing water in accordance with ASTM C1293. However, NaOH was dissolved in ½ of the total mixing water and the other ½ of clean mixing water was mixed for one minute with the coarse RCA prior to adding the cement and the non-reactive fine aggregates. The remaining water (with the NaOH) was then added to the mixer. This was done to prevent the

NaOH that was dissolved in the water from being rapidly absorbed by the high porosity RCA.

The mix sequence involved one minute of mixing the coarse RCA with $\frac{1}{2}$ the total clean mix water, then the mixer was stopped and the cement was added on top the RCA and the fine aggregate was added on top the cement. The mixing time was: three minutes mix, two minutes stopped, two minutes mix, two minutes stopped, and 1 minute mix. The $\frac{1}{2}$ NaOH mix water was slowly added in the first 30 seconds, after the PC and SCMs were introduced.

The molds were filled in two layers with concrete. Each layer was tamped with a metal tamper 20 times and the ends, where the studs were, were spaded 10 times. This was done to insure that the studs were covered with concrete. Each layer was also consolidated by lifting and dropping the ends of the mold. Care was taken to avoid bleeding. The top surface of the material was finished with a metal spade. The samples were cured for one day by covering the molds using damp burlap at room temperature. After demolding, a reading of the initial length was taken and recorded. The samples were then placed in 20 litre buckets with wicking cloth from the inside as per test standard. The wicking material used was Wypall X70, in order to maintain a relative humidity of approximately 100% and placed in a heat room at a temperature of 38°C. Prior to using the buckets, they were tested to assure that they would maintain the humidity and seal the water in. After 3

months of placing the test bucket in the heat room, the bucket lost 13g of water. This was determined acceptable and the samples were placed in similar buckets. It should be noted that every time a measurement was taken, the buckets always contained water.

3.2.2 Concrete Mixes

In order to determine the reactivity of the RCA's a number of mixes were tested. The total alkali content of the concrete was adjusted to determine what alkali content is required to control the reactivity. Four samples were made to study the effects of alkali content; they were 6.3 kg/m³, 5.25 kg/m³, 3.36 kg/m³, and 2.31 kg/m³ alkali content, which correspond to a Portland cement of Na₂O_e of 1.5%, 1.25%, 0.8%, and 0.55%, respectively when using 420 kg/m³ of cement.

Samples with the same total alkali content but, using cement of different alkali content, was also investigated. This was done in order to clarify whether the alkalis released by the cement into the pore solution were as readily available as the alkalis in NaOH form; that are added in the concrete to raise the total alkalinity. Three different alkali content type 10 GU cements were investigated; they were 0.89, 0.96, and 1.06 Na₂O_e.

It was assumed that the alkalis in the RCA contribute to raising the alkali content in the pore solution; therefore, an attempt to remove excess alkalis from the RCA was

performed and samples were casted; they were one control and one with 25% Fly ash F, LA replacement. The coarse RCA was placed in a bucket full of water. At the bottom of the bucket a hose was placed to add a continuous flow of water. The bucket was left with the continuous flow of water, over night. An example of the set up can be seen in Figure 24. The intention was to expel and remove the alkalis from the RCA. During the preparation of concrete mixes, the washed RCA was placed in the mixer in Saturated Surface Dry condition (SSD); therefore, premixing of these aggregates with clean water was unnecessary.

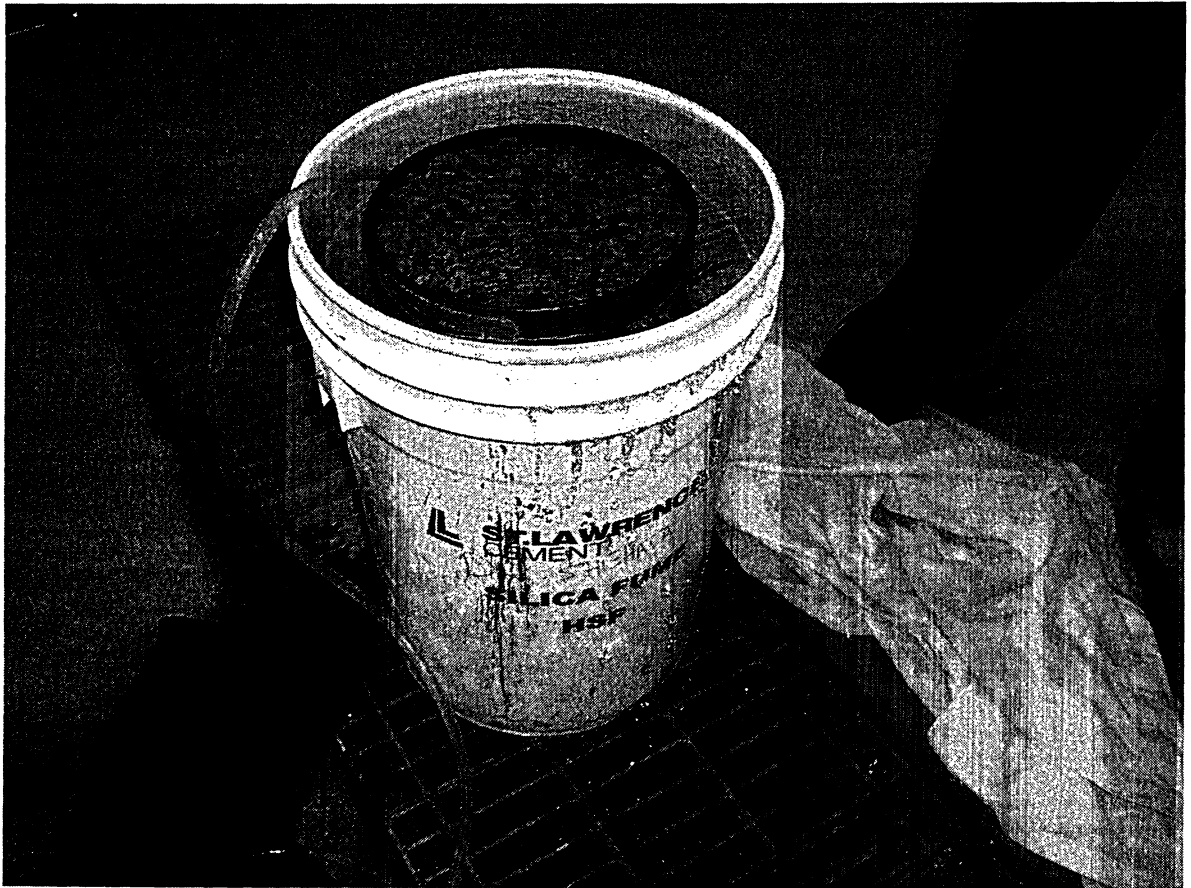


Figure 24: Set-up for removal of alkalis from the RCA (washed RCA).

Different supplementary cementing materials (SCM) were tested to control the expansion of the concrete containing RCA. Three samples of silica fume with 5, 8, and 10 percent cement replacement were tested. It became apparent that adding a greater amount of silica fume to the concrete would make the mixes difficult to work with, thus not feasible to real life applications, especially at levels above 8%. Fly ashes with different chemical compositions were tested in order to investigate the effects of chemical composition of the ash on reducing the expansion. The percentage replacement of the concrete samples with slag and different types of Fly ash can be reviewed in Table 4. Since ternary blends have been found to be effective in controlling ASR (Shehata & Thomas, 2002), mixes using 5% silica and fly ash or slag were made. These mixes can also be reviewed in Table 4.

Furthermore, a number of samples with Spratt coarse aggregate and Jobe fine aggregates were also prepared. These samples were made as a comparison with the RCA samples. The details of these mixes can be reviewed in Table 4.

Table 4: Mix Design for ASR Prisms

Binary Blends		RCA Mixes										Jobe Mixes					Spratt Mixes				
		5	8	10	15	20	25	30	35	40	50	7	10	15	20	25	15	20	25		
	Fly ash F, LA					X	X	X													
	Fly ash F, HA				X	X	X		X					X	X	X	X	X		X	
	Fly ash CI, LA					X		X													
	Fly ash CH, LA #1						X	X			X										
	Silica Fume	X	X	X								X	X								
	Slag						X	X			X										
	Ternary Blends																				
5% SF	FA F LA				X	X		X							X						
5% SF	FA F HA					X	X											X		X	
5% SF	FA CI LA					X	X														
5% SF	FA CH LA					X	X	X													
5% SF	Slag						X	X		X											
5% SF	FA CH LA #3																		X		
3% SF	FA CH LA #2																		X		

NOTE: Chemical composition for all blends can be found in table 2 & 3.

3.3 ASR Mortar Bars (CSA A23.2-25A, ASTM C 1260)

3.3.1 Sample Preparation

The mix procedure was according to ASTM standard C1260. During consolidation, the studs were spaded with a rubber tamper, ten times each in order to ensure proper consolidation around the studs. After the samples were placed in the molds, they were then placed in a mist room. The molds were covered with a thick glass sheet that was placed 2 inches above the samples and was angled to ensure that dripping did not occur.

In accordance with ASTM C1260 the samples were removed the next day and placed in a container of water for one day then in a container of 4% NaOH solution the next day. The containers used were ERA SEAL by Era Ware. They were tested prior to use, to guarantee that the water in the solution would not evaporate. This would raise the concentration of NaOH in the solution. To prevent the loss of water the containers were sealed with GLAD®. Press 'n Seal™ Wrap and the lids placed on top. They would then be placed in the oven with a rectangular piece of wood on top of the containers. Weights were placed on top of the wood to make sure that the pressure from the evaporated water would not lift the wrap and lid and allow the evaporated water to escape. The containers on average would lose 160g of water after 1 month of testing. The amount of solution used was 2700g in order to cover the samples in the containers. Most of the water loss was assumed

to be the run off from the specimens as they were removed and tested, thus this loss would not greatly increase the concentration.

3.3.2 Mortar Mixes

Control samples using natural fine aggregates were prepared in order to determine the reactivity of the fine aggregates. It was discovered that the natural fine aggregates achieved an expansion of 0.20%, which is greater than the CSA 0.1% expansion limit indicating a potential reactivity. Since the ASR prisms were made using the natural fine aggregates, control samples using Ottawa sand were prepared for prisms and mortar bars. The Ottawa sand mortar bar samples were made to determine if the Ottawa sand was reactive. After it was determined that the Ottawa sand was exceedingly non-reactive, a control prism using coarse RCA and Ottawa sand were made to compare against the prisms with coarse RCA and natural fine aggregates. The control sample using Ottawa sand with RCA proved to expand similarly to the control sample with the natural fine aggregates with RCA. Thus the use of natural fine aggregate is assumed to have no effect on the total expansion of prisms containing RCA.

Control samples with fine RCA (primary crushed fine RCA) were also prepared so as to determine the reactivity of the Fine RCA. Although the RCA failed the CSA expansion requirement, the expansion of the fine RCA was found to be small. The fine RCA was initially crushed only once and it was assumed that the majority of

the sand was cement paste particles, thus some coarse RCA was recrushed into fine aggregates (secondary recrushed fine RCA) and a control sample was tested. The results showed that the secondary recrushed fine RCA was much more reactive than the primary crushed fine RCA by almost doubling the expansion as will be shown in Chapter 4. Therefore, the secondary recrushed fine aggregates were used in all other mortar bar mixes.

One of the reasons that the mortar bar test was constructed is to determine if a relationship exists between mortar bars & concrete prisms for samples containing RCA. Accordingly, the mixes prepared with mortar bars are, for the most part, identical to the prisms and can be seen in Table 4. The mixes in Table 4 that were not done with mortar bars were:

- 1) 15% and 35% Fly ash Low Calcium, High Alkali
- 2) 5% Silica Fume 25% Fly ash High Calcium, Low Alkali
- 3) 5% Silica Fume 25% Slag

These mixes were not done due to lack of material. The mixes not listed in Table 4, but were done with mortar bars were 15% and 20% Fly ash High Calcium, Low Alkali. For the purpose of determining the effect of differing alkali content on the reactivity of the mortar bars, control mixes (containing no SCM) were tested in solution of different alkali content. The alkali concentrations examined were 0.89,

0.65, 0.55, and 0.3. The concentration of alkalis in the testing solution was chosen to be 0.7 of the Na_2O_e of the PC used. This value was chosen to represent the alkali level that is likely to be encountered in the pore solution of concrete samples (Thomas et al., 2006). Therefore, the alkalinity of the soaking solution was 0.62, 0.46, 0.39, and 0.2.

3.4 Salt Scaling Testing

3.4.1 Mix Procedure and Curing Method

It should be noted that the coarse RCA used in the salt scaling samples were not the ASR reactive aggregates and was not from the same location, which was done to save the reactive RCA for ASR testing. The mix procedure was according to ASTM C672. Each specimen was rodded 65 times and left to bleed for one hour before final finishing. Since the RCA has an increased porosity and to some extent could be considered a lightweight aggregate, then the air content of freshly mixed concrete by the pressure method was modified according to ASTM C231. Therefore, a correction factor was applied to the air meter reading for both RCA aggregates and in order to be consistent to the limestone coarse aggregate as well.

The samples were cured using damp burlap that was placed a couple inches above the samples. Care was taken to avoid the burlap from touching the samples, since this would add water to the slabs top layer and produce a weaker layer. Mist

curing was avoided for this reason because droplets of water would fall on the top layer.

3.4.2 Mix Design

The mixes that were tested were control samples with RCA and limestone coarse aggregates plus a 5% silica fume, 30% slag samples for RCA and limestone coarse aggregates. Also included are air entrained and non-air entrained concrete samples containing gravel with 5% silica fume, 25% fly ash, 5% silica fume and 20% fly ash, plus a control (no SCM). All mix design and concrete properties for RCA and limestone slabs are shown in Tables 5 through 16. For the RCA control (Table 5) there was an initial slump, a slump taken after 30 minutes, and 40 minutes because the amount of water reducer was excessive. The time was necessary to allow the concrete to have a reasonable slump. After 45 minutes of mixing, the molding and air content were performed. The other mixes were adjusted and reasonable slumps were achieved on the initial reading. The mix of 5% silica fume and 30% slag containing the limestone aggregate had an adjustment as well. Initially the air content was below the acceptable limit, thus more air entrainment was added and the concrete was remixed once for three minutes. The desirable air content was achieved after the remixing. It should also be noted that all the samples were subjected to one additional day in the humidity chamber in order to allow the silicon, used to provide a reservoir for the salt solution, to fully cure.

Table 5: RCA Control Mix Design and Concrete properties

RCA Control		
	Mass (Kg/m³)	Product Name
Coarse Aggregate	963.3	RCA
Fine Aggregate	642.2	Natural Sand
Cement	375.0	Portland Type 10
(0.4 W/C) Water with Correction	188.0	Tap
Water Reducer	3.3	(Eucon 37) Euclid Admixture
Air Entrainment	0.7	(Micro-Air) Master builders Technologies
Date Mixed	22-Jun-06	
Date Stored at 50% RH	06-Jul-06	
Date Stored in Freezer	21-Jul-06	
AIR	7.1%	5.53% After Correction
Initial Slump	220	
30 min Slump	160	
45 min Final Slump	90 mm	

Table 6: RCA 5/30 Slag Mix Design and Concrete properties

RCA 5% Silica 30% Slag		
	Mass (Kg/m³)	Product Name
Coarse Aggregate	957.0	RCA
Fine Aggregate	638.0	Natural Sand
Cement	243.8	Portland Type 10
(0.4 W/C) Water with Correction	187.7	Tap
Slag	112.5	
Silica Fume	18.8	
Water Reducer	1.7	(Eucon 37) Euclid Admixture
Air Entrainment	0.7	(Micro-Air) Master builders Technologies
Date Mixed	22-Jun-06	
Date Stored at 50% RH	06-Jul-06	
Date Stored in Freezer	21-Jul-06	
AIR	8.6%	7.03% After Correction
Initial Slump	110 mm	

Table 7: Limestone Control Mix Design and Concrete properties

Control		
	Mass (Kg/m³)	Product Name
Coarse Aggregate	1065.3	Gravel
Fine Aggregate	710.3	Natural Sand
Cement	375.0	Portland Type 10
(0.4 W/C) Water with Correction	164.5	Tap
Water Reducer	1.7	(Eucon 37) Euclid Admixture
Air Entrainment	0.5	(Micro-Air) Master builders Technologies
Date Mixed	29-Jun-06	
Date Stored at 50% RH	13-Jul-06	
Date Stored in Freezer	28-Jul-06	
AIR	8.5%	6.35% After Correction
Initial Slump	70 mm	

Table 8: Limestone 5/30 Slag Mix Design and Concrete properties

5% Silica 30% Slag		
	Mass (Kg/m³)	Product Name
Coarse Aggregate	1061.2	Gravel
Fine Aggregate	707.3	Natural Sand
Cement	243.8	Portland Type 10
(0.4 W/C) Water with Correction	144.3	Tap
Slag	112.6	
Silica	18.8	
Water Reducer	2.5	(Eucon 37) Euclid Admixture
Air Entrainment	0.5	(Micro-Air) Master builders Technologies
Date Mixed	29-Jun-06	
Date Stored at 50% RH	13-Jul-06	
Date Stored in Freezer	28-Jul-06	
AIR	8.5%	6.35% After Correction
Initial Slump	80 mm	

Table 9: Non-air Entrained Control Mix Design

Non-air entrained Control		
	Mass (Kg/m³)	Product Name
Coarse Aggregate	1103.5	Gravel
Fine Aggregate	715.1	Natural Sand
Cement	400.0	Portland Type 10
(0.4 W/C) Water	160.0	Tap
Final Slump	90 mm	

Table 10: Non-air Entrained 25% fly ash Mix Design

Non-air entrained 25% fly ash		
	Mass (Kg/m³)	Product Name
Coarse Aggregate	1103.5	Gravel
Fine Aggregate	697.6	Natural Sand
Cement	300.0	Portland Type 10
Fly ash	100.0	Type CH
(0.4 W/C) Water	160.0	Tap
Final Slump	90 mm	

Table 11: Non-air Entrained 5% Silica Fume Mix Design

Non-air entrained 5% Silica Fume		
	Mass (Kg/m³)	Product Name
Coarse Aggregate	1103.5	Gravel
Fine Aggregate	710.8	Natural Sand
Cement	380.0	Portland Type 10
Silica Fume	20.0	
(0.4 W/C) Water	160.0	Tap
Final Slump	90 mm	

Table 12: Non-air Entrained 5/20 SF/FA Mix Design

Non-air entrained 5/20 SF/FA		
	Mass (Kg/m³)	Product Name
Coarse Aggregate	1103.5	Gravel
Fine Aggregate	696.8	Natural Sand
Cement	300.0	Portland Type 10
Silica Fume	20.0	
Fly ash	80.0	Type CH
(0.4 W/C) Water	160.0	Tap
Final Slump	90 mm	

Table 13: Entrained Control Mix Design and Concrete properties

Air entrained Control		
	Mass (Kg/m³)	Product Name
Coarse Aggregate	1103.5	Gravel
Fine Aggregate	715.0	Natural Sand
Cement	400.0	Portland Type 10
(0.4 W/C) Water with Correction	156.0	Tap
Water Reducer	2.6	(Eucon 37) Euclid Admixture
Air Entrainment	0.8	(Micro-Air) Master builders Technologies
Date Mixed	Nov 1 2005	
Date Stored at 50% RH	Nov 15 2005	
Date Stored in Freezer	Nov 30 2005	
AIR	8.6%	
Initial Slump	100 mm	

Table 14: Air Entrained 25% Fly ash Mix Design and Concrete properties

Air entrained 25% Fly ash		
	Mass (Kg/m³)	Product Name
Coarse Aggregate	1103.5	Gravel
Fine Aggregate	697.5	Natural Sand
Cement	300.0	Portland Type 10
Fly ash	100.0	Type CH
(0.4 W/C) Water with Correction	156.5	Tap
Water Reducer	0.00	(Eucon 37) Euclid Admixture
Air Entrainment	0.5	(Micro-Air) Master builders Technologies
Date Mixed	Nov 1 2005	
Date Stored at 50% RH	Nov 15 2005	
Date Stored in Freezer	Nov 30 2005	
AIR	7.0%	
Initial Slump	80 mm	

Table 15: Air Entrained 5% Silica Fume Mix Design and Concrete properties

Air entrained 5% Silica Fume		
	Mass (Kg/m³)	Product Name
Coarse Aggregate	1103.5	Gravel
Fine Aggregate	711.0	Natural Sand
Cement	380.0	Portland Type 10
Silica Fume	20.0	
(0.4 W/C) Water with Correction	156.0	Tap
Water Reducer	3.5	(Eucon 37) Euclid Admixture
Air Entrainment	0.8	(Micro-Air) Master builders Technologies
Date Mixed	Nov 2 2005	
Date Stored at 50% RH	Nov 16 2005	
Date Stored in Freezer	Dec 1 2005	
AIR	6.2%	
Initial Slump	110 mm	

Table 16: Air Entrained 5/20 SF/FA Mix Design and Concrete properties

Air entrained 5/20 SF/FA		
	Mass (Kg/m³)	Product Name
Coarse Aggregate	1103.5	Gravel
Fine Aggregate	697.0	Natural Sand
Cement	300.0	Portland Type 10
Silica Fume	20.0	
Fly ash	80.0	Type CH
(0.4 W/C) Water with Correction	156.5	Tap
Water Reducer	1.6	(Eucon 37) Euclid Admixture
Air Entrainment	0.7	(Micro-Air) Master builders Technologies
Date Mixed	Nov 2 2005	
Date Stored at 50% RH	Nov 16 2005	
Date Stored in Freezer	Dec 1 2005	
AIR	5.6%	
Initial Slump	90 mm	

3.4.3 Molding Procedure

In total there were 12 air entrained slabs cast and they had dimensions of 30 X 30 X 8 cm. The molds were made of wood 2 X 4 and thick plywood for the base. Before subjecting the slabs to the freeze thaw cycles, dikes made of foam board were placed on the surface of the slab in order to pool the salt solution on top of the slab. The foam board had dimensions of 2.54 cm X 2.54 cm and was glued to the top surface of the specimen using Lepage PL300 Foam Board Adhesive. GE Silicone II 100% Silicone White was placed in the contact areas of foam board and concrete with the purpose of preventing salt solution from attacking the foam board adhesive and further sealing the concrete in order to allow the solution to pool on top of the sample.

3.5 Freezing and Thawing

3.5.1 Mix Design and Curing Method

The freezing and thawing samples were batched along with the salt scaling samples. This was done with the intention of comparing the samples in the freezing and thawing test with the salt scaling test. This would also reduce any variability between similar mixes. The samples tested were control samples with RCA and limestone coarse aggregates plus a 5% silica fume, 30% slag samples for RCA and again limestone coarse aggregates. The samples tested are from mixes listed in Table 5 through 8.

The aggregate gradation was similar to the gradation used in the ASR prisms. The gradation was 1/3 of the total coarse aggregate retained on sieve 12.5mm, 1/3 retained on 9.5mm, and 1/3 retained on 4.75mm. The maximum size aggregate was 19mm. The fine aggregates used were the natural fine aggregates and these aggregates were not graded; however, the fine aggregates gradation meets the requirements of ASTM C136.

The molds used were the same molds used for the ASR prisms. Thus the dimensions were 75mm X 75mm X 250mm. Studs were included in the samples so as to measure the length change during the freezing and thawing cycles.

The freezing and thawing samples were cured along with the salt scaling samples using damp burlap for the first 24 hours, followed by placement in a mist curing room. The burlap was placed a couple inches above the samples. Care was taken to prevent the burlap from coming in contact with the samples. Again, this was done with the purpose of reducing any variability.

3.5.2 Freezing Procedure

The freezing and thawing procedure was according to procedure A in the ASTM standard C666. Thus, the samples were submerged under water during the freezing and thawing cycles. Each specimen was placed in a metal container with the purpose of maintaining saturation during the cycles.

The samples were carefully removed approximately every 35 cycles of freezing and thawing. The samples were removed from the metal containers and readings were quickly taken in order to preserve the saturation of the samples. Upon removal, the specimens were lightly rinsed to remove deteriorated particles from the surface of the specimens. The specimens were weighed, the lengths were measured using a length comparator, and the fundamental transverse and longitudinal resonant frequencies were obtained using ASTM C 215. The metal containers were rinsed clean and the specimens were immersed again with fresh water after the readings were taken. Each day, water was added if necessary to cap the water level in the metal containers.

Chapter 4

Results and Discussion

4.1 Alkali Silica Reaction (ASR)

4.1.1. Reactivity of RCA

As was mentioned, the prism readings to determine ASR expansion in concrete were performed according to ASTM C1293. Expansions of the tested samples at various ages are listed in Appendix A. Given that the concrete sample contained no SCM in the mix, the required amount of elapsed time is one year according to the ASTM. As can be seen in Figure 25, Spratt control has not completed the required time; however, it is still useful because a general curve can be extrapolated. It is also notable that all the concrete samples in Figure 25 considerably failed the ASTM expansion limit of 0.04%.

It should be restated that the RCA aggregates were crushed from Spratt containing concrete that had deteriorated due to ASR. It was unexpected that the RCA aggregate would expand as it has, similar to Spratt aggregate. It was believed that the sample had discontinued expansion due to the consumption of reactive silica during the 13 years of service as a sidewalk. Also, the gel that was formed would have slowly decreased in absorbing water due to carbonation and the leaching of the gel out of the concrete.

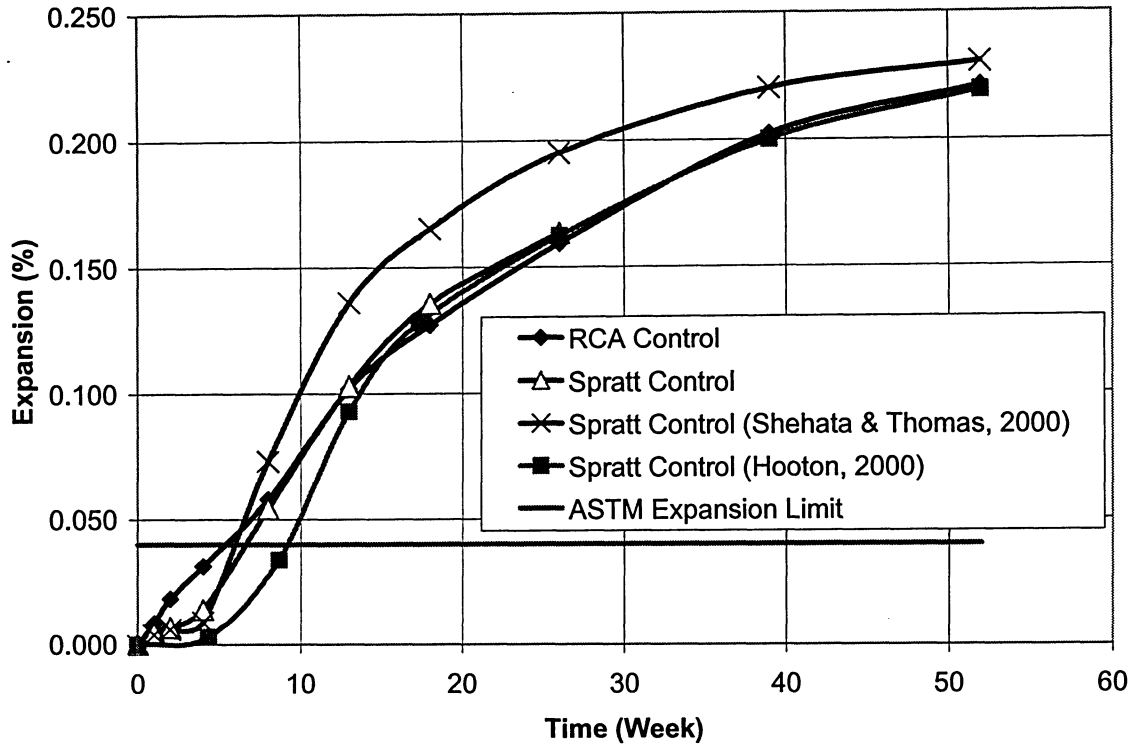


Figure 25: Expansion of concrete prism samples containing Spratt and RCA.

However, it is evident that the use of RCA aggregate still results in expansion of the concrete. Possible reasons are the production of new ASR gel and the expansion due to the old ASR gel absorbing water.

It is clear from comparing the ASR mortar bars with the primary and secondary fine recrushed aggregates that there was still reactive silica that was not consumed during the service life of the old concrete. The primary crushed aggregates, as can be seen in Figure 26, have a reduced expansion compared to the secondary recrushed aggregates. Although both of the fine RCA aggregates failed the ASTM

expansion limit, it is evident that the secondary recrushed expansion is nearly twice that of the primary crushed expansion. This has been concluded to be due to the secondary recrushed aggregates containing more reactive aggregate as a percentage than the primary crushed aggregate. The increased reactive constituents in the secondary recrushed aggregate is assumed to be due to further reduction in cement paste upon recrushing of the aggregate.

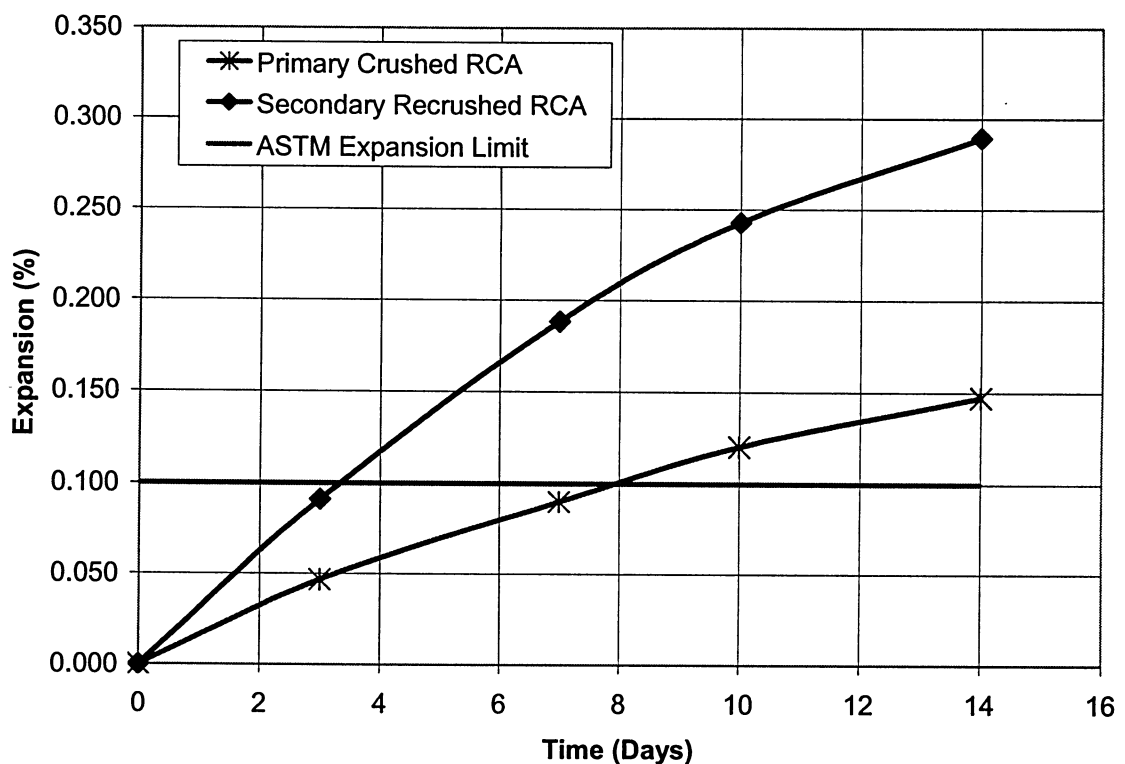


Figure 26: ASR Mortar bar of primary crushed and secondary recrushed recycled aggregates.

Furthermore, when referring back to Figure 25 it can be deduced that the RCA aggregates still contain an appreciable quantity of the reactive constituents because

the expansion is similar to the Spratt aggregate. Therefore, the original concrete contaminated with ASR had an immense reduction in expansion due to either consumption of the alkalis in the pore solution due to the formation of ASR gel or perhaps the silica in the middle section of larger aggregates was less able to react with the alkalis due to reduced permeability or some other factor. Whatever the factors involved, it is evident that the crushing of the concrete and the reintroduction of alkalis results in ASR expansion.

It is also evident in Figure 25, that the RCA aggregates have an early rate of expansion compared to Spratt. This can be contributed to a combination of the old gel expansion but most probably because of the increased permeability observed with RCA, which facilitates the contact of alkalis with reactive silica. In addition, all the types of aggregates (including RCA) in Figure 25 seem to be reducing expansion rate after the first 15 weeks and almost reaching equilibrium at 52 weeks.

When the total amount of alkali contributed by the addition of NaOH and from the cement are varied as seen in Figure 27, then a number of similarities and differences are noticed between the RCA and the Spratt aggregate. One of the similarities is that the decrease of alkalis also decreases the expansion that occurs in both aggregate types. Another, similarity is that the further addition of alkalis beyond a certain point does not increase expansion in both aggregates, indicating

that the silica content in the aggregate is now controlling the reaction that will occur

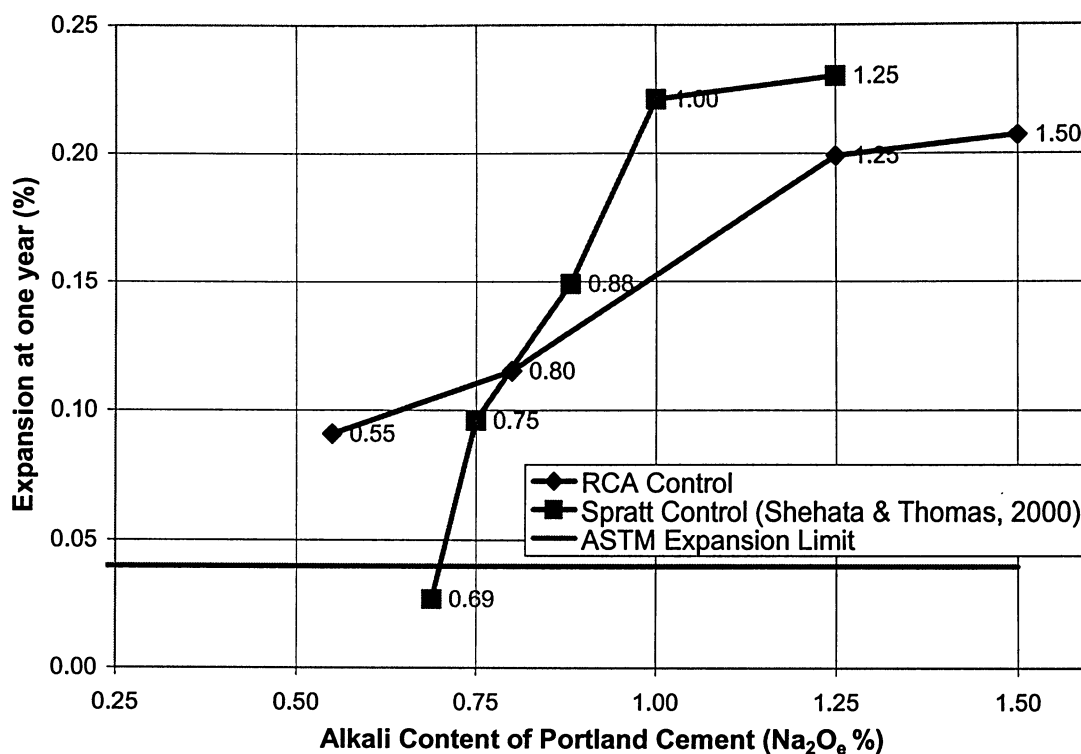


Figure 27: ASR prism expansion comparing RCA with Spratt aggregate when different alkali content is used.

It is interesting to note that the Spratt aggregate reaches acceptable expansion at an alkali content of approximately 0.7 % Na₂O_e whereas, the RCA would require alkali content less than 0.55 % to meet the 0.04 % expansion limit. This would suggest that the RCA contributes to the addition of alkalis in the pore solution. The alkalis in the RCA were probably remaining from the pore solution of the old concrete or recycled from the hydration products or the ASR gel of the old

concrete. This means that in order to produce recycled concrete with acceptable expansion limits, cement of very low alkali content is necessary.

Additionally, the Spratt aggregate has a greater rate of expansion when increasing the alkali content beyond 0.69 compared with the RCA. This would indicate that the Spratt aggregate is more sensitive to Alkali content within this range. For the RCA, the Alkali contributed from the old concrete push the Alkali content of the samples to the level where expansion is not highly affected by Alkali content.

4.1.2 Effects of Binary blends of SCM

Although the 2-year measurements are not yet due, it can be stated that the addition of SCM's has a great benefit in reducing ASR expansion of concrete as depicted in Figure 28. Recall that the 52-week expansion of the control sample was 0.221%. All the SCM's reduce expansion with the increase of replacement level.

The low efficiency high calcium, low alkali fly ash reduces expansion by approximately 45% compared to the control sample at the same time interval.

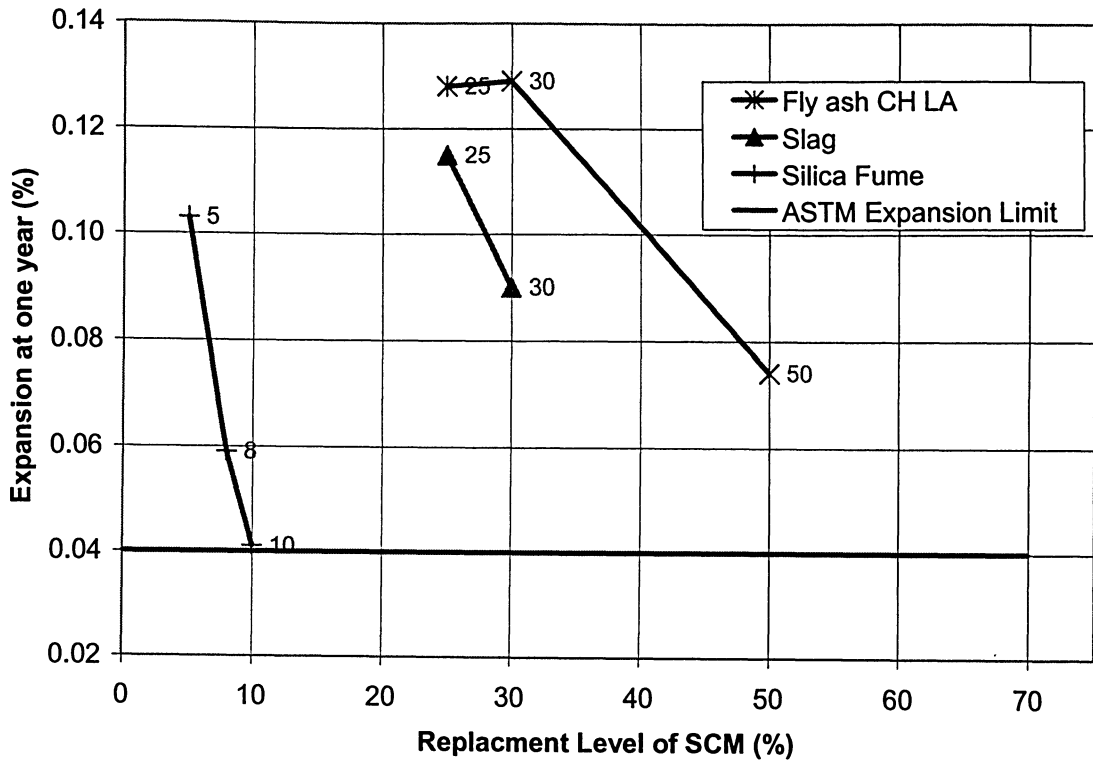


Figure 28: ASR prism expansion comparing the use of different SCMs at different cement replacement levels in concrete containing RCA.

It is clear that a smaller amount of silica fume is required compared with the other SCM's; however, it is not feasible to use greater than 10% for silica fume due to the properties of the concrete at such levels. It is apparent that the equivalent quantity of slag has a greater benefit in reducing the expansion when compared to high calcium, low alkali fly ash. The fly ash in Figure 28 has somewhat similar chemical composition with slag compared to the other fly ashes used. Referring to Table 2 and 3, the alkali of the slag is less than the fly ash by 1.2%; whereas, the calcium of the fly ash is 14.5% less than the slag. This could indicate that at similar chemical

compositions the slag either has a greater capacity in absorbing alkalis into the C-S-H that is formed or it produces more C-S-H compared to the fly ash.

Although the samples were tested for one year it can be stated that when referring to Figure 28, in order to suppress expansion using silica fume a replacement level greater than 10% is required. For the fly ash used it can be extrapolated that a replacement level greater than 70% is required and for slag a replacement level greater than 45% is required. In any case, it is apparent that the SCM's must have a greater replacement level in order to reduce ASR expansion to acceptable limits.

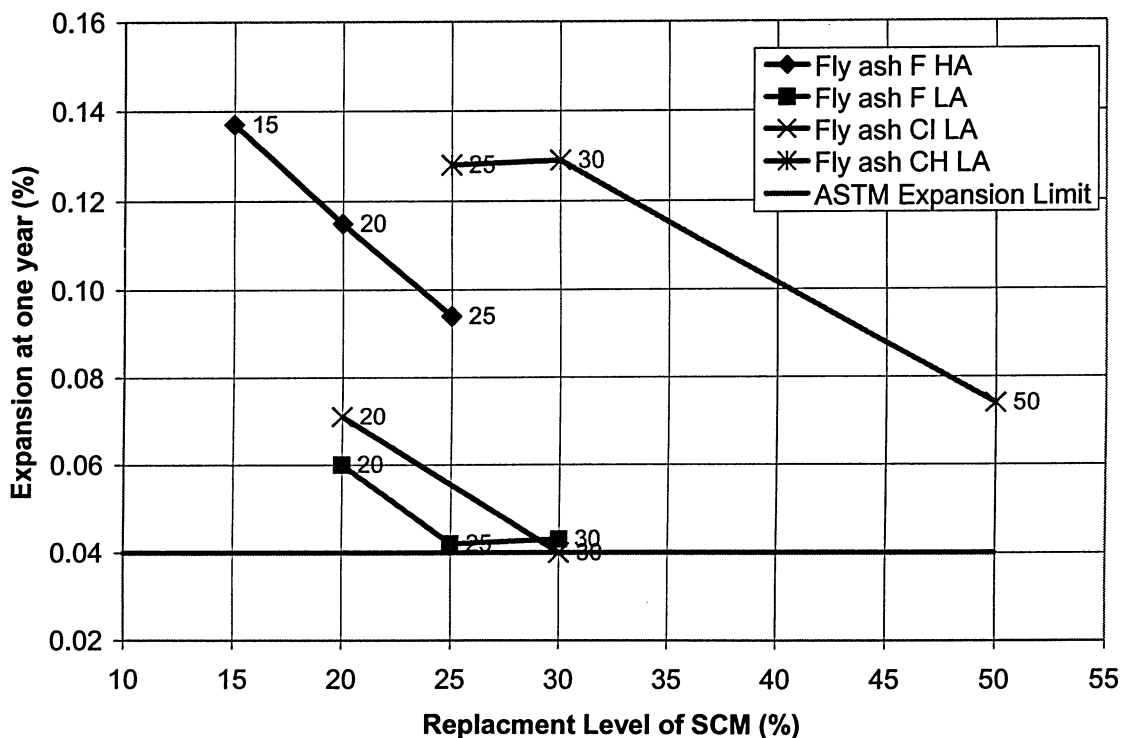


Figure 29: ASR prism expansion comparing the use of fly ash with different chemical composition at different replacement levels.

It can also be seen in Figure 29 that the more fly ash that is added the greater the ability of the concrete to resist ASR expansion. Again due to the fact that these are only 1 year readings, conclusive results on how much is required to limit expansion can not be made; however, it is useful in comparing the effects of using fly ashes with different chemical compositions. For instance, it can be deduced that an increase in the calcium content in the fly ash decreases its ability in reducing expansion; as can be seen when comparing low calcium, low alkali fly ash with high calcium, low alkali fly ash in Figure 29. A similar observation can be made when comparing the alkali content of the low calcium high alkali fly ash with the low calcium low alkali fly ash in Figure 29. The increase in alkalis increases the expansion in the concrete. It is interesting to note that the moderate calcium fly ash has a similar expansion as the low calcium fly ash. Since the only noticeable difference between these fly ashes is the calcium content it can be stated that the observations made by Shehata & Thomas (2000), that the calcium content beyond 20% significantly increases expansion, can apply to this case as well.

Referring to Figure 29, replacement levels of above 30%, 30%, 40%, and 70% of fly ash F LA, fly ash CI, LA, fly ash F HA, and fly ash CH HA, respectively are required to produce concrete within the acceptable limit after one year.

It would seem that the use of fly ash does not produce the required reduction in expansion with RCA samples as it does with the original Spratt aggregate samples.

This could be explained by the increase of alkalis that the RCA aggregates exhibit. Figure 30 shows a comparison of RCA samples containing fly ash F LA with Spratt aggregate samples containing fly ash F LA from Shehata & Thomas (2000). All the samples contain a 20% fly ash F LA replacement; however, the sample labelled as 20% *Fly ash F LA Spratt added alkalis* (Shehata & Thomas, 2000) was produced at an alkali level of 1.25% of total cementing materials rather than 1.25% of Portland cement in the case of the other two samples.

As can be seen in Figure 30, the RCA aggregates have a greater expansion compared with the original Spratt samples and similar expansion to the increased alkali Spratt sample. The Spratt sample with the added alkalis expanded more than doubled the expansion of the Spratt sample without the added alkalis. This shows that the mechanisms that fly ash has in reducing expansion is greatly influenced by the total amount of alkalis in the pore solution, if high level of alkalis is present in the pore solution, part of it will be consumed by the C-S-H of the fly ash and the rest would be available for reaction. This can also be seen in Figure 31 which shows Spratt samples containing several types of fly ash CI, LA with different amount of alkalis. Significant increase in expansion was observed when the fly ash had greater amounts of alkalis. For the expansion of the RCA samples containing fly ash CI LA at one year (0.055%) the equivalent alkali content of similar fly ash would be 4.7%, which is approximately 2.7% greater than the actual alkali content of the fly ash, as shown in Table 3. This again would indicate that

the increase in expansion must be related to the increase in alkali content. This would suggest that the RCA having an expansion greater than the original Spratt samples is due to alkalis contributed by RCA.

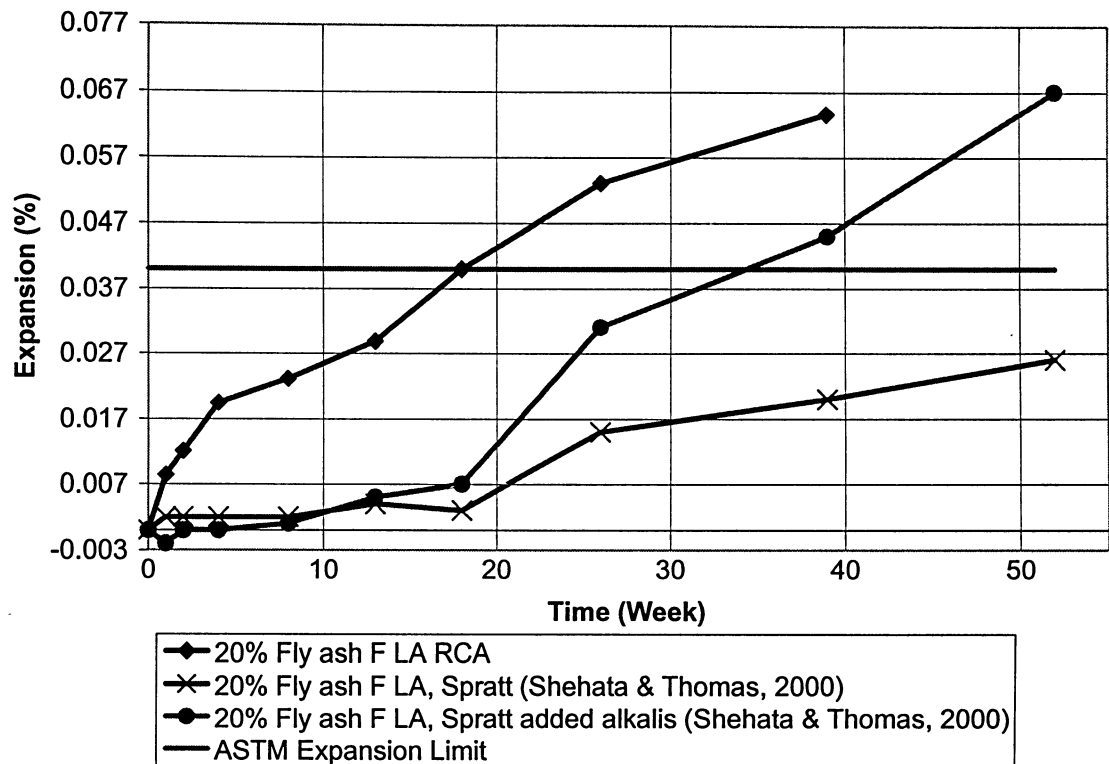


Figure 30: ASR prism expansion comparing RCA containing fly ash with Spratt containing fly ash at different alkali content.

However, since the Spratt aggregate with the added alkalis started with a slow rate of reaction along side the Spratt without added alkalis, it would then imply that the immediate increase in expansion with the RCA in Figure 30 is not only due to the increase in alkalis but some other factor as well. This phenomenon was also observed in the comparison with the control samples in Figure 25; therefore, the

immediate increase could be deduced to be caused by the expansion from the absorption of water of the old ASR gel present in the RCA. The availability or the high concentration of alkalis could also be a contributing factor.

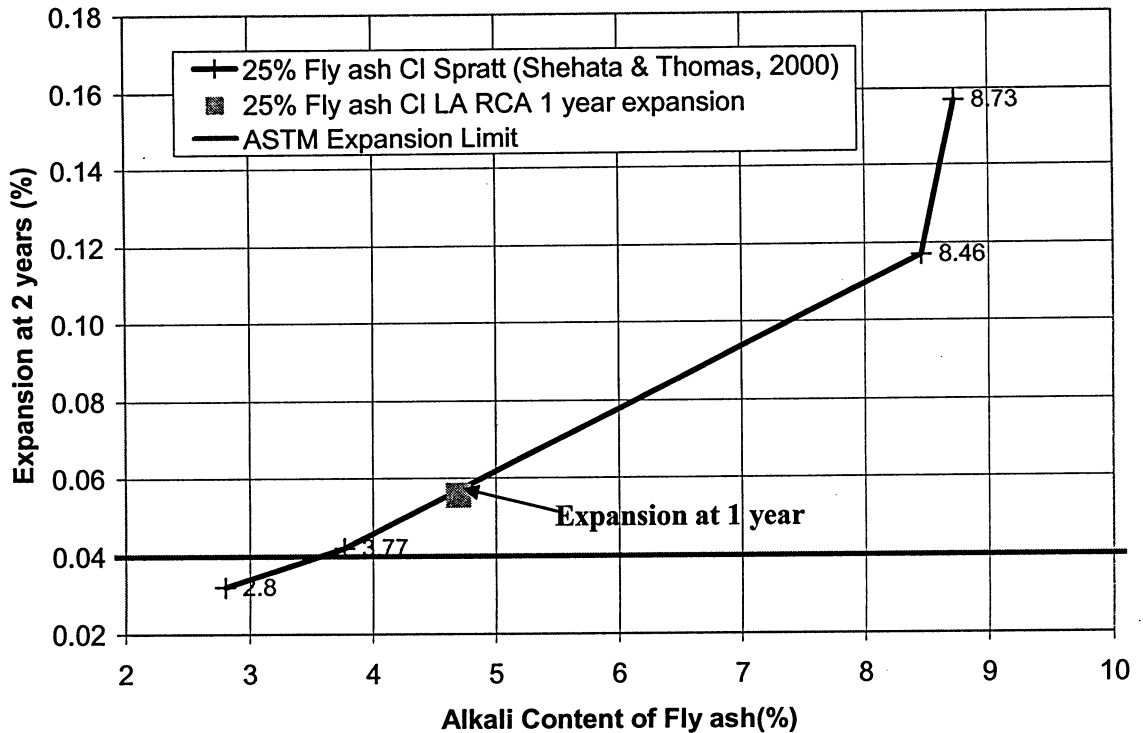


Figure 31: ASR prism expansion comparing RCA containing FA CI LA with several Spratt samples containing FA CI with varying alkali content.

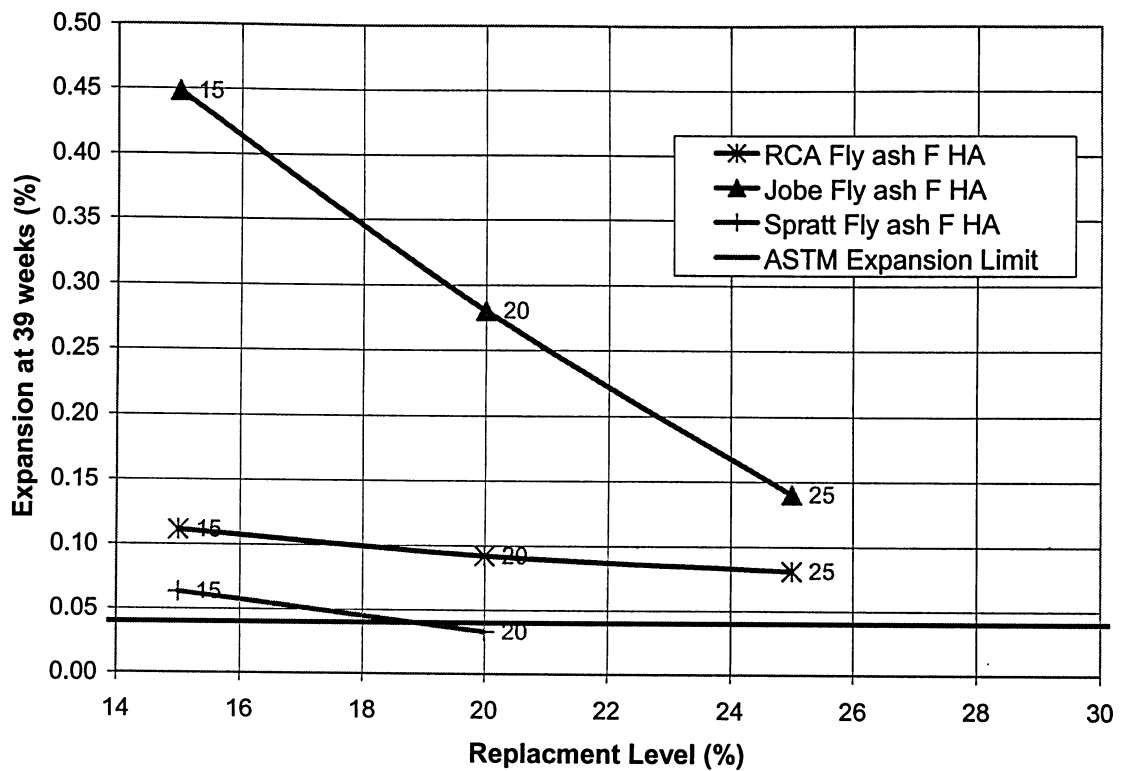


Figure 32: prism expansion comparing the reactivity of RCA, Spratt, and Jobe aggregates.

It is interesting to note when comparing different aggregates, each exhibits a different decrease in the rate of expansion when adding a SCM, as seen in Figure 32. The increase in fly ash F HA has a significant effect with Jobe aggregates, which can be considered highly reactive aggregates. A similar decrease can be observed with the addition of fly ash to the Spratt aggregates. However, the RCA demonstrates minimal reduction in expansion with the increase of fly ash. This could be contributed to the fact that the RCA have shown to contribute to the alkalis in the pore solution. The availability of calcium from the old paste may

have also reduced the ability of the hydration products of SCM to bind alkalis (by raising the Ca/Si ratio of the CSH).

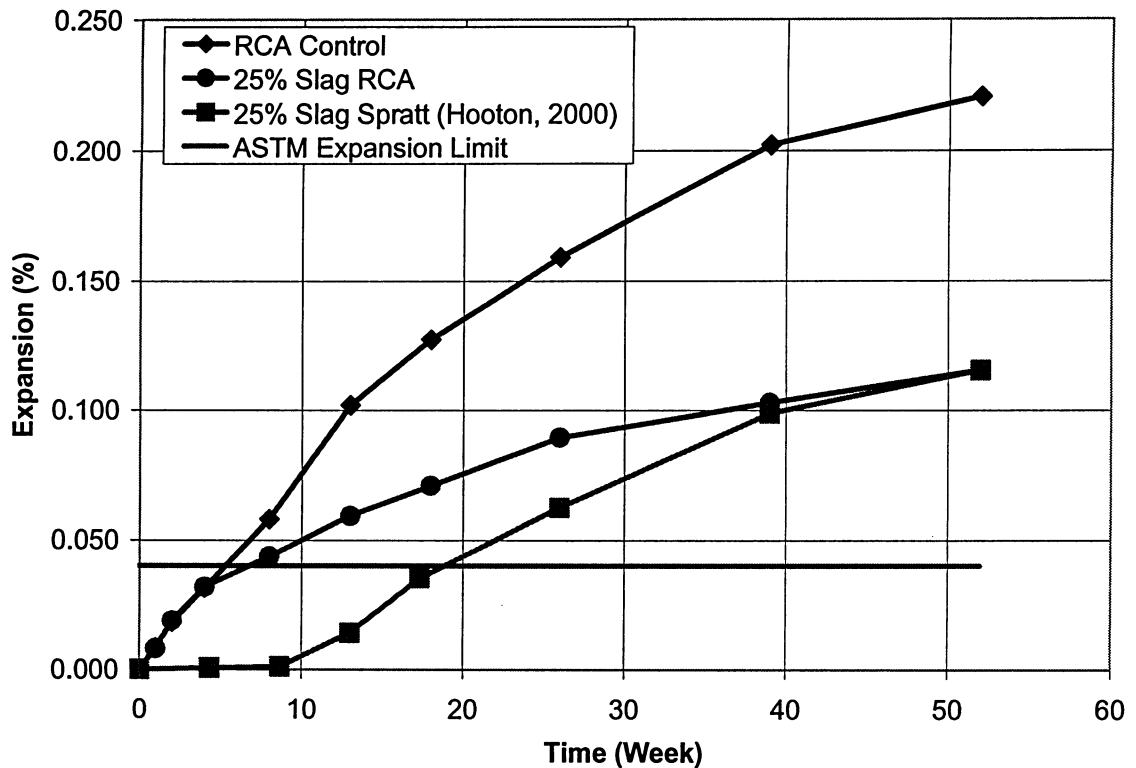


Figure 33: ASR prism expansion comparing the effect of slag on expansion of concrete containing Spratt with concrete containing RCA.

In Figure 33, a comparison between the effects that slag has with Spratt aggregate and the effects it has with RCA can be reviewed. The same slow expansion in the first 10 weeks for Spratt aggregate can be seen which confirms the rate observed with fly ash. It can also be seen that the Spratt aggregate initially has a slow reaction but with time it expands similarly with RCA. This was not observed with the fly ash and could be explained by the assumption that RCA samples containing slag are less affected by the increase in alkalis or calcium content than the RCA

exhibits. Unfortunately the 50% replacement level has not completed its one-year study and could not be included in this study.

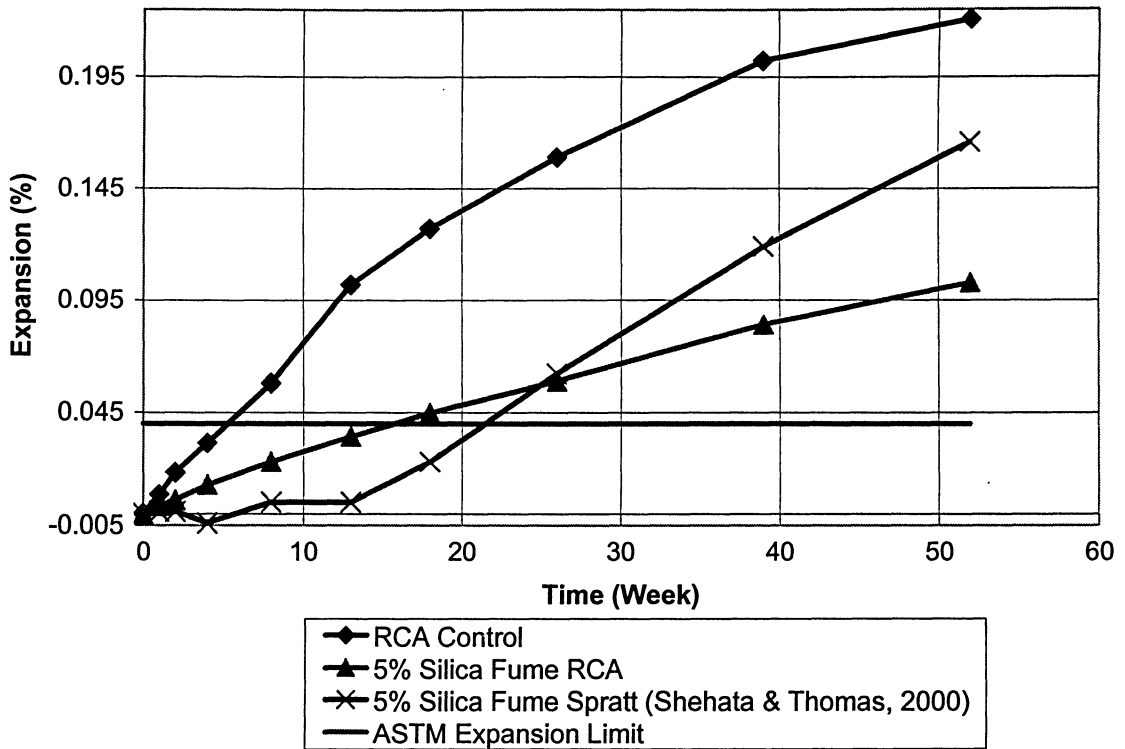


Figure 34: ASR prism expansion comparing samples with RCA containing silica fume with Spratt containing silica fume.

It is interesting to note in Figure 34, that the concrete containing 5% silica fume and Spratt aggregate has a greater expansion than the concrete with RCA. This has not been noticed in any other SCM. This can be explained by the mechanism that silica fume has demonstrated when applied to concrete containing ASR aggregates. As mentioned previously, when silica fume is added to the concrete the reaction of ASR is delayed because the silica reacts with the alkalis in a similar manor as ASR. The delay is dependent on the release of CaO into the pore solution. As the

production of $\text{Ca}(\text{OH})_2$ continues the silica releases the alkalis and combines with the Ca. With silica fume, the increase of alkalis has shown to have an expansion similar to the control and in some cases exceeding at low levels (Shehata and Thomas, 2002). With RCA containing silica fume, one would expect that alkalis contributed from the RCA should have resulted in an increase in expansion compared with the Spratt aggregate; however, this was not the case. It can be deduced that the RCA contains CaO from the previous concrete pore solution. This increase in CaO would produce C-S-H products similar to the other SCMs, which would not release alkalis to the pore solution. Furthermore, the addition of silica fume reduces the permeability of the concrete, which may have reduced the initial expansion resulting the swelling of the old ASR gel by reducing the rate at which water reaches the gel. The reduced permeability also reduces the rate at which alkalis reaches the site of the reactive silica and cause expansion.

4.1.3 Effects of Ternary blends of SCM

Recall from Figure 29, that no SCM alone could reduce expansion due to ASR below the 0.04% after one year. It is clear from Figure 35 that when adding silica fume to the concrete samples, the low alkali fly ashes containing moderate calcium and low calcium are able to reduce expansion below the limit. Silica fume with the 30% cement replacement of slag and fly ash CH LA are also able to reduce the expansion to acceptable limits after one year. This does not mean that the ternary blends shown in the figure will meet the 0.04% limit after two years, but it does

give some hope in producing a viable mix; especially the fly ash F LA and fly ash CI LA. The ternary samples containing slag and silica fume have the greatest decrease in the expansion when a greater replacement level is used. It could be that the increase in calcium to silica ratio (Ca/Si) of CSH and the low alkali content of both SCMs have contributed to reducing the expansion. The increased replacement level of the fly ash F HA does not have a noticeable contribution in reducing expansion. This is most likely attributed to the high alkali content of this fly ash.

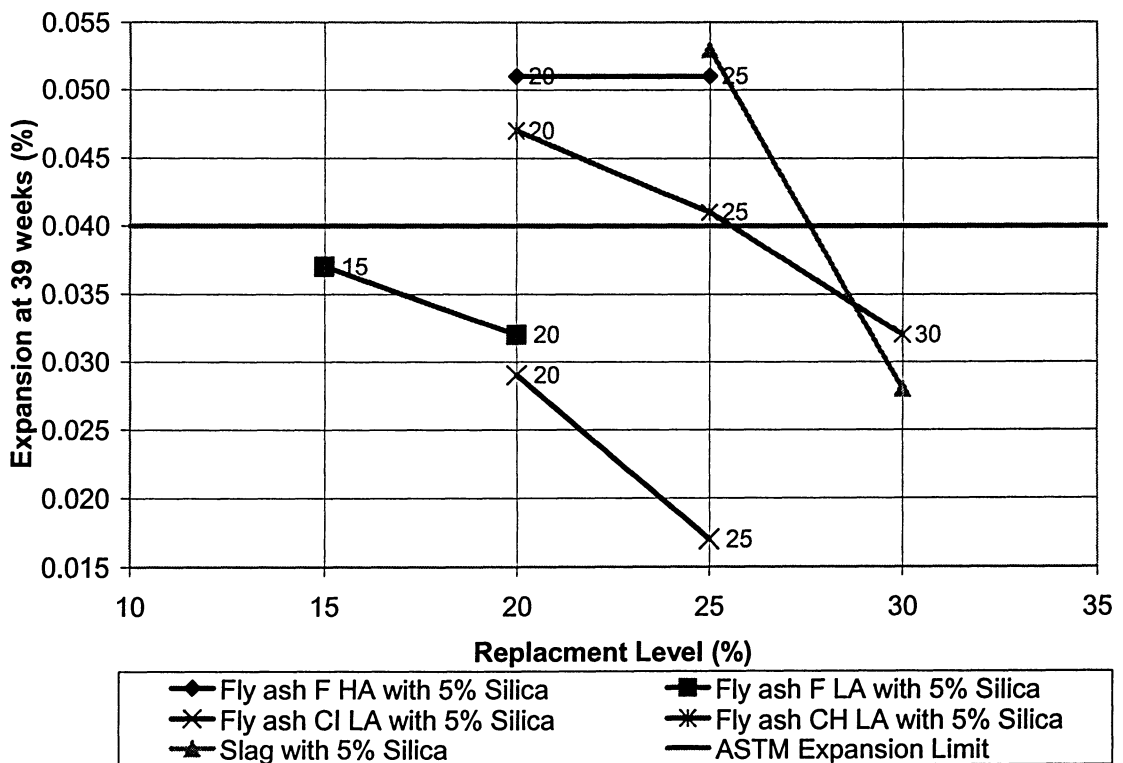


Figure 35: Prism expansion comparing fly ash of different chemical composition used in ternary blends.

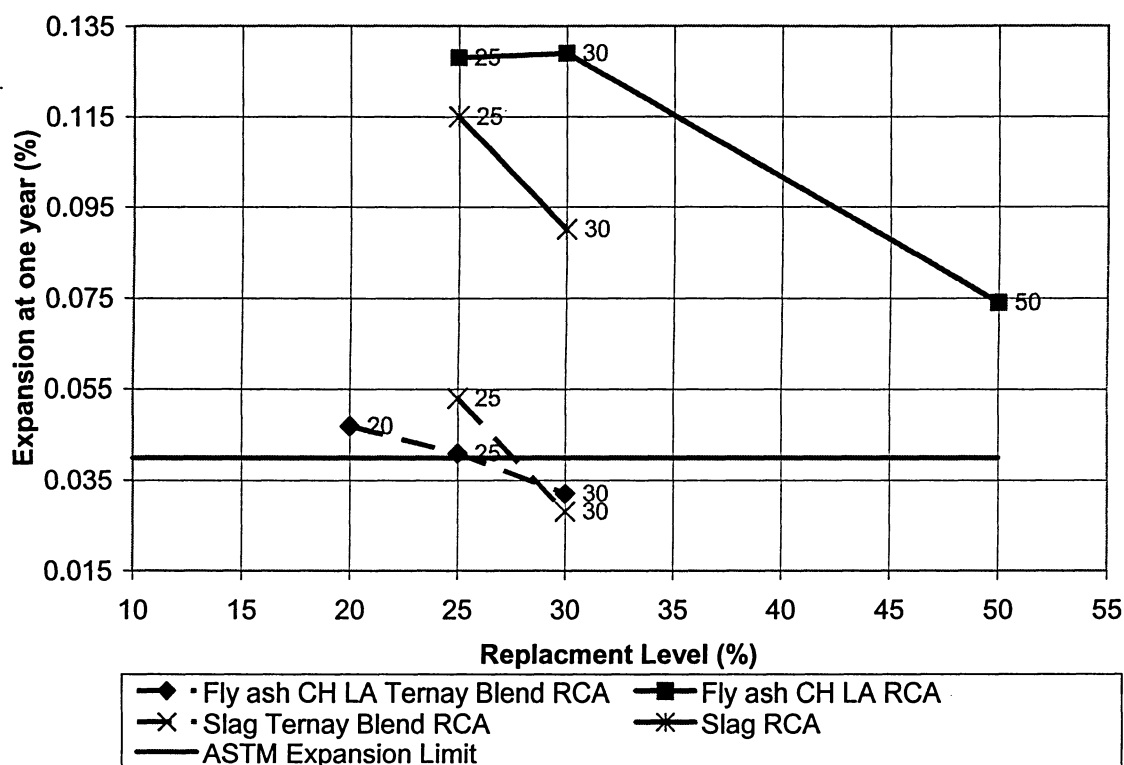


Figure 36: Prism expansion comparing samples containing binary blends verses ternary blends.

It is evident when comparing the mixes with a single SCM and mixes with ternary blends of SCMs that the addition of silica fume, even at low percentages, has a beneficial effect. It can be seen in Figure 36 that an addition of 5% silica has reduced the expansion of the fly ash CH LA to a 1/4 of the binary sample; whereas, the sample with slag is reduced by 1/3 the binary sample. In other words, the fly ash CH LA sample could be extrapolated to require above 70% cement replacement in order to reduce the expansion below the limit; however, with the addition of 5% silica the fly ash requires above 25% replacement. The

same is true for the slag, which required above 45% and with the addition of silica fume requires 30%.

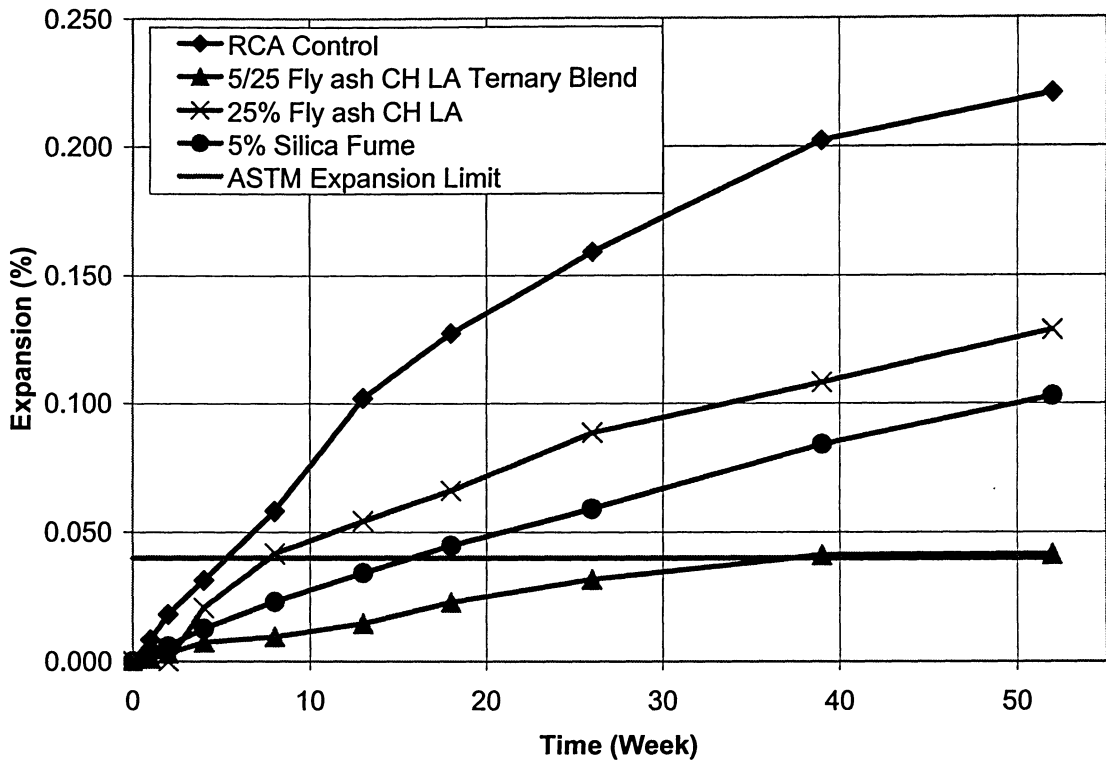


Figure 37: Prism expansion comparing samples containing 5% silica fume, 25% fly ash CH LA, and 5/25 SF/FA.

Shehata & Thomas (2002) describe a “synergy” that exists when silica fume was combined with fly ash. Figure 37 depicts a comparison between samples containing silica fume or fly ash CH LA and a ternary blend with both SCMs. The purpose of this comparison was to determine if the “synergy” occurs with RCA. The reduction in expansion from the use of fly ash compared with the control is 0.092% and the reduction in expansion from the use of silica fume is 0.118%. The combined reduction in expansion would be 0.210%; whereas, the reduction in

expansion of the ternary blend is 0.180%. Although the Figure 37 displays that the use of a ternary blends is superior to the use of a binary blends, it is clear that the “synergy” does not occur similarly as determined by Shehata & Thomas (2002). The same conclusions can be made with the use of slag as seen in Figure 38. The use of ternary blends is superior to binary blends; however, it can be visible that a “synergy” does occur but it does not have a noticable impact as determined by Shehata & Thomas (2002).

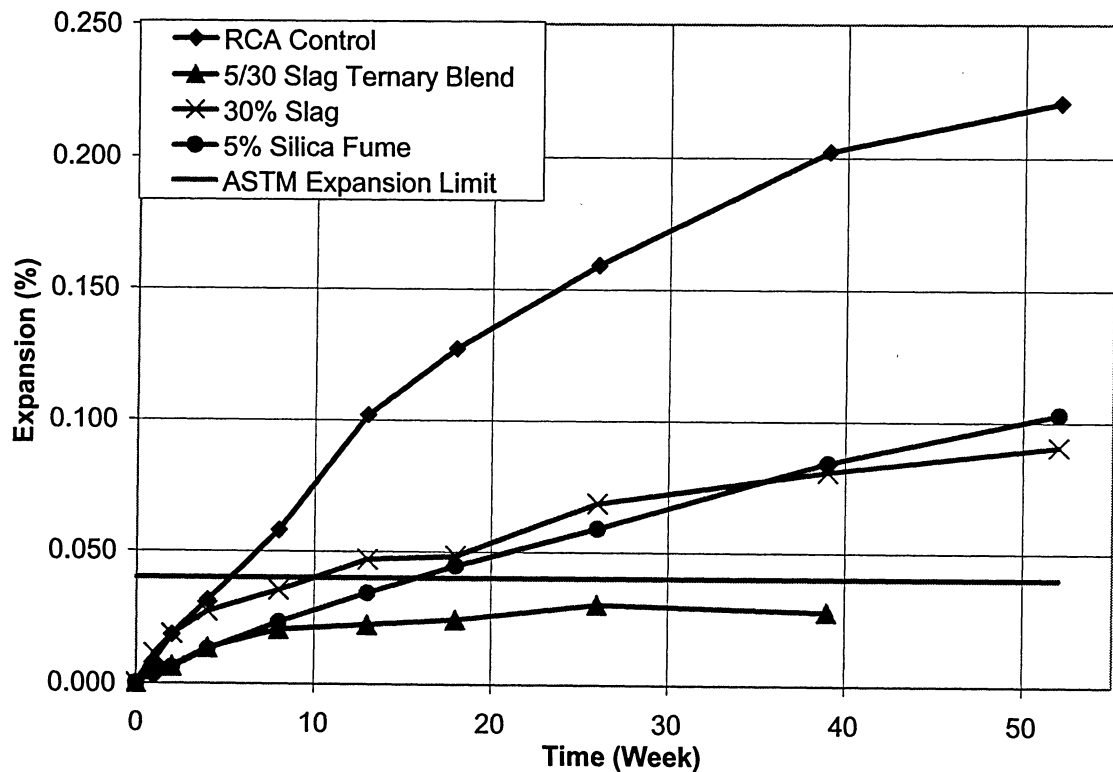


Figure 38: Prism expansion comparing samples containing 5% silica fume, 30% slag, and 5/25 SF/Slag.

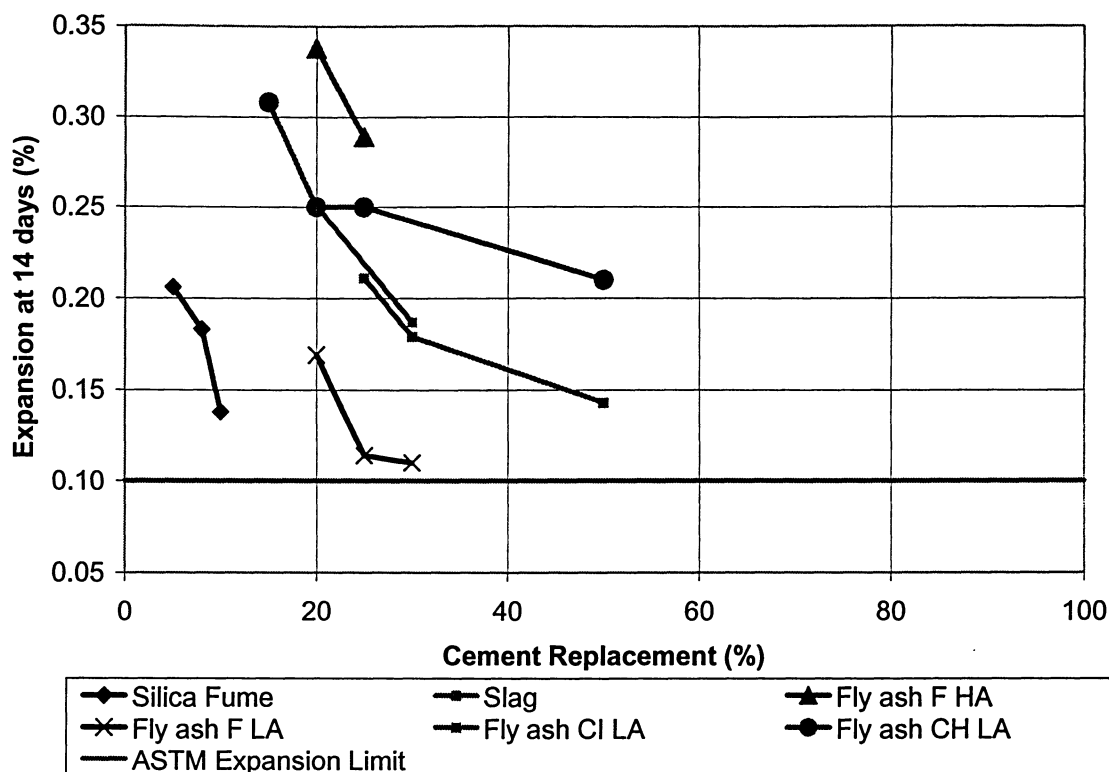


Figure 39: Mortar bar expansion comparing different SCMs with varying cement replacement.

4.1.4 Relation between accelerated Mortar bar and concrete prism test

Several authors have determined a good correlation when comparing the prism test against the mortar bar test (Shehata & Thomas (2000), Hooton (2000)). Results of the mortar bar test are shown in Appendix B. Figure 39 presents results of binary blends using the mortar bar method. Despite the fact that the prism results in this study are of one-year values, several conclusions using the results in Figure 39 can be deduced. The first is that the increase in a SCM results in a decrease in expansion. This was also observed in the prisms. Secondly, replacement levels required to reduce expansion to an acceptable level are great and would most

likely not be feasible in most construction projects. According to the figure the amount of silica fume required to reduce expansion to tolerable limits would be 12%. Slag would require greater than 76%. Fly ash would require greater than 30%, 44%, and 45%, using F LA, CI LA, and F HA, respectively. Thirdly, the increase in alkalis and calcium increases the expansion when using fly ash, or put another way, the high alkali and calcium levels in RCA reduces the efficiency of FA in mitigating the expansion.

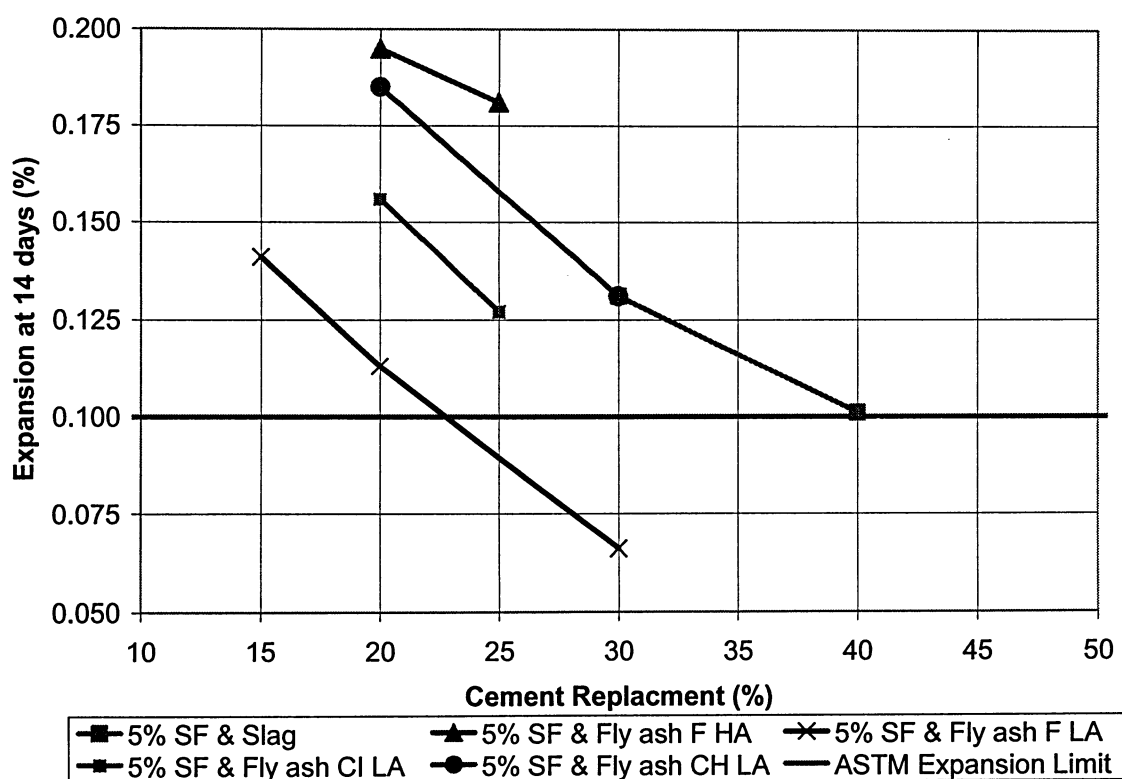


Figure 40: ASR Mortar bar expansion comparing different ternary blends with varying cement replacement.

It follows that the same observations noted in the prisms would also apply when comparing ternary blends using the mortar bar method as seen in Figure 40.

Again the use of ternary blends is superior to the use of binary blends. The increase in alkalis and calcium increases the expansion in fly ash mixes. Whereas the binary samples required 30%, 44%, and 76% using fly ash F LA, CI LA, and slag, respectively to obtain acceptable limits. The ternary requires 23%, 29%, and 40% when using fly ash F LA, CI LA, and slag, respectively. In general, when viewing individual cement replacement levels it is clear that the ternary blend reduces expansion in all cases. For example the 25% fly ash F HA has a greater expansion then the 20% fly ash F HA with 5% silica fume.

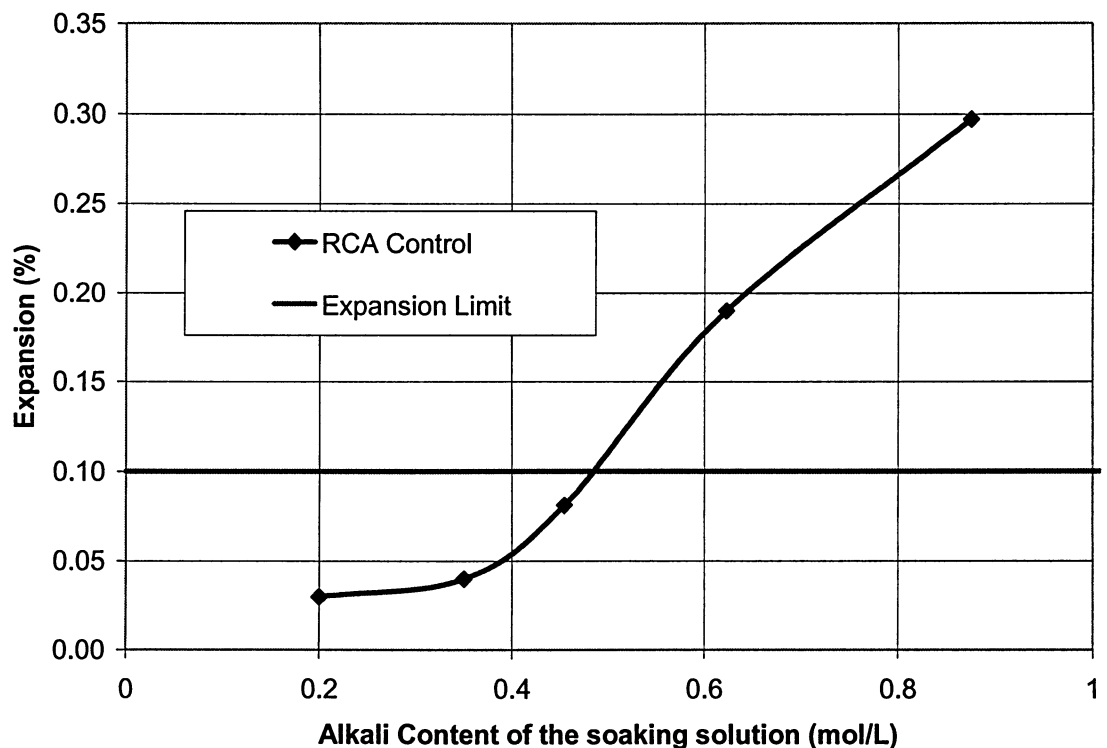


Figure 41: ASR Mortar bar expansion of RCA control samples soaked in solution of different alkalinity levels

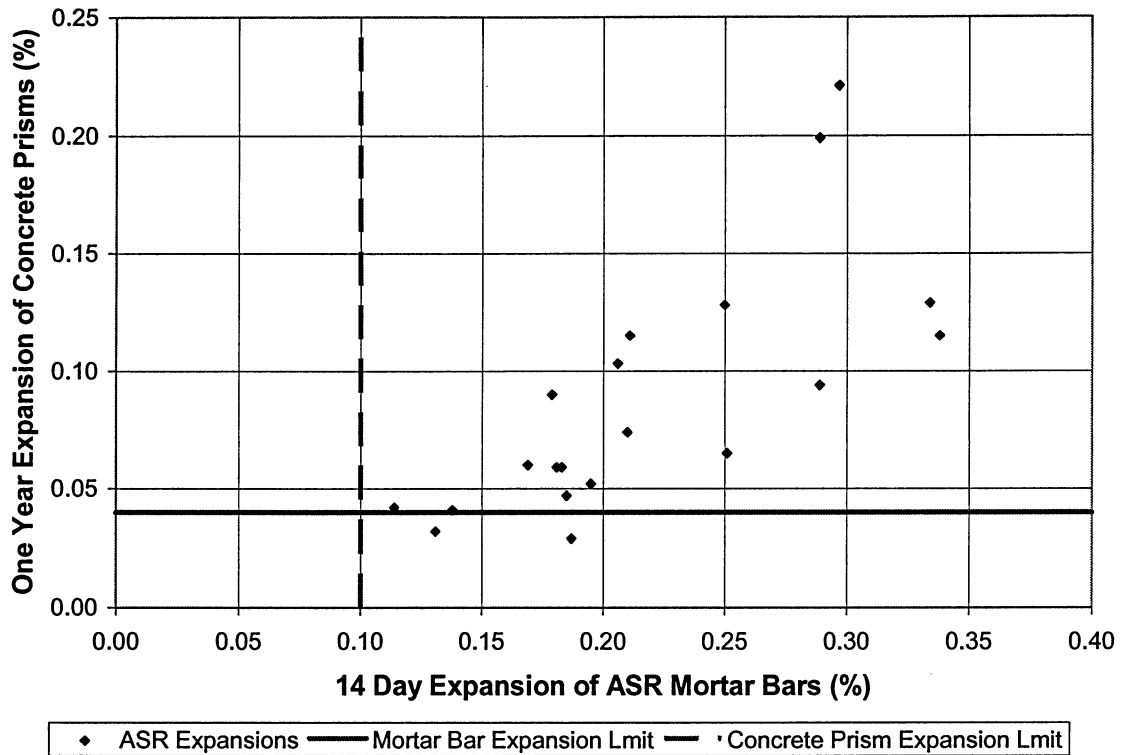


Figure 42: Comparison between ASR prisms at 1 year and ASR mortar bars at 14 days

When reviewing Figure 41, it becomes apparent that the increase in alkalis increases the expansion. This was also, seen in Figure 27 with the RCA control prism samples. Though extrapolation in Figure 27, the RCA control sample indicates that the optimum alkali content that would reduce expansion to below 0.04% limit would be approximately 0.34%. Whereas in Figure 41, the optimum alkali content would be approximately 0.5%. It could be deduced that the high alkali level of the soaking solution of the mortar bar test has masked the effects of alkalis contributed from RCA. In Figure 42, it is clear that the samples that failed the mortar bar test will fail the prism test. This confirms that the accelerated

mortar bar can be used to evaluate the efficiency of SCM in counteracting ASR in concrete containing RCA.

4.2 Salt Scaling

4.2.1. Effects of SCM on salt scaling

Results of the salt scaling test are shown in Appendix C. It should be noted that the non-air entrained slabs were repaired at 30 cycles due to the deterioration of the dikes that were containing the solution on the surface of the slabs. This changed the surface area of the slab, which was in contact with the salt solution from 23 X 23 cm to 19.5 X 19.5 cm. This change only affects the 50-cycle reading. The results of the 50 cycles of non-air entrained can be seen in Figure 43. It can be seen that all non-air samples have unexpected readings at 50 cycles which probably resulted from the repair that occurred; regardless, all non-air samples failed the 0.8 Kg/m² limit set by the MTO after 15 cycles which would make the 50th cycle readings unnecessary. Any SCM's added to a concrete mix are expected to improve bleed properties of the concrete; however, as expected (Pigeon et al., 1996) the fly ash greatly reduces the salt scaling resistance of the concrete. Also, the ternary blend is the worst in terms of salt scaling resistance. The ternary was supposed to at least surpass the fly ash because of the addition of the silica, which significantly reduces bleeding. It may be that the addition of silica slightly reduced the ability to properly finish the concrete surface thus; the surfaces of the samples were over finished in order to obtain the same finish as the fly ash mix. It might also depend

on the amount of water reducer that was added and the slumps obtained from the mix. A greater slump could increase the chance of segregation and in turn bleeding. Whatever the case maybe, the addition of fly ash seems to adversely affect the salt scaling resistance of the concrete. Again the results obtained for the silica fume slab were slightly unexpected. The silica would greatly reduce bleeding, therefore improving the surface resistance of the slab and performing better than the control slab. However, these are non-air entrained samples and this could have been a significant contributing factor. Again this might be accounted for by over finishing or the slump of the slab. Although there is a difference between the silica and the control, the difference is minimal and the silica could be viewed as having a similar effect on salt scaling resistance as the control slab.

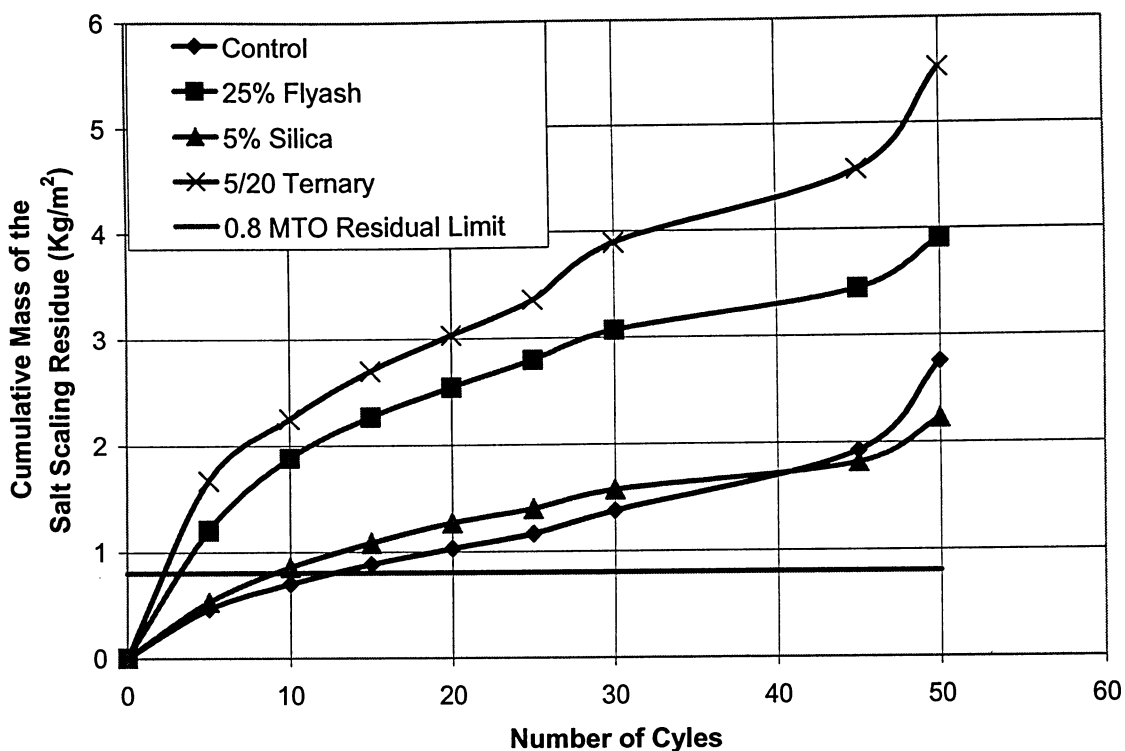


Figure 43: Salt scaling results of the 50 cycles of freezing and thawing on non-air entrained samples containing natural silicious stone.

4.2.2 Effect of air entering on salt scaling

As was expected the air entrained mix does perform better than their non-air counterpart as shown in Figure 44. It should be noted that the sample labelled A were cast and prepared first, followed by the samples labelled B which partially included some concrete from the air entrainment test. The first 2 inches from the air content sample were removed and some portion of the concrete in the container was used. This may have affected the results of the samples labelled B containing fly ash. Furthermore, the ternary blend A and B seem to have an irregularity with the 50th cycle. Disregarding the 50th cycle of ternary B and extrapolating the end

result, it is clear with the exception of fly ash B that most of the air-entrained concrete passes the MTO of 0.8 Kg/m². It can also be deduced that the concrete containing fly ash is more sensitive to changes in the concrete. It would appear that the addition of air entrainment has a greater impact with the ternary blends compared to the fly ash as reflected from the fact that the fly ash samples have a greater amount of residue with air entrainment compared to non air entrainment. It would also appear that the air entrainment combined with the use of silica fume has a greater resistance to salt scaling compared to the control sample. This could be attributed to less bleeding in case of SF samples.

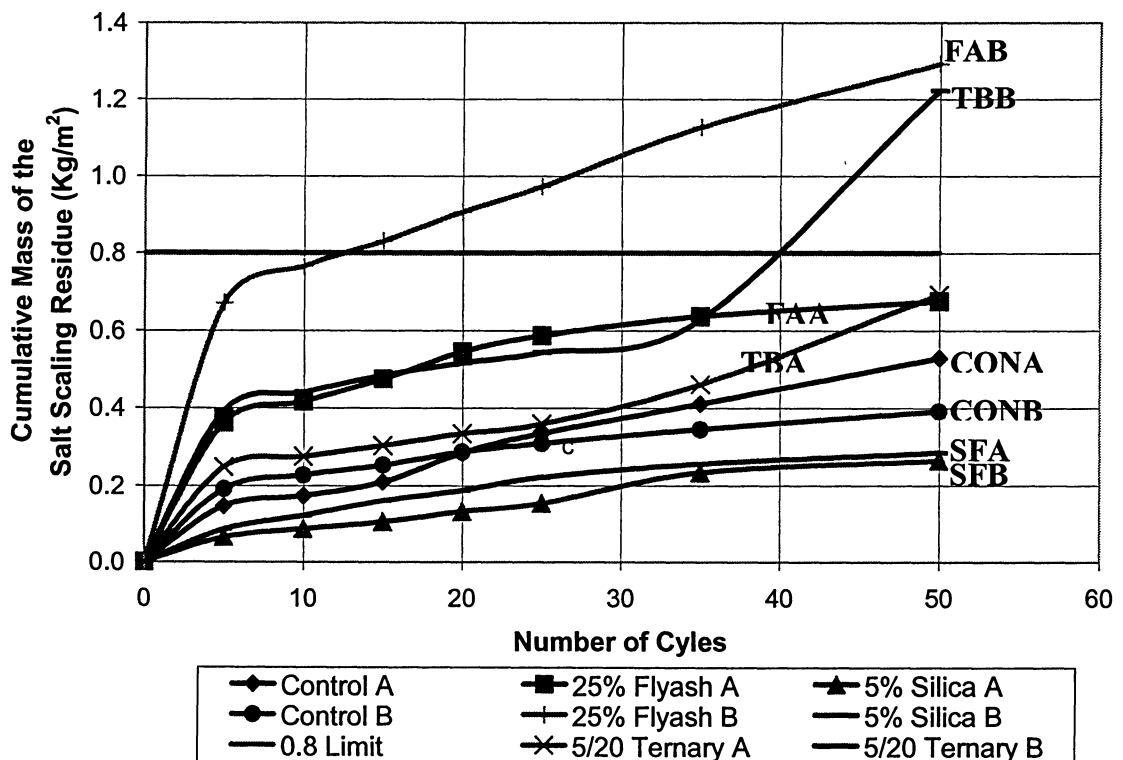


Figure 44: Salt scaling results of the 50 cycles of freezing and thawing on air entrained samples.

In order to understand the effects of finishing and bleeding on the concrete surface, the rates of deterioration for the non-air entrained samples were determined and the graph can be seen in Figure 45. Unfortunately the 50th cycle resulted in irregular behaviour due to the repair and was removed. As displayed on the graph, the initial deterioration rates varied between the samples. This indicates the effect of bleeding and finishing on each sample. It is also known that SCM's generally increase the setting time of the concrete, especially fly ash. The difference might also be an effect of the setting times of each sample, where the fly ash has the greatest setting time thus, and more time to bleed. Since silica and the control mix set faster, the affects of bleeding would be reduced.

Furthermore, with extrapolation it would appear that the final rates seem to almost converge to the same point. This would indicate that as soon as the deterioration consumes the surface of the concrete slabs, the deterioration rate continues equally through all the samples. This means that the surface of the concrete has a reduced salt scaling resistance and after the surface, the samples almost have the same resistance to scaling. This reduced scaling resistance would confirm that the finishing and bleeding of the samples have a great effect on the salt scaling resistance of concrete. Similar conclusion can be observed with the air-entrained rate of deterioration seen in Figure 46 with the exception of ternary B sample, which had a problem with the 50th cycle.

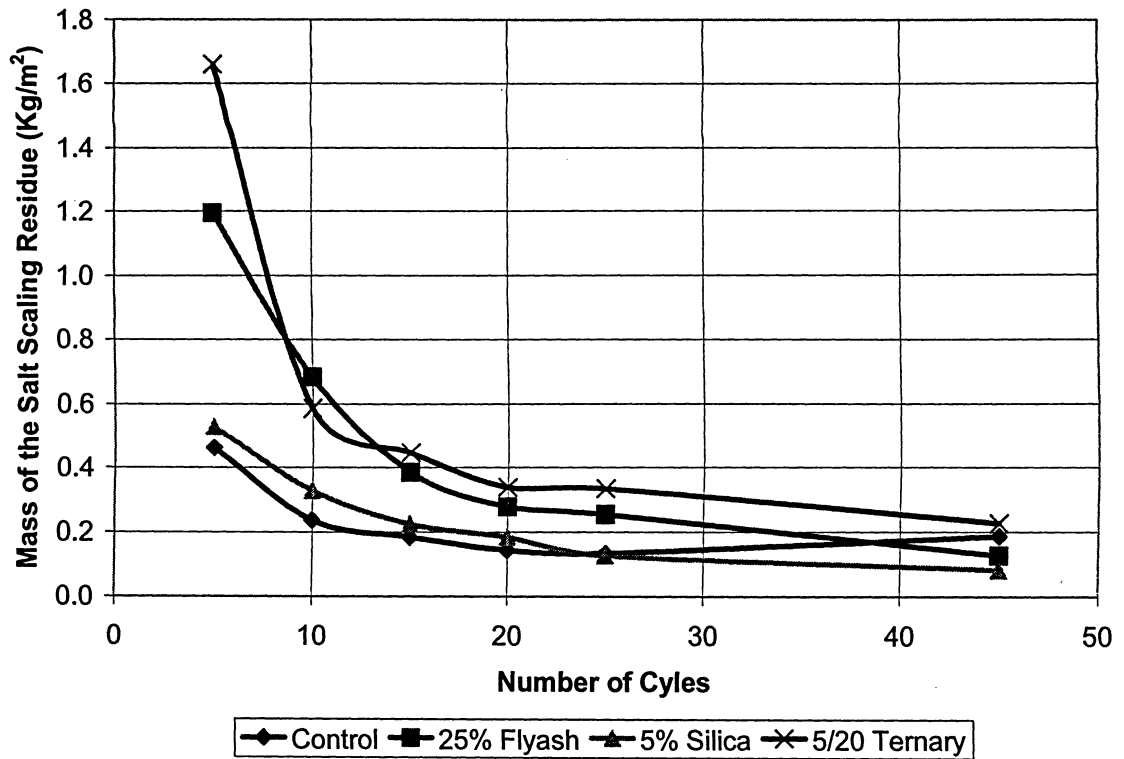


Figure 45: Salt scaling results of non-air entrained rate of deterioration per 5 days.

4.2.3 Effects of RCA on salt scaling:

The results obtained from the non-air and air entrained samples could be interpreted that RCA produced from non-air entrained concrete would be susceptible to salt scaling and RCA produced from air entrained concrete would not be as susceptible. Figure 47 displays the results of concrete made with RCA aggregates and limestone aggregates containing air entrainment. Ternary blends of 5/30 SF/slag was tested as this is the blend that showed promising results in terms of counteracting ASR.

Unfortunately 20 cycles have been tested to date; however, the results are being updated and some conclusions can be assumed based on the trend observed in Figures 43 & 44. After the 15th cycle almost all the samples seen in Figures 43 & 44 tend to have a constant salt scaling residue accumulation. Therefore, it can be deduced that the limestone samples and the RCA control sample will most probably remain under the MTO limit. It is also clear that the use of RCA aggregates increases salt scaling deterioration when compared to the limestone. This could be due to the increased paste fraction of concrete containing RCA. In addition, the ternary blend consisting of 30% slag and 5% silica has a reduced resistance to salt scaling compared to the control sample. It would seem that using slag has a similar response to salt scaling, as does fly ash. Since the 50th cycle has not completed it is difficult to determine if the ternary blend with RCA will pass the MTO limit of 0.8 kg/m²

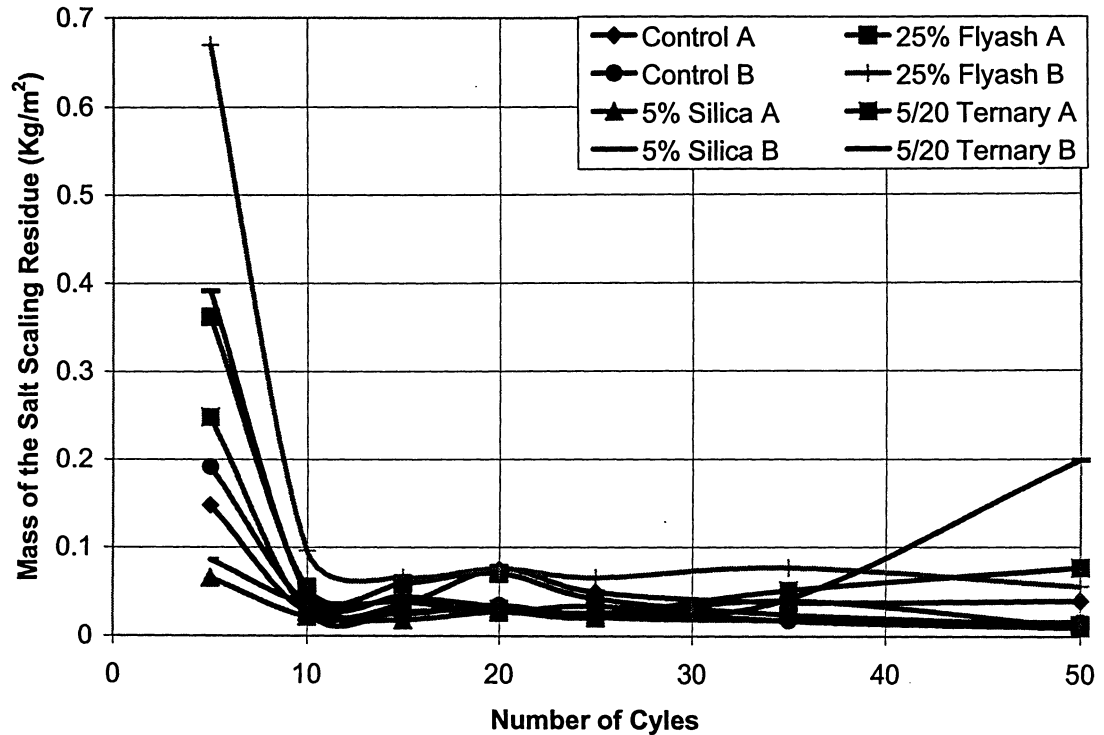


Figure 46: Salt Scaling rate of deterioration per 5 days.

The results showed that concrete containing RCA could be successfully produced with high resistance to salt scaling deterioration; however, when using recycled concrete for demolition debris, care should be taken to reduce the non-air entrained aggregates from combining with the air-entrained aggregates. It is the opinion of the author that recycled aggregates with previous exposure to de-icer salts could introduce a greater concentration of salt ions making it less susceptible to salt scaling as determined by Marchand et al., 1999.

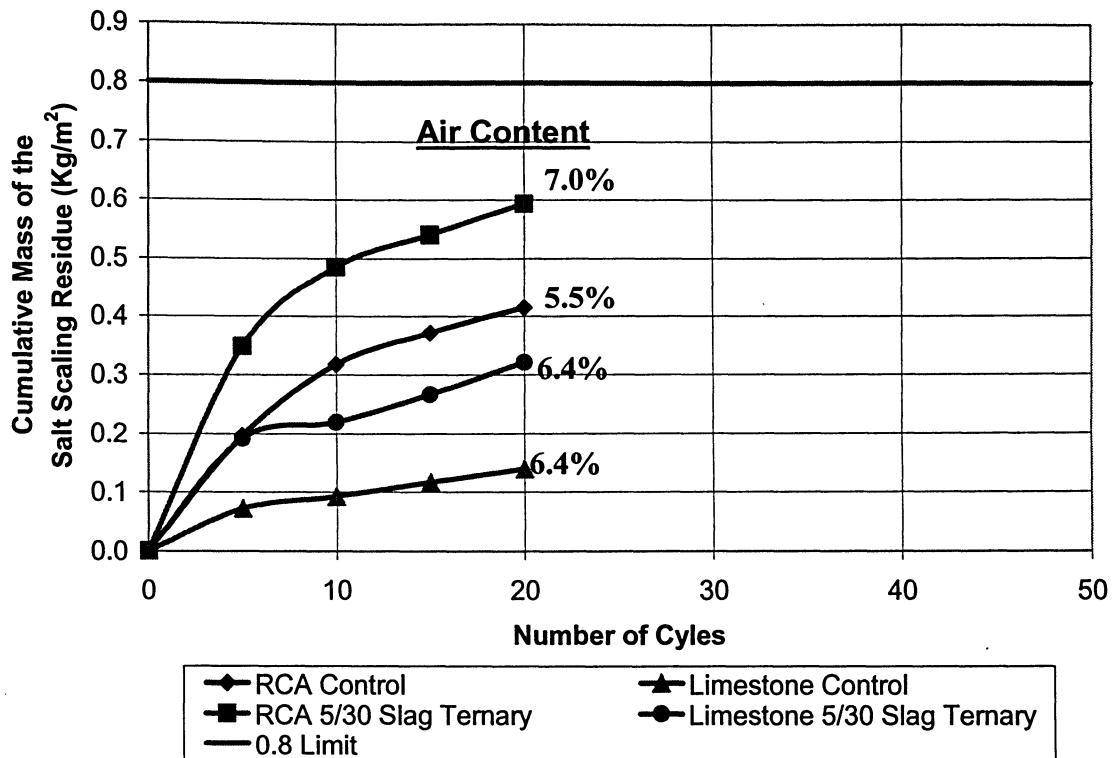


Figure 47: Salt Scaling results comparing RCA with Spratt aggregates.

4.3 Resistance of concrete containing RCA to Freezing and Thawing

A problem arose when testing the samples for freezing and thawing. Some of the samples in the freezer were not becoming frozen and were over heated; therefore, these specimens were not included in the calculations. The results of the freeze and thaw test are shown in Appendix D. The specimens not included were RCA control B and Limestone Control A and B. Regardless of the exclusion of these specimens, some fluctuations were observed. An average of the results can be viewed in Figure 48. It is clear from the graph that a lot of fluctuations occur during testing. The RCA samples seem to have a reduced resistance to freeze thaw

deterioration due to a reduced strength observed with these aggregates. It is also possible that the inclusion of recycled aggregates with previous exposure to de-icer salts can cause further damage in this case. Since the samples are not introduced to salt ions, it is possible that the salt ions present in the recycled aggregates could cause further deterioration through a process similar to salt scaling. This is especially true with non-air entrained concrete. Further testing in this area would be necessary in order to have conclusive results.

It can be seen that the samples containing 30% slag and 5% silica have a reduced durability factor as seen in Figure 49 compared to its control counter part. RCA absorbed a greater amount of water than limestone aggregates and this extra water could cause an increased amount of deterioration when exposed to colder temperatures. Since slag concrete requires more time to cure, it is possible that the two-week curing period required by the ASTM standard was not sufficient and different results could be obtained if longer curing is adopted.

The differences detected in the RCA aggregates compared to the limestone aggregates could have a great impact on the results of salt scaling. Since the RCA aggregates are more freeze thaw susceptible, it would be deduced that the RCA samples would also have an increased deterioration in the salt scaling samples.

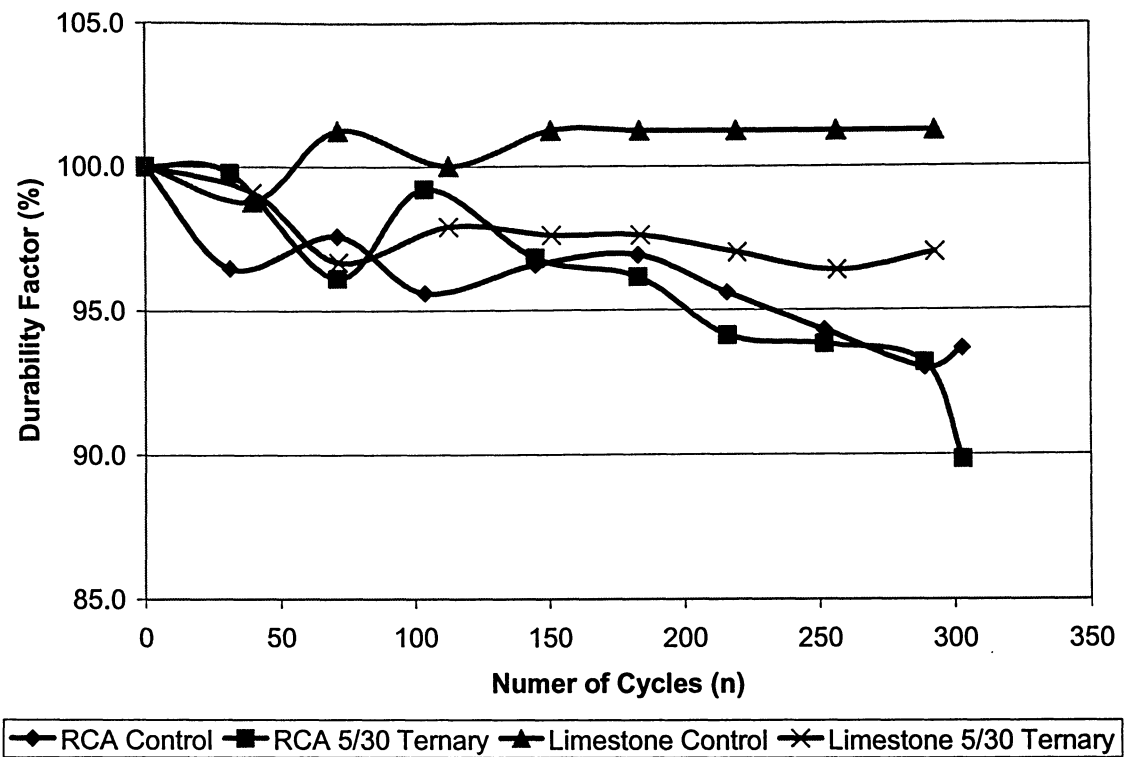


Figure 48: Freeze thaw results comparing RCA with Spratt aggregates.

It should be noted that when interpreting the freezing and thawing results, one should bear in mind that the sample did not undergo a proper curing period. If the samples were cured in water and allowed to dry out, the results may have been different, especially the samples containing ternary blends.

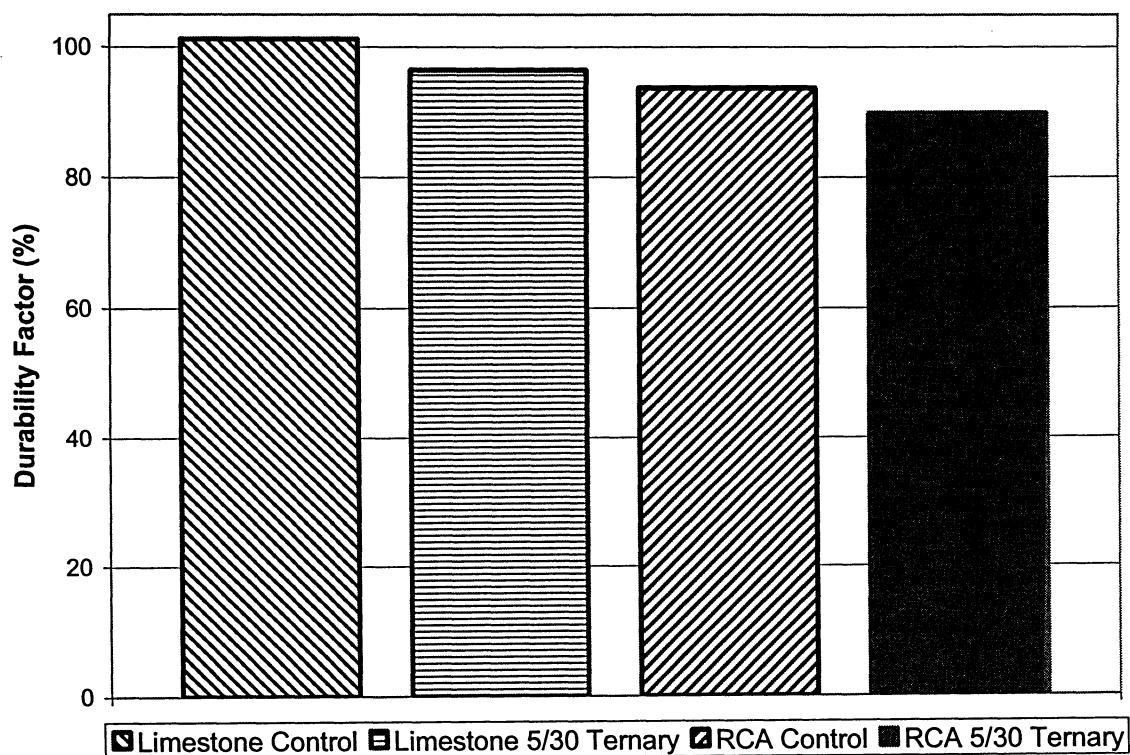


Figure 49: Freeze thaw durability factor comparing RCA with limestone aggregates.

Chapter 5

Summary, Conclusions, and Recommendations

5.1 Summary

Using recycled aggregates has beneficial effects on reducing landfill accumulation and reducing the consumption of natural and non-renewable resources. However, the properties of the recycled aggregates present a difficult task in using the aggregates in concrete structures. The purpose of this study was:

- I. To investigate the reactivity of recycled concrete aggregate produced from concrete that was suffering from ASR.
- II. To investigate whether or not preventative measures used for controlling expansion in concrete containing raw reactive aggregate are effective in controlling the expansion in new concrete containing ASR recycled aggregate
- III. To investigate some durability issues of concrete containing RCA with emphasis on salt scaling & freezing and thawing. Other durability issues are covered though other programmes at Ryerson University.

5.2 Conclusions

The testing of recycled aggregates in ASR mortar bars and freeze thaw resistance has completed and results are available; however, the results of ASR prisms and salt scaling are continuously being updated. This should be kept in mind when reviewing the results and conclusion. There results obtained thus far have concluded the following:

- 1) Whereas Spratt required approximately 0.70% cement alkalinity to reduce expansion to below ASTM and CSA limit, RCA required around 0.35%; therefore, very low alkali cement is required to reduce expansion to acceptable limits. Such Portland cement is hard to find in North America. Accordingly, other preventative measures should be considered.
- 2) The results obtained from the control prism samples indicated that the RCA contain a percentage of alkalis that increase the total alkalinity of concrete containing RCA.
- 3) Higher levels of SCMs were required to reduce the expansion to below the ASTM and CSA limit of 0.04% for RCA containing Spratt compared to the levels required for Spratt aggregate.

- 4) The increase in alkali content and calcium in fly ash composition increases the expansion.
- 5) 5% silica fume seems to perform better in concrete containing RCA with ASR rather than with concrete containing Spratt aggregate.
- 6) Slag with similar cement replacement as high calcium fly ash appears to be more effective in reducing expansion.
- 7) Ternary blends of SF/FA or SF/Slag were advantageous compared to binary blends.
- 8) Concrete containing RCA has a reduced resistance to both salt scaling and freezing and thawing compared to concrete with natural aggregates.
- 9) Finishing and bleeding have displayed an increasing effect on the surface scaling deterioration of concrete.
- 10) Concrete containing ternary blends of silica fume and slag has a reduced resistance to freezing and thawing, and salt scaling. However, concrete with acceptable resistance could be obtained.

- 11) Fly ash in both air entrained and non-air entrained concrete results in a decrease in salt scaling resistance.
- 12) Silica fume in non-air entrained concrete does not seem to significantly affect the scaling resistance of concrete; however, in air-entrained concrete silica fume improves the scaling resistance of ordinary and ternary concrete mixes.

5.3 Recommendation for Further Studies

According to the results and conclusions obtained in this study the following recommendations for further studies can be considered:

1. **Use of Lithium Salts:** High quantities of SCMs are required in order to reduce expansion below the 0.04% limit; however, these levels of SCM are not feasible for many applications. Therefore, the use of lithium salts to counteract ASR should be tested.
2. **Use of metakaolin as cement replacement:** Since the use of metakaolin has proven to reduce the alkalis in the pore solution, then the addition of metakaolin could prove to have a greater effect in controlling ASR expansion compared to the other SCMs used in this study.

3. **Higher levels of ternary blends:** Ternary blends have proven to be superior to binary blends; however, some ternary blends could not reduce expansion below the 0.04% limit at their current cement replacement level. Therefore, higher levels of ternary blends could reduce expansion below 0.04% limit.
4. **Blocks at exposure sites:** The results obtained in this study have been conducted in controlled environments. Since Rogers et al. (2000) has shown that the results obtained in the laboratory can vary to real life structures; it is recommended that concrete blocks made of RCA containing ASR be constructed and evaluated.

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Appendix A: Expansion of Concrete Prisms

RCA Expansion %											
CONTROL											
Bucket #	#9	#47	#1	#6	#2	#13	#22	#72	#15	#66	#65
Type	0.69 Con	0.69 Con	60:40 Con	0.70 Con	0.69 Con	0.69 Con	0.69 Con	0.69 Con	0.69 Con	60:40 Con	60:40 Con
Na ₂ O _e	2.31 (kg/m ³)	3.36 (kg/m ³)	5.25 (kg/m ³)	5.25 (kg/m ³)	5.25 (kg/m ³)	5.25 (kg/m ³)	5.25 (kg/m ³)	5.25 (kg/m ³)	6.30 (kg/m ³)	5.25 (kg/m ³)	5.25 (kg/m ³)
	0.55 Alkali	0.80 Alkali	1.25 Alkali	1.25 Alkali	1.25 Alkali	1.25 Alkali	1.25 Alkali	1.25 Alkali	1.5 Alkali	1.25 Alkali	1.25 Alkali
Weeks							Ottawa Sand	WASHED		FINE AGG.	N.A.
0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
1	0.009	0.010	0.018	0.021	0.007	0.008	0.016	0.012	0.010	0.010	0.005
2	0.020	0.018	0.029	0.025	0.011	0.018	0.026	0.028	0.015	0.015	0.007
4	0.025	0.032	0.036	0.035	0.020	0.031	0.033	0.029	0.025	0.018	0.009
8	0.036	0.041	0.066	0.067	0.037	0.058	0.054	0.051	0.053	0.030	0.015
13	0.049	0.059	0.101	0.121	0.085	0.102	0.088	0.073	0.097	0.037	0.020
18	0.056	0.080	0.123	0.157	0.117	0.127	0.110		0.125	0.042	0.025
26	0.063	0.103	0.144	0.192	0.147	0.159			0.164	0.051	0.024
39	0.077	0.119	0.160	0.220	0.185	0.202			0.207	0.045	0.028
52	0.091	0.115	0.172	0.238	0.199	0.221			0.228		

RCA Expansion %											
	SILICA			FLY ASH Type F, HA				FLY ASH Type F, LA			
Bucket #	#3	#8	#14	#10	#16	#17	#68	#48	#49	#73	#50
Type	0.69 RCA	0.69 RCA	0.69 RCA	Type F, HA	Type F, HA	Type F, HA	Type F, HA	Type F, LA	Type F, LA	Type F, LA	Type F, LA
Na ₂ O _e	4.99 (kg/m ³)	4.83 (kg/m ³)	4.73 (kg/m ³)	4.46 (kg/m ³)	4.20 (kg/m ³)	3.94 (kg/m ³)	3.41 (kg/m ³)	4.20 (kg/m ³)	3.94 (kg/m ³)	3.94 (kg/m ³)	3.68 (kg/m ³)
	5% SF	8% SF	10% SF	15% FA	20% FA	25% FA	35% FA	20% FA	25% FA	25% FA	30% FA
Weeks										WASHED	
0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
1	0.003	-0.001	-0.004	0.005	0.008	0.012	0.014	0.008	0.008	0.010	0.007
2	0.006	-0.002	-0.001	0.006	0.009	0.014	0.012	0.012	0.010	0.022	0.016
4	0.013	0.006	0.002	0.018	0.021	0.018	0.011	0.019	0.017	0.018	0.018
8	0.023	0.014	0.009	0.037	0.030	0.031	0.011	0.023	0.017	0.024	0.021
13	0.034	0.022	0.012	0.050	0.043	0.043	0.022	0.029	0.021	0.029	0.028
18	0.045	0.028	0.017	0.061	0.051	0.049		0.040	0.030		0.032
26	0.059	0.032	0.027	0.079	0.072	0.066		0.053	0.038		0.034
39	0.084	0.047	0.033	0.111	0.092	0.081		0.064	0.042		0.043
52	0.103	0.059	0.041	0.137	0.115	0.094		0.060	0.042		0.044

RCA Expansion %											
Bucket #	FLY ASH CI, LA			FLY ASH Type CH LA			SLAG				
	#45	#46		#18	#19	#20	#35	#32	#36		
Type	Type CI, LA	Type CI, LA		Type CH	Type CH	Type CH	0.69 RCA	0.69 RCA	0.69 RCA		
Na ₂ O _e	4.20	3.68		3.94	3.68	2.63	3.94	3.68	2.63		
	(kg/m ³)	(kg/m ³)		(kg/m ³)	(kg/m ³)	(kg/m ³)	(kg/m ³)	(kg/m ³)	(kg/m ³)		
	20% FA	30% FA		25% FA	30% FA	50% FA	25% Slag	30% Slag	50% Slag		
Weeks											
0	0.000	0.000		0.000	0.000	0.000	0.000	0.000	0.000		
1	0.012	0.005		0.005	0.005	0.001	0.008	0.011	0.010		
2	0.015	0.005		0.000	0.000	0.021	0.019	0.019	0.017		
4	0.016	0.005		0.021	0.022	0.024	0.032	0.027	0.020		
8	0.026	0.013		0.042	0.034	0.027	0.044	0.035	0.024		
13	0.037	0.017		0.054	0.051	0.036	0.059	0.047	0.031		
18	0.049	0.027		0.066	0.063	0.043	0.071	0.048	0.027		
26	0.058	0.031		0.088	0.082	0.054	0.090	0.068			
39	0.071	0.040		0.108	0.104	0.062	0.103	0.081			
52	0.065	0.029		0.128	0.129	0.074	0.115	0.090			

RCA Expansion %									
Ternary Blends									
Bucket #	FLY ASH F, HA			FLY ASH F, LA			FLY ASH CI, LA		
	#33	#34	Type F, HA	#51	Type F, LA	Type F, LA	#53	#55	#69
Type	Type F, HA	Type F, HA	Type F, HA	Type F, LA	Type F, LA	Type F, LA	Type CI	Type CI	Type CI
Na ₂ O _e	3.94 (kg/m ³)	3.68 (kg/m ³)	3.68 (kg/m ³)	4.20 (kg/m ³)	3.94 (kg/m ³)	3.94 (kg/m ³)	3.94 (kg/m ³)	3.68 (kg/m ³)	3.68 (kg/m ³)
	5% Silica	5% Silica	5% Silica	5% Silica	5% Silica	5% Silica	5% Silica	5% Silica	5% Silica
	20% FA	25% FA	25% FA	15% FA	20% FA	20% FA	20% FA	25% FA	25% FA
Weeks									
0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
1	0.008	0.012	0.012	0.003	0.010	0.010	0.003	0.000	-0.002
2	0.011	0.011	0.011	0.010	0.012	0.012	0.004	0.002	-0.004
4	0.014	0.016	0.016	0.012	0.009	0.009	0.006	0.005	-0.002
8	0.014	0.019	0.019	0.014	0.012	0.012	0.009	0.014	-0.001
13	0.026	0.026	0.026	0.021	0.018	0.018	0.013	0.014	0.011
18	0.029	0.027	0.027	0.026	0.023	0.023	0.018	0.015	
26	0.043	0.046	0.046	0.027	0.022	0.022	0.018	0.022	
39	0.051	0.051	0.051	0.037	0.032	0.032	0.029	0.017	
52	0.052	0.059	0.059	0.039	0.033	0.033	0.028		

RCA Expansion %									
Ternary Blends									
Bucket # Type	FLY ASH Type CH LA				SLAG				
	#29 Type CH	#30 Type CH	#31 Type CH		#54	#56	#70	#71	
Na ₂ O _e	3.94 (kg/m ³)	3.68 (kg/m ³)	3.41 (kg/m ³)		3.68 (kg/m ³)	3.41 (kg/m ³)	3.41 (kg/m ³)	2.89 (kg/m ³)	
	5% Silica	5% Silica	5% Silica		5% Silica	5% Silica	5% Silica	5% Silica	
	20% FA	25% FA	30% FA		25% Slag	30% Slag	30% Slag	40% Slag	
Weeks									
0	0.000	0.000	0.000		0.000	0.000	0.000	0.000	
1	0.004	0.001	-0.002		0.010	0.007	0.013	0.015	
2	0.008	0.003	0.000		0.017	0.006	0.015	0.019	
4	0.010	0.007	0.007		0.018	0.013	0.017	0.025	
8	0.012	0.010	0.008		0.025	0.020	0.012	0.021	
13	0.019	0.015	0.012		0.034	0.022	0.020	0.023	
18	0.027	0.023	0.016		0.040	0.024			
26	0.039	0.032	0.023		0.042	0.030			
39	0.053	0.041	0.029		0.053	0.028			
52	0.047	0.041	0.032		0.053				

JOBE Expansion %									
	TAYLORVILLE				DEELY	COMANCHE	ST. LAWRENCE		
	FLYASH			BLEND	BLEND	BLEND	SILICA		
Bucket #	#41	#42	#43	#44	#57	#58	#59	#60	
Type	Type F, HA	Type F, HA	Type F, HA	Type F, HA					
Na ₂ O _e	4.46 (kg/m ³)	4.20 (kg/m ³)	3.94 (kg/m ³)	3.94 (kg/m ³)	3.78 (kg/m ³)	3.68 (kg/m ³)	4.88 (kg/m ³)	4.73 (kg/m ³)	
	15% FA	20% FA	25% FA	5% Silica	3% Silica	5% Silica	7% Silica	10% Silica	
				20% FA	25% FA	25% FA			
Weeks									
0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	
1	0.002	0.001	0.002	-0.003	-0.001	-0.008	0.000	-0.001	
2	0.012	0.009	0.004	-0.003	-0.001	-0.006	0.005	0.001	
4	0.039	0.011	0.007	-0.001	0.006	0.000	0.014	0.009	
8	0.124	0.044	0.011	0.004	0.011	0.001	0.020	0.014	
13	0.205	0.092	0.029	0.006	0.014	0.006	0.025	0.018	
18	0.272	0.143	0.056	0.011	0.016	0.008	0.038	0.020	
26	0.337	0.197	0.085	0.011	0.026	0.015	0.202	0.022	
39	0.448	0.280	0.140	0.023	0.015	0.020	0.359	0.013	
52	0.446	0.314	0.145	0.020					

SPRATT Expansion %											
Bucket #	CONTROL			FLYASH					BLEND		
	#67	#74		#61	#62	#63	#23	#24	#75	#64	#28
Type	60:40 Con	60:40 Con		Type F, HA	Type F, HA	Type F, HA	Type F, HA	Type F, HA	Type F, HA	Type F, HA	Type F, HA
Na ₂ O _e	5.25 (kg/m ³)	5.25 (kg/m ³)		4.46 (kg/m ³)	4.20 (kg/m ³)	3.94 (kg/m ³)	4.46 (kg/m ³)	3.94 (kg/m ³)	3.94 (kg/m ³)	3.94 (kg/m ³)	3.68 (kg/m ³)
	1.25 Alkali	1.25 Alkali		15% FA	20% FA	25% FA	15% FA	25% FA	5% Silica	5% Silica	5% Silica
									20% FA	20% FA	25% FA
Weeks											
0	0.000	0.000		0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
1	0.005	0.005		-0.002	-0.005	0.004	0.002	0.002	-0.008	-0.004	-0.002
2	0.007	0.015		-0.001	-0.004	0.003	0.007	0.006	0.004	-0.001	0.000
4	0.014	0.009		0.006	0.002	0.008	0.008	0.002	-0.003	-0.001	-0.001
8	0.054	0.005		0.011	0.006	0.012	0.016	0.011	-0.008	0.003	0.002
13	0.103	0.027		0.020	0.012	0.019	0.023	0.012	-0.004	0.008	0.007
18	0.136	0.080		0.036	0.019	0.025	0.032	0.019	-0.005	0.011	0.011
26	0.163			0.049	0.025	0.038	0.047	0.027		0.017	0.010
39	0.173			0.063	0.033	0.035	0.041	0.019			0.013
52											

Appendix B: Expansion of Concrete Mortar Bars

RCA Expansion %												
CONTROL												
Time	Primary	Secondary	Secondary	0.89 Alkali	0.65 Alkali	0.5 Alkali	0.2 Alkali	NA	NA #2	Ottawa		
Days	RCA	RCA	RCA	Secondary	Secondary	Secondary	Secondary			Sand		
0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		
3	0.047	0.091	0.093				0.020		0.031			
4				0.043	0.020	0.007		0.027				
7	0.090	0.189					0.017	0.090		0.007		
9			0.236	0.119	0.051	0.023			0.170			
10	0.120	0.243						0.155				
11			0.269	0.154	0.067	0.038			0.205	0.017		
14	0.147	0.289	0.297	0.190	0.081	0.040	0.030	0.202	0.238	0.017		
21			0.341	0.271	0.123	0.058		0.260	0.298			
28			0.371	0.337	0.159	0.068		0.300	0.336	0.023		

RCA Expansion %												
Time Days	SILICA			SLAG			FLYASH Type F LA					
	5% Silica RCA	8% Silica RCA	10% Silica RCA	25% Slag RCA	30% Slag RCA	50% Slag RCA	30% FA RCA	25% FA RCA	20% FA RCA			
0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000			
3				0.043	0.035	0.031	0.024	0.024	0.029			
4	0.074	0.050	0.038									
7	0.122	0.089	0.063	0.111	0.095	0.075	0.050	0.053	0.079			
9												
10				0.155	0.131	0.102	0.075	0.079	0.125			
11	0.175	0.143	0.109									
14	0.206	0.183	0.138	0.211	0.179	0.143	0.110	0.114	0.169			
21	0.283	0.279	0.218	0.273	0.240	0.201						
28	0.342	0.367	0.298	0.319	0.282	0.250	0.204	0.210	0.307			

RCA Expansion %												
	FLYASH Type F HA			FLYASH Type CI LA			FLYASH Type CH LA					
Time	20% FA	25% FA		20% FA	30% FA	15% FA	20% FA	25% FA	30% FA	50% FA		
Days	RCA	RCA		RCA	RCA	RCA	RCA	RCA	RCA	RCA		
0	0.000	0.000		0.000	0.000	0.000	0.000	0.000	0.000	0.000		
3						0.065	0.063	0.052				
4	0.141	0.113		0.058	0.042							
7	0.237	0.197		0.137	0.096	0.181	0.152	0.147	0.204	0.124		
9												
10						0.251	0.210	0.206				
11	0.294	0.248		0.211	0.152				0.286	0.182		
14	0.338	0.289		0.251	0.187	0.308	0.250	0.250	0.334	0.210		
21	0.413	0.367		0.343	0.271							
28	0.499	0.443		0.428	0.360	0.386	0.315	0.323	0.513	0.348		

RCA Expansion %									
Ternary Blends									
	FLYASH Type F HA		FLYASH Type CI LA		FLYASH Type CH LA		Slag		
Time	5/20 FA	5/25 FA	5/20 FA	5/25 FA	5/20 FA	5/30 FA	5/30 FA	5/40 FA	
Days	RCA	RCA	RCA	RCA	RCA	RCA	RCA	RCA	
0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	
3	0.026	0.026							
4			0.035	0.023	0.039	0.033	0.088	0.071	
7			0.067	0.051					
8	0.091	0.078			0.098	0.071	0.131	0.101	
10	0.146	0.133							
11			0.129	0.097					
14	0.195	0.181	0.156	0.127	0.185	0.131	0.220	0.186	
21	0.297	0.276	0.241	0.197					
28	0.378	0.350	0.339	0.279					

Appendix C: Salt Scaling Residue

Non-Air Entrained Chart Values				
Area (m ²)	0.0529	0.0529	0.0529	0.0529
Cycles	Control (Kg/m ²)	Fly ash (Kg/m ²)	Silica (Kg/m ²)	Ternary blend (Kg/m ²)
0	0	0	0	0
5	0.46	1.19	0.53	1.66
10	0.70	1.88	0.85	2.25
15	0.88	2.26	1.08	2.69
20	1.02	2.54	1.26	3.03
25	1.16	2.79	1.39	3.36
30	1.37	3.07	1.56	3.89
45	1.92	3.45	1.81	4.57
50	2.75	3.91	2.22	5.53
Repaired Area (m ²)	0.0380	0.0380	0.0380	0.0380

Air Entrained Chart Values				
Area (m ²)	0.0484	0.0484	0.0484	0.0462
Cycles	Control A (Kg/m ²)	Control B (Kg/m ²)	Fly ash A (Kg/m ²)	Fly ash B (Kg/m ²)
0	0	0	0	0
5	0.15	0.19	0.36	0.67
10	0.17	0.23	0.42	0.77
15	0.21	0.25	0.48	0.83
20	0.28	0.29	0.55	0.91
25	0.33	0.31	0.59	0.97
35	0.41	0.34	0.64	1.13
50	0.53	0.39	0.67	1.29

Air Entrained Chart Values				
Area (m ²)	0.0484	0.0462	0.0462	0.0462
Cycles	Silica A (Kg/m ²)	Silica B (Kg/m ²)	Ternary blend A (Kg/m ²)	Ternary blend B (Kg/m ²)
0	0	0	0	0
5	0.07	0.09	0.25	0.39
10	0.09	0.12	0.28	0.44
15	0.11	0.16	0.30	0.48
20	0.13	0.19	0.33	0.52
25	0.15	0.22	0.36	0.54
35	0.23	0.26	0.46	0.63
50	0.26	0.28	0.69	1.22

RCA Air Entrained Chart Values				
Area (m ²)	0.0506	0.0506	0.0506	Average
Cycles	Control A (Kg/m ²)	Control B (Kg/m ²)	Control C (Kg/m ²)	
0	0.000	0.000	0.000	0.000
5	0.209	0.199	0.181	0.196
10	0.349	0.326	0.276	0.317
15	0.409	0.384	0.321	0.371
20	0.461	0.426	0.356	0.415
25				
30				
35				
40				
45				
50				

RCA Air Entrained Chart Values				
Area (m ²)	0.0506	0.0529	0.0506	Average
Cycles	5/30 Slag Ternary A (Kg/m ²)	5/30 Slag Ternary B (Kg/m ²)	5/30 Slag Ternary C (Kg/m ²)	
0	0.000	0.000	0.000	0.000
5	0.355	0.310	0.381	0.348
10	0.417	0.487	0.550	0.485
15	0.449	0.518	0.652	0.540
20	0.471	0.580	0.728	0.593
25				
30				
35				
40				
45				
50				

Limestone Air Entrained Chart Values				
Area (m ²)	0.0484	0.0484	0.0484	Average
Cycles	Control A (Kg/m ²)	Control B (Kg/m ²)	Control C (Kg/m ²)	
0	0.000	0.000	0.000	0.000
5	0.102	0.035	0.078	0.072
10	0.123	0.050	0.103	0.092
15	0.151	0.069	0.127	0.116
20	0.173	0.085	0.157	0.138
25				
30				
35				
40				
45				
50				

Limestone Air Entrained Chart Values				
Area (m²)	0.0484	0.0484	0.0484	Average
Cycles	5/30 Slag Ternary A (Kg/m²)	5/30 Slag Ternary B (Kg/m²)	5/30 Slag Ternary C (Kg/m²)	
0	0.000	0.000	0.000	0.000
5	0.162	0.166	0.247	0.191
10	0.182	0.184	0.291	0.219
15	0.211	0.277	0.311	0.266
20	0.261	0.363	0.337	0.321
25				
30				
35				
40				
45				
50				

Appendix D: Freezing and Thawing Durability Factor

RCA Control								
Specimen A								
Date	Cycles	Transverse		Longitudinal	Length		L _C	Weight
		(Khz)	P _C (%)	(Khz)	(mm)	(mm)	%	(g)
07-Jul-06	0	2.98	100.0	6.48	3.352	3.350	0.000	3808.7
14-Jul-06	31.5	2.96	98.2	6.42	3.342	3.340	-0.004	3820.0
21-Jul-06	71.5	2.96	98.4	6.46	3.372	3.370	0.008	3824.0
27-Jul-06	104	2.95	97.8	6.46	3.354	3.354	0.002	3827.5
04-Aug-06	145	2.95	97.8	6.45	3.276	3.274	-0.030	3826.1
11-Aug-06	183	2.93	96.5	6.43	3.390	3.374	0.010	3826.0
17-Aug-06	216	2.93	96.5	6.39	3.266	3.262	-0.035	3825.9
24-Aug-06	252	2.89	93.8	6.35	3.438	3.436	0.034	3815.6
31-Aug-06	289	2.85	91.3	6.39	3.464	3.458	0.043	3907.6
3-Sep-06	303	2.87	92.5	-	3.476	3.474	0.050	3809.1

RCA Control								
Specimen B								
Date	Cycles	Transverse		Longitudinal	Length		L _C	Weight
		(Khz)	P _C (%)	(Khz)	(mm)	(mm)	%	(g)
07-Jul-06	0	2.93	100.0	6.44	2.104	2.086	0.000	3769.4
14-Jul-06	31.5	2.93	100.0	6.41	2.090	2.074	-0.005	3774.5
21-Jul-06	71.5	2.91	98.6	6.43	2.130	2.122	0.014	3772.8
27-Jul-06	104	2.92	99.3	6.43	2.174	2.152	0.026	3781.6
04-Aug-06	145	2.97	102.7	6.47	2.108	2.100	0.006	3785.0
11-Aug-06	183	2.97	102.7	6.52	2.248	2.212	0.050	3986.2
17-Aug-06	216	2.95	101.4	6.48	2.308	2.292	0.082	3787.1
24-Aug-06	252	2.95	101.4	6.52	2.460	2.440	0.142	3792.7
31-Aug-06	289	2.97	102.7	6.52	2.480	2.470	0.154	3788.4
3-Sep-06	303	2.96	102.1	6.54	2.506	2.494	0.163	3789.6

RCA Control								
Specimen C								
Date	Cycles	Transverse		Longitudinal	Length		L_C	Weight
		(Khz)	P _C (%)		(mm)	(mm)		
07-Jul-06	0	2.99	100.0	6.43	4.384	4.372	0.000	3731.8
14-Jul-06	31.5	2.91	94.7	6.43	4.358	4.350	-0.009	3741.7
21-Jul-06	71.5	2.94	96.7	6.41	4.416	4.414	0.017	3745.3
27-Jul-06	104	2.89	93.4	6.43	4.384	4.370	-0.001	3750.0
04-Aug-06	145	2.92	95.4	6.48	4.338	4.332	-0.016	3752.0
11-Aug-06	183	2.95	97.3	6.50	4.434	4.428	0.022	3747.2
17-Aug-06	216	2.91	94.7	6.48	4.462	4.452	0.032	3746.2
24-Aug-06	252	2.91	94.7	-	4.444	4.438	0.026	3736.2
31-Aug-06	289	2.91	94.7	-	4.482	4.472	0.040	3711.4
3-Sep-06	303	2.91	94.7	-	4.484	4.480	0.043	3708.1

5/30 Slag RCA								
Specimen A								
Date	Cycles	Transverse		Longitudinal	Length		L_C	Weight
		(Khz)	P _C (%)		(mm)	(mm)		
07-Jul-06	0	2.89	100.0	6.28	3.654	3.648	0.000	3694.6
14-Jul-06	31.5	2.88	99.1	6.36	3.634	3.632	-0.006	3704.4
21-Jul-06	71.5	2.81	94.5	6.33	3.644	3.634	-0.006	3692.5
27-Jul-06	104	2.86	97.9	6.35	3.672	3.664	0.006	3695.2
04-Aug-06	145	2.87	98.6	6.36	3.586	3.580	-0.027	3694.7
11-Aug-06	183	2.89	100.0	6.41	3.656	3.634	-0.006	3690.1
17-Aug-06	216	2.85	97.3	6.39	3.556	3.552	-0.038	3685.7
24-Aug-06	252	2.87	98.6	6.41	3.692	3.690	0.017	3677.1
31-Aug-06	289	2.87	98.6	6.41	3.754	3.750	0.041	3665.3
3-Sep-06	303	2.81	94.5	6.36	3.760	3.758	0.044	3663.8

5/30 Slag RCA								
Specimen B								
Date	Cycles	Transverse		Longitudinal	Length		L_C	Weight
		(Khz)	P _C (%)		(mm)	(mm)		
07-Jul-06	0	2.90	100.0	6.28	3.648	3.644	0.000	3687.3
14-Jul-06	31.5	2.84	96.4	6.20	3.616	3.610	-0.014	3699.1
21-Jul-06	71.5	2.83	95.4	6.18	3.624	3.624	-0.008	3689.3
27-Jul-06	104	2.81	94.1	6.10	3.626	3.614	-0.012	3676.0
04-Aug-06	145	2.77	91.4	6.00	3.660	3.652	0.003	3665.6
11-Aug-06	183	2.70	86.9	0.53	3.676	3.654	0.004	3656.8
17-Aug-06	216	2.60	80.6	-	3.718	3.712	0.027	3637.7
24-Aug-06	252	2.38	67.5	-	3.768	3.766	0.049	3617.3
25-Aug-06	260	2.24	59.8	-	3.840	3.840	0.078	3596.2

5/30 SF/Slag RCA								
Specimen C								
Date	Cycles	Transverse		Longitudinal	Length		L_C	Weight
		(Khz)	P _C (%)	(Khz)	(mm)	(mm)	%	(g)
07-Jul-06	0	2.88	100.0	6.32	2.522	2.518	0.000	3688.0
14-Jul-06	31.5	2.89	100.5	6.29	2.490	2.486	-0.013	3700.4
21-Jul-06	71.5	2.85	97.7	6.25	2.514	2.504	-0.006	3705.6
27-Jul-06	104	2.89	100.5	6.33	2.562	2.556	0.015	3707.8
04-Aug-06	145	2.81	95.0	6.35	2.506	2.504	-0.006	3700.7
11-Aug-06	183	2.77	92.3	6.25	2.526	2.514	-0.002	3688.4
17-Aug-06	216	2.75	91.0	6.16	2.516	2.516	-0.001	3683.9
24-Aug-06	252	2.72	89.0	-	2.582	2.582	0.026	3669.5
31-Aug-06	289	2.70	87.7	-	2.622	2.620	0.041	3656.0
3-Sep-06	303	2.66	85.1	-	2.660	2.658	0.056	3647.9

Limestone Control								
Specimen A								
Date	Cycles	Transverse		Longitudinal	Length		L_C	Weight
		(Khz)	P _C (%)	(Khz)	(mm)	(mm)	%	(g)
14-Jul-06	0	3.22	100.0	7.16	3.814	3.806	0.000	3959.3
21-Jul-06	40.0	3.14	95.1	6.98	3.810	3.798	-0.003	3962.5
27-Jul-06	72	3.22	100.0	7.15	3.834	3.832	0.010	3966.2
04-Aug-06	113	3.24	101.2	7.20	3.750	3.748	-0.023	3967.7
11-Aug-06	151	3.26	102.5	7.29	3.834	3.824	0.007	3964.5
17-Aug-06	184	3.26	102.5	7.30	3.882	3.878	0.029	3964.7
24-Aug-06	220	3.30	105.0	7.34	3.870	3.860	0.022	3962.2
31-Aug-06	257	3.30	105.0	7.36	3.908	3.902	0.038	3757.4
7-Sep-06	293	3.32	106.3	7.30	3.902	3.900	0.038	-

Limestone Control								
Specimen B								
Date	Cycles	Transverse		Longitudinal	Length		L_C	Weight
		(Khz)	P _C (%)	(Khz)	(mm)	(mm)	%	(g)
14-Jul-06	0	3.21	100.0	7.08	3.312	3.296	0.000	3961.8
21-Jul-06	40.0	3.14	95.5	6.94	3.314	3.304	0.003	3966.5
27-Jul-06	72	3.26	102.9	7.13	3.368	3.356	0.024	3970.0
04-Aug-06	113	3.28	104.2	7.23	3.286	3.284	-0.005	3912.6
11-Aug-06	151	3.30	105.5	7.29	3.350	3.336	0.016	3974.0
17-Aug-06	184	3.30	105.5	7.27	3.378	3.374	0.031	3972.6
24-Aug-06	220	3.32	106.7	7.32	3.382	3.370	0.030	3972.6
31-Aug-06	257	3.32	106.7	7.32	3.422	3.410	0.046	3970.4
7-Sep-06	293	3.32	106.7	7.36	3.406	3.400	0.042	-

Limestone Control								
Specimen C								
Date	Cycles	Transverse		Longitudinal	Length		L _C	Weight
		(Khz)	P _C (%)	(Khz)	(mm)	(mm)	%	(g)
14-Jul-06	0	3.24	100.0	7.16	2.206	2.202	0.000	3990.7
21-Jul-06	40.0	3.22	98.8	7.10	2.232	2.218	0.006	3987.7
27-Jul-06	72	3.26	101.2	7.21	2.262	2.258	0.022	3994.5
04-Aug-06	113	3.24	100.0	7.18	2.234	2.230	0.011	3999.4
11-Aug-06	151	3.26	101.2	7.19	2.208	2.176	-0.010	3998.7
17-Aug-06	184	3.26	101.2	7.20	2.226	2.224	0.009	3997.8
24-Aug-06	220	3.26	101.2	7.23	2.222	2.218	0.006	3994.4
31-Aug-06	257	3.26	101.2	7.23	2.246	2.244	0.017	3987.8
7-Sep-06	293	3.26	101.2	7.21	2.252	2.252	0.020	-

5/30 SF/Slag Limestone								
Specimen A								
Date	Cycles	Transverse		Longitudinal	Length		L _C	Weight
		(Khz)	P _C (%)	(Khz)	(mm)	(mm)	%	(g)
14-Jul-06	0	3.28	100.0	7.29	3.174	3.172	0.000	3989.6
21-Jul-06	40.0	3.26	98.8	7.23	3.202	3.202	0.012	3991.0
27-Jul-06	72	3.22	96.4	7.13	3.162	3.158	-0.006	3993.5
04-Aug-06	113	3.22	96.4	7.10	3.162	3.162	-0.004	3990.4
11-Aug-06	151	3.22	96.4	7.13	3.162	3.156	-0.006	3990.3
17-Aug-06	184	3.22	96.4	7.13	3.186	3.184	0.005	3988.0
24-Aug-06	220	3.20	95.2	7.15	3.172	3.172	0.000	3983.1
31-Aug-06	257	3.20	95.2	7.15	3.192	3.190	0.007	3974.4
7-Sep-06	293	3.20	95.2	7.15	3.196	3.194	0.009	-

5/30 SF/Slag Limestone								
Specimen B								
Date	Cycles	Transverse		Longitudinal	Length		L _C	Weight
		(Khz)	P _C (%)	(Khz)	(mm)	(mm)	%	(g)
14-Jul-06	0	3.28	100.0	7.23	3.514	3.510	0.000	3992.0
21-Jul-06	40.0	3.27	99.4	7.27	3.556	3.552	0.017	3986.8
27-Jul-06	72	3.23	97.0	7.15	3.514	3.482	-0.011	3996.4
04-Aug-06	113	3.27	99.4	7.21	3.516	3.514	0.002	3997.0
11-Aug-06	151	3.26	98.8	7.22	3.512	3.484	-0.010	3994.8
17-Aug-06	184	3.26	98.8	7.23	3.522	3.516	0.002	3994.7
24-Aug-06	220	3.26	98.8	7.21	3.522	3.520	0.004	3993.4
31-Aug-06	257	3.24	97.6	7.21	3.540	3.534	0.010	3989.9
7-Sep-06	293	3.26	98.8	7.19	3.560	3.558	0.019	-