Multiaxially Loaded Concrete Undergoing Alkali–Silica Reaction (ASR)

by

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University of Toronto

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Abstract

Alkali-silica reaction (ASR), the predominant type of alkali-aggregate reaction, is a deleterious reaction that causes expansion, cracking, and degradation of mechanical properties of concrete. ASR remains a major problem for concrete structures worldwide. Many factors, such as alkali level, reactive component in aggregate, humidity, temperature, and stress state, affect the manifestation of ASR. While being a serious concern for ASR-affected concrete structures, the effect of stress state on the ASR expansion, cracking, and degradation of mechanical properties of concrete is poorly understood.

In this study, the effect of multiaxial stresses on the ASR-induced expansion, cracking, and degradation of mechanical properties of concrete was investigated. A novel method was developed for applying multiaxial (uniaxial, biaxial or triaxial) compressive stresses in concrete specimens and sustaining the stresses for the duration of the reaction. By measuring triaxial expansion, this study elucidated the axial and volumetric expansion of multiaxially loaded ASR-affected concrete. By performing damage rating index (DRI) analysis, ASR cracking was portrayed along three mutually perpendicular planes. ASR expansion and DRI were correlated for multiaxially loaded ASR-affected concrete. Moreover, by testing cores taken along different
directions and at different stages of ASR, this study determined the influence of stress on the degradation of mechanical properties. The test results revealed that ASR-affected concrete behaves as an orthotropic material. This study developed an expansion-stress relationship for ASR-affected concrete. The proposed relationship was validated by performing a finite element analysis of the concrete specimens from the experiment. The predicted expansions are in reasonable agreement with the measured expansions.

This study also investigated the influence of temperature and of coarse aggregate grading on the ASR performance of concrete. Increasing temperature from 38 to 50 °C shortened the test duration by more than 3 times with little effect on the response of concrete. A 10% deviation in coarse aggregate grading may result in up to 50% larger concrete expansion. Moreover, through microscopic examination, this study explained the mechanism of the partial recovery that followed the loss in the mechanical properties of concrete caused by ASR.
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Table of Contents

Abstract ................................................................................................................................. ii
Acknowledgments ................................................................................................................ iv
Table of Contents .................................................................................................................. v
Chapter 1 Introduction ......................................................................................................... 1
  1.1 Background .................................................................................................................. 1
    1.1.1 Alkali-silica reaction as a major problem in concrete structures ......................... 1
    1.1.2 State-of-the-art .................................................................................................... 2
  1.2 Research Scope .......................................................................................................... 6
    1.2.1 Objectives ......................................................................................................... 6
  1.3 Thesis Organization .................................................................................................... 7
  1.4 References .................................................................................................................. 8
Chapter 2 A New Method of Applying Long-Term Multiaxial Stresses in Concrete Specimens Undergoing ASR, and Their Triaxial Expansions .................................................. 12
  Abstract ............................................................................................................................ 12
  2.1 Introduction ................................................................................................................ 12
  2.2 Method ....................................................................................................................... 15
    2.2.1 Concrete specimens and stress scheme .............................................................. 15
    2.2.2 Specimen preparation ....................................................................................... 17
    2.2.3 High-strength bolts .......................................................................................... 20
    2.2.4 Preparatory work before loading ...................................................................... 24
    2.2.5 Loading of cube specimens ............................................................................. 26
  2.3 Concrete Stresses ....................................................................................................... 27
    2.3.1 Applied stresses ................................................................................................. 27
    2.3.2 Effect of elevated temperature ......................................................................... 27
  2.4 Results and Discussion of Triaxial Expansions ......................................................... 28
Chapter 3 Expansion–Stress Relationship for Concrete with Alkali–Silica Reaction

Abstract

3.1 Introduction

3.2 Research Significance

3.3 Experimental Investigation

3.3.1 Concrete specimens

3.3.2 Loading and conditioning of specimens

3.3.3 Expansion measurement

3.4 Experimental Results and Analysis

3.4.1 Evolution of ASR expansion

3.4.2 Stress distribution and its effect on expansion

3.5 Expansion-Stress Relationship

3.5.1 Uncoupled axial expansion

3.5.2 Maximum possible axial expansion

3.5.3 Expansion distribution along the three directions

3.6 Validation

3.7 Discussion

3.8 Conclusion

3.9 References

Chapter 4 The Effect of Multiaxial Stresses on the ASR Damage of Concrete

Abstract

4.1 Introduction

4.2 Research Significance
Chapter 4 The Effect of ASR on the Expansion, Damage Rating Index and Mechanical Properties of Concrete

4.3 Experimental Investigation

4.3.1 Concrete ingredients and mix design

4.3.2 Casting of cube specimens

4.3.3 Loading and conditioning of specimens

4.3.4 Core testing

4.3.5 Damage rating index (DRI)

4.4 Experimental Results and Discussion

4.4.1 Expansion

4.4.2 Cracking

4.4.3 Effect of ASR on mechanical properties

4.5 Summary and Conclusions

4.6 References

Chapter 5 The Effect of Elevated Temperature on the Expansion, Damage Rating Index and Mechanical Properties of ASR-Affected Concrete

5.1 Introduction

5.2 Materials and Methods

5.2.1 Concrete mix design and materials

5.2.2 Casting and conditioning of the specimens

5.2.3 Test details

5.3 Results and Discussions

5.3.1 Expansion

5.3.2 Modulus of rupture

5.3.3 Dynamic modulus of elasticity

5.3.4 Damage rating index (DRI)

5.4 Summary and Concluding Remarks
7.2.2 Test details ..................................................................................................................143

7.3 Mechanical Properties ....................................................................................................144

7.4 Microscopic Analysis of the Mechanism of Partial Recovery .......................................145

7.4.1 Early stage: fluid reaction products and pore pressure .............................................145

7.4.2 Later stage: Filling of cracks and interstitial transition zone (ITZ) with reaction products and transformation into solid products ........................................147

7.5 Conclusion .....................................................................................................................150

7.6 References .....................................................................................................................152

Chapter 8 Conclusions and Recommendations .....................................................................154

8.1 Summary .........................................................................................................................154

8.2 Concluding Remarks .......................................................................................................155

8.3 Suggestions for Future Work ........................................................................................162

Appendices ..........................................................................................................................164

Appendix A: Accelerated mortar bar test (AMBT) of Orillia sand ........................................165

Appendix B: Construction of the acceleration chamber .......................................................166

Appendix C: Strain in the high-strength bolts in the cube specimens ...................................170

Appendix D: Preliminary analysis of the cube specimens by using ANSYS ..........................172

Appendix E: Crack visualization using ultraviolet fluorescent imaging ...............................174

Appendix F: Mechanical properties of cores of reactive cube specimens ............................176

Appendix G: Ultrasonic pulse velocity (UPV) .....................................................................178

Appendix H: Transverse expansion measurement of concrete prisms .................................182

Appendix I: DRI along the longitudinal direction of concrete prisms .................................185

References for the Appendices .............................................................................................187
Chapter 1
Introduction

1.1 Background

1.1.1 Alkali-silica reaction as a major problem in concrete structures

Alkali-aggregate reaction (AAR) has been identified as one of the major deterioration problems in concrete structures. The most prevalent type of AAR is the alkali-silica reaction (ASR). After the first identification of ASR in the 1930s by Stanton [1], the chemistry of the reaction has been extensively studied and adequately understood. ASR is a chemical reaction between the reactive silica available in certain types of aggregates and the alkali hydroxides in the concrete pore fluid. The alkali in concrete is supplied by cement and in some cases by admixtures, aggregates or the external environment. The product of the reaction is a gel that imbibes water. The consequences are the initiation of cracks in the reactive aggregate particles, propagation of the cracks into the paste matrix, exudation of the gel from the cracks, volumetric expansion of the concrete, stress development due to expansive pressure, and degradation of mechanical properties. The damage caused by the reaction depends on several factors including the type of concrete [2,3], the rate of formation of the gel [4], and its volume concentration within concrete [5,6]. Extensive studies on ASR over the past several decades have made it possible to construct new structures reasonably resistant to ASR [7–9]. However, the greatest challenge regarding ASR remains to ensure the satisfactory performance of existing ASR-affected structures by assessing ASR-damage.

What makes ASR a serious problem is that once it occurs, no repair methods are available to efficiently and definitely stop the process of ASR and associated damage. For instance, the Lady Evelyn Dam in Canada suffered extensively from ASR and had to be replaced by a new dam in 1971 [9]. The 35 m high Maentwrog Dam in the UK had to be replaced in 1992 owing to the ASR damage [10]. Similarly, the Lake Townsend Dam in the U.S.A. suffered from ASR so extensively that any repair attempts could not control the cracking and expansion, leading to a replacement at a cost of 35 million U.S. dollars [11]. A bridge in Finland required demolition and replacement owing to ASR damage [12]. Cracking due to ASR was also observed in the concrete containment structures of nuclear power plants in Canada [13], which are designed as crack-free structures by applying biaxial prestressing. Likewise, the Seabrook Nuclear Power Plant in the U.S.A. has suffered from ASR and has been subjected to an investigation regarding
its aging management [14]. ASR has caused problems in at least 46 countries [15], thus, making ASR a serious worldwide problem.

1.1.2 State-of-the-art

1.1.2.1 The reaction

ASR is a slow reaction and usually takes 10 years or more to show its effects in field concrete structures [16]. Three components need to be present together for the reaction to occur, namely alkali, water and reactive aggregate. The amount of alkali content in concrete in many existing structures is generally sufficient to initiate the reaction. Water is always available in the capillary pores of concrete, and is generally sufficient to allow the reaction to proceed unless concrete is exceptionally dry [17].

An increase in temperature accelerates the rate of ASR by thermoactivating the two processes involved in ASR [18]. Firstly, increase in temperature accelerates the dissolution of silica in aggregates. For instance, the rates of dissolution of three types of silica particles measured at 25 °C in 3 M NaOH solution for 3 days were 71%, 8% and 7%, whereas these amounts were, respectively, 100%, 66% and 31% when measured at 80 °C [19]. Secondly, increased temperature accelerates the rate of formation of reaction products [18]. While several theories exist in the literature about the chemistry of ASR, the reaction can be considered as taking place in the following three steps [20,21], where R stands for the alkali atom Na or K.

1. Scission (breaking) of siloxane networks by OH− ions to form alkali silicate and silicic acid—

\[
\text{Si} - \text{O} - \text{Si} - \text{R}^+ + \text{OH}^- \rightarrow \text{Si} - \text{O} - \text{R} + \text{H} - \text{O} - \text{Si} - 
\]

2. Immediate conversion of silicic acid to hygroscopic alkali silicate gel—

\[
\text{H} - \text{O} - \text{Si} - \text{R}^+ + \text{OH}^- \rightarrow \text{Si} - \text{O} - \text{R} + \text{H}_2\text{O}
\]

3. Imbibition of water by hygroscopic alkali-silicate gel and swelling—

\[
\text{Si} - \text{O} - \text{R} + n\text{H}_2\text{O} \rightarrow \text{Si} - \text{O}^{\cdot} - (\text{H}_2\text{O})_n + \text{R}^+
\]
1.1.2.2 ASR expansion and the effect of stress state

Hydrostatic pressure of ASR gel due to water absorption leads to the expansion of concrete [22]. Expansion is a common manifestation of ASR and has been extensively studied for decades. Expansion measurement has been used as an indicator for the reactivity of aggregate and concrete mixtures [23–25]. Measurements on unrestrained specimens have established expansion as a characteristic feature of ASR. The ultimate longitudinal expansion of concrete due to ASR exceeded 0.5% for experimental mix designs using extremely reactive aggregate [26,27]. This range of strain is very large compared to the typical strains related to reinforced concrete structures, such as concrete shrinkage strain of 0.05%, ultimate compressive strain of concrete of approximately 0.3%, and the yield strain of reinforcing steel which is typically 0.2%.

However, concrete in structures is often subjected to restraints or stresses in one or multiple directions, and hence, expansion measured on unrestrained specimens does not appropriately translate to the expansion of concrete structures. For instance, restraining effect provided by longitudinal steel reinforcement in concrete beams reduced the axial expansion by approximately 40% relative to the unreinforced beams [7]. Effect of stress on the ASR expansion of concrete has been studied by maintaining uniaxial compressive stress in concrete that was undergoing ASR [6,28–32]. Such studies have revealed that expansion is reduced in the direction of compressive stress and is transferred into the unstressed directions. The phenomena of reduction and transfer complicate the relationship between the ASR expansion and the stress state. Studies limited to uniaxial stress are inadequate for understanding the relationship between the ASR expansion that occurs volumetrically and the stress state that can be multiaxial.

In order to investigate the effect of multiaxial stress on ASR expansion, Gravel et al. [33] measured triaxial expansion of a cube specimen subjected to biaxial stresses. Stress in the range of 3.3 to 6 MPa was shown to prevent expansion in the stressed direction. However, the study revealed very little about the ASR expansion of concrete under multiaxial stress state. Multon and Toutlemonde [31] tested ASR expansion of axially loaded concrete specimens confined by steel cylinders in the radial directions. The authors [31] concluded that “the ASR volumetric imposed strain can be considered as constant whatever the stress state”. However, the experimental setup [31] was such that the lateral confinement provided passive pressure, which was activated only when concrete expanded laterally. The authors [31] tried to explain the effect
of stress on the volumetric strain based on an experiment in which the stress was actually the effect of the volumetric expansion. Thus, the conclusion regarding volumetric expansion may not be true and may not apply to real concrete structures in which concrete has been subjected to stresses even before the reaction takes place.

1.1.2.3 Cracking due to ASR

Another common characteristic of ASR is the cracking of concrete. Several studies have been devoted to explain the cracking mechanism [3,20,34–36]. Even though the mechanisms of cracking have been reported to be different for different types of aggregates, the pressure imposed by ASR gel is commonly believed as the cause of internal cracking of concrete. The cracking in ASR initiates in reactive aggregate particles and is followed by the paste cracking [20], which is influenced by the strength of concrete and the rate of reaction [4].

Cracks have been monitored as a means of assessing damage of existing concrete structures. Surface cracks have been mapped to assess the ASR damage experienced by concrete structures, sometimes in terms of “cracking index” [37,38]. The cracking index method involves monitoring cracks along the four sides and two diagonals of a square area on a concrete surface, and does not differentiate the cracks by their orientation, which can, however, be influenced by the stress state of concrete [39]. Another method to assess the damage due to ASR by monitoring cracking is the “damage rating index” (DRI). The DRI method was developed in the 1990s as a means of quantitatively assessing the damage of the concrete microstructure due to ASR [40]. The DRI is rooted on cracking that is characteristic of ASR damage and accounts for seven damage features. For concrete structures, the DRI method can be applied to concrete samples taken from the structures even without a reference measurement. While the DRI method has been extensively applied to laboratory concrete specimens [3,41,42] and to the cores from existing structures [40], the influence of stress state on damage has not been studied. Stress state may cause spatial and directional variation in the cracking and DRI values of concrete in the ASR-affected structures.

1.1.2.4 Degradation of mechanical properties of concrete due to ASR

Blight and Alexander [17] summarized the work of different researchers on the effect of ASR on concrete properties. There is a clear indication that mechanical properties are degraded when concrete suffers from ASR. Many studies have reported on the degradation of mechanical
properties of concrete due to ASR [2,27,43–45]. However, such studies mostly test unrestrained specimens that do not truly represent the ASR damage of structural concrete having a variety of stresses, as indicated by Giaccio et al. [46]. Degradation of mechanical properties in the context of concrete structures is sometimes assessed by testing cores taken from ASR-affected concrete structures [13,40,47]. Such tests have been the basis of the “stiffness damage test” that was proposed as a tool to evaluate the ASR damage [48,49]. Even though testing of over 1000 cores from several ASR-affected structures was reported [50], the influence of stress-state on the evolution and consequence of ASR has been inadequately acknowledged. Since ASR damage can be influenced by the concrete’s stress state, the information provided by two cores at the same location but along two directions represent two widely different scenarios. Large variation has been observed in the mechanical properties of cores from reinforced concrete members taken parallel and perpendicular to the longitudinal reinforcement [47,51]. The prestressing effect produced by the reinforcing steel due to the expansive pressure from ASR was attributed for such directional variation [13,51]. However, the stress level and its relationship to the degradation of mechanical properties of concrete due to ASR was not explored. Since the effect of restraint is essentially through the prestressing stress it applies on concrete, a clear understanding of the effect of stress state on the degradation of mechanical properties of concrete due to ASR can help in reliably evaluating the performance of ASR-affected concrete structures.

1.1.2.5 Numerical analysis of ASR-affected concrete structures

The attack of ASR has been generally recognized as an urgent problem for many important concrete structures, such as dams, bridges and nuclear power plants. An increasing interest has arisen among public and private institutions about addressing the deteriorating problems in concrete structures and extending the life of the structures. With the advancement of computational technology, it is now possible to make a thorough analysis of a structure numerically. Many researchers have attempted to predict the structural performance of ASR-affected concrete structures through numerical simulations. Numerical studies have been performed by considering the influence of stress on ASR expansion [5,52–57]. The reaction kinetics has been sometimes coupled into the relationship between ASR expansion and stress state [5,55]. While ASR expansion in each direction was independently assessed in some models [52,53], the interaction among the expansions along the three mutually perpendicular directions has been considered in some other models [5,55]. However, “due to lack of sufficient
experimental data” for biaxial and triaxial confinement, the relationship between expansion and stress and the distribution of volumetric expansion along different directions had to rely on “engineering common sense” [55]. Numerical simulation appears to be the most potent tool to answer many questions related to the consequences of ASR in the existing structures. However, the effect of multiaxial stresses on the ASR performance of concrete, which is a key input for a plausible numerical simulation, has been insufficiently studied.

It is clear that one of the reasons why managing ASR-affected structures is difficult is because concrete in structures is subjected to multiaxial stresses, and the effect of multiaxial stress on the expansion, cracking and degradation of the mechanical properties of concrete is least understood. Thus, it has become essential to investigate the evolution and consequences of ASR in concrete specimens that can simulate the multiaxial stress state of real concrete structures. Since no standard method exists for applying and maintaining triaxial stress in concrete specimens for an extended duration sufficient for the reaction to evolve and exhaust, such an investigation should begin by developing a new method for applying and maintaining multiaxial stresses.

1.2 Research Scope

This study was conducted to investigate the response of concrete undergoing ASR and subjected to a set of stresses. Expansion, cracking and the degradation of mechanical properties of concrete were the primary mechanisms investigated using several tools and tests, such as expansion measurement, concrete microscopy, and destructive and non-destructive testing of concrete for mechanical and transport properties. In addition, numerical analysis was conducted. The outcomes of this study are expected to be useful in the performance assessment of existing concrete structures suffering from ASR.

1.2.1 Objectives

The objective of this PhD study was to design, characterize and model multiaxially loaded ASR-affected concrete. In order to achieve this overarching objective, the following were the primary sub-objectives:

1. To develop an experimental method to apply and sustain multiaxial compressive stresses in concrete specimens undergoing ASR
2 To evaluate the performance of unstressed and stressed ASR-affected concrete by using different methods, such as expansion measurement, testing of mechanical and transport properties by destructive and non-destructive testing, and microscopic analysis of concrete specimens

3 To develop a relationship between the ASR expansion and the stress state of concrete that can be utilized to analyze ASR-affected concrete structures

1.3 Thesis Organization

This first chapter gives an overall introduction to the thesis. Chapters 2 to 4 focus on the multiaxially loaded concrete specimens undergoing ASR which builds the central concept of this thesis. Chapter 2 presents the new method of stress application in concrete specimens. Chapter 3 presents the development of an expansion-stress relationship for the ASR-affected concrete. Chapter 4 is focused on the effect of multiaxial stress on the ASR damage of concrete, specifically the expansion, cracking and degradation of mechanical properties of concrete.

Chapter 5 presents the effect of elevated temperature on the expansion, damage rating index and mechanical properties of ASR-affected concrete. Chapter 6 presents the effect of coarse aggregate grading on the ASR expansion and damage of concrete. Chapter 7 is devoted to the microscopic study of concrete samples in an attempt to understand the partial recovery in the mechanical properties of concrete following a loss due to ASR. Finally, Chapter 8 concludes this thesis by summarizing the key novel scientific contributions and suggests some ideas towards the potential advancement of the present research. Additionally, some relevant details have been included in the appendices of this thesis.

The content of this thesis has been organized on the framework of several themes that can form the basis of several stand-alone manuscripts for scientific publication. The list of manuscripts is as follows:


1.4 References


Chapter 2
A New Method of Applying Long-Term Multiaxial Stresses in Concrete Specimens Undergoing ASR, and Their Triaxial Expansions

Abstract
An understanding of the response of multiaxially loaded concrete to the expansive reactions, such as alkali-silica reaction (ASR), remains a prerequisite for predicting the long-term behavior of ASR-affected concrete structures, such as dams and containment structures. However, no suitable techniques exist to apply biaxial and triaxial stresses in concrete specimens and to sustain the stresses for monitoring the long-term behavior of concrete. Consequently, existing numerical models of concrete structures undergoing ASR rely on the experimental findings obtained from specimens with insufficient restraints. This chapter presents the development of a new method to apply multiaxial (uniaxial, biaxial or triaxial) stresses. The stresses were applied to multiple concrete cube specimens, and were sustained for over 1 year as ASR evolved in the specimens subjected to an accelerated condition of 50±0.5 °C temperature and > 95% relative humidity. The specimens were measured for expansion in three directions. The merit of the proposed method in the study of ASR is illustrated by presenting the results on the triaxial expansions of several unstressed and multiaxially stressed cube specimens until the exhaustion of the reaction. The results demonstrate a strong influence of multiaxial stress states on the ASR expansion of concrete.

Keywords: concrete; alkali-silica reaction (ASR); expansion; multiaxial stresses; high-strength bolts

2.1 Introduction
Alkali-silica reaction (ASR) is a major deterioration problem in existing concrete structures such as dams, bridges and nuclear power plants. Cracking due to ASR was observed in the concrete containment structure of a nuclear power plant in Canada [1] even though the structure was biaxially prestressed and, thus, would be expected to be free from cracking. As highlighted by Saouma and Hariri-Ardebili [2] with the case of the Seabrook nuclear power plant in the USA,
ASR is an important aspect of aging management in existing nuclear power plants globally. Presumably, the greatest challenge in such concrete structures is to perform the prognosis of the consequences of ASR and decide the future of the structure as whether to demolish, treat or utilize as is.

Many researchers have attempted to predict the structural performance of ASR-affected concrete structures through numerical simulations. To mention a few, Leger et al. [3] analyzed a concrete dam by calculating the ASR strain in each direction based on the confinement stress in the respective direction. However, the three strains were independent of each other. This independency is not realistic since ASR expansion can transfer from the direction of relatively larger stress to the direction of relatively smaller stress or no stress [4]. Huang and Pietruszczak [5] analyzed a hydroelectric dam by modeling the thermo-mechanical effects of ASR but chose many parameters arbitrarily by acknowledging that the only experimental information on ASR-induced expansion of concrete was that “pertaining to the rate of free expansion”. Ulm et al. [6] implemented a thermo-chemo-mechanics model of ASR in the finite element analyses of a concrete dam and a bridge box girder. However, they did not include the “stress-induced anisotropy” of the ASR expansion. Grimal et al. [7,8] proposed a numerical model that included the capability of modeling orthotropic damage by considering the expansion transfer from restrained to unrestrained directions. However, the experimental basis for the model was deficient in assessing the ASR expansion under multiaxial stresses. Saouma and Perotti [9] proposed a rigorous constitutive model for multiaxially loaded concrete. The model proposed to redistribute the expansion along the principal stress directions based on a set of assumed weight factors, owing to the “lack of sufficient experimental data” for biaxial and triaxial confinement. Numerical simulation appears to be the most potent tool to answer many questions related to the consequences of ASR in the existing structures. However, the effect of ASR on multiaxially loaded concrete structures, which is a key input for a plausible numerical simulation, has been insufficiently studied. Perhaps, the underlying reason for that is the unavailability of a method that can apply multiaxial stresses in multiple concrete specimens and sustain them for a long period until ASR evolves in them.

An experimental setup to apply and sustain uniaxial stress in mortar or concrete specimens undergoing ASR was developed by Ferraris et al. [10]. A similar setup was subsequently implemented by other researchers [11,12] to study the effect of ASR under uniaxial stress state.
Dunant and Scrivener [13] devised a creep frame setup to study concrete cylinder specimens maintained at uniaxial stresses of 0, 5, 10 and 15 MPa. Experiments with uniaxial restraint have indicated that cement mortar specimens with reactive aggregate can produce expansive pressure up to 14 MPa, and concrete specimens with reactive aggregate can produce 2.5 to 10 MPa pressure [10–13]. ASR expansion in concrete prisms was suppressed in the stressed direction by a stress as low as 2.2 MPa [12] but the stress caused more expansion in the lateral directions [12,14]. These findings clearly indicate the need to extend the study of ASR for the specimens with biaxial and triaxial stresses.

Yurtdas et al. [15] recognized the importance of a triaxial test and implemented it to study the response of ASR-affected mortar specimens. However, the test was performed in a conventional triaxial cell; the stresses were applied only during testing but not during the evolution of ASR. Multon and Toutlemonde [4] realized the importance of an experimental investigation to study the ASR expansion while the specimen is under multiaxially stressed condition. To study the effect of triaxial stresses, they monitored ASR expansion of axially loaded concrete cylinders confined by steel rings in the radial directions. Even though their study brought some valuable insight in understanding ASR expansion under triaxial stresses, the experimental setup was such that the lateral confinement provided passive pressure, which was activated only when concrete expanded laterally. As the lateral pressure depends on the lateral expansion, the lateral expansion does not occur at a constant stress state but rather at an increasing stress state from zero to a final value. The authors [4] tried to explain the effect of stress on the volumetric strain based on an experiment in which the stress was actually the effect of the volumetric expansion. Thus, the findings may not apply to real concrete structures in which concrete has been subjected to stresses even before the ASR takes place.

In order to address the need of a setup for multiaxial stresses, this chapter reports the development of a novel experimental method which is able to apply and maintain uniaxial, biaxial or triaxial stresses on concrete specimens that can be conditioned for the evolution of ASR while maintaining the applied stress state. Relying on the idea of prestressing as in the historic study by Furr [16], the proposed method utilizes high-strength bolts to apply and sustain compressive stresses in concrete cube specimens in three mutually perpendicular directions. The method was adopted in an experimental program in which twenty-nine cube specimens were loaded to different stress states namely, no-stress, uniaxial, biaxial and triaxial stress. In order to
illustrate the usefulness of the proposed practical method, this chapter presents the triaxial expansions of the multiaxially loaded cube specimens subjected to an accelerated condition of 50±0.5 °C temperature and > 95% relative humidity. The method will not only allow one to understand the ASR expansion under different stress states but will also help to elucidate the damage of ASR-affected concrete in consideration with the stress states. Furthermore, this method will be useful for experimental investigations of concrete related to ASR and other expansive reactions.

2.2 Method

2.2.1 Concrete specimens and stress scheme

The proposed method of stress application was developed to investigate the effect of multiaxial (X, Y, or Z, or combinations thereof) stresses on the triaxial (X, Y and Z) expansions and on the damage of concrete undergoing ASR. In order to rule out any effect the aspect ratio might induce, this study is based on concrete cube specimens of 254 mm size. The post-tensioning method of prestressing was chosen as the technique to apply and sustain the desired compressive stresses in the concrete specimens. The stress application in one direction required four high-strength bolts to be inserted into the four rigid sheaths embedded in the concrete during casting. Figure 2.1 shows the configuration of sheaths oriented in the three directions of a concrete specimen. Each bolt was stretched to a desired stress level by producing reaction against a pair of bearing plates placed symmetrically on the opposite surfaces of a concrete specimen. Seven stress states were considered in a range of 0 to 10 MPa compressive stress as shown in Table 2.1. The stress level and precision level were chosen to be representative of typical concrete structures. For the two levels of stresses (10 MPa and 4 MPa), two sizes of bolts were used: four bolts of 19.0 mm diameter for 10 MPa and four bolts of 12.7 mm diameter for 4 MPa stress in concrete.

As shown in Table 2.1, four (three reactive and one non-reactive) cube specimens were cast for each of the seven stress states. One more reactive specimen was cast so that it could be tested as a reference specimen before the application of any stresses. Three reactive specimens were cast so that each could undergo destructive testing at a desired test age. Moreover, multiple specimens could help to assess the repeatability of the proposed scheme. Non-reactive specimens were cast to serve as the control specimen for each stress type. Expansion due to ASR for a
reactive specimen can be decoupled from the creep and shrinkage strains by taking the algebraic difference between the expansion of a reactive specimen and the corresponding expansion (strain) of the control specimen (non-reactive concrete) of the identical stress state.

Figure 2.1: Configuration of a concrete cube specimen with sheaths oriented in the three mutually perpendicular directions

Table 2.1: Details of concrete specimens and stress scheme

<table>
<thead>
<tr>
<th>Type</th>
<th>Designation</th>
<th>$f_x$</th>
<th>$f_y$</th>
<th>$f_z$</th>
<th>Non-reactive</th>
<th>Reactive</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>No-stress</td>
<td>n</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>1</td>
<td>$1^b$+$3^c$</td>
<td>5</td>
</tr>
<tr>
<td>Uniaxial</td>
<td>u</td>
<td>4</td>
<td>0</td>
<td>0</td>
<td>1</td>
<td>$3^c$</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>U</td>
<td>10</td>
<td>0</td>
<td>0</td>
<td>1</td>
<td>$3^c$</td>
<td>4</td>
</tr>
<tr>
<td>Biaxial</td>
<td>b</td>
<td>4</td>
<td>4</td>
<td>0</td>
<td>1</td>
<td>$3^c$</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>10</td>
<td>4</td>
<td>0</td>
<td>1</td>
<td>$3^c$</td>
<td>4</td>
</tr>
<tr>
<td>Triaxial</td>
<td>t</td>
<td>4</td>
<td>4</td>
<td>4</td>
<td>1</td>
<td>$3^c$</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>T</td>
<td>10</td>
<td>4</td>
<td>4</td>
<td>1</td>
<td>$3^c$</td>
<td>4</td>
</tr>
</tbody>
</table>

* The stress shown is the nominal value. The actual value was different as explained in Section 2.2.5.2

$^b$ Destructively tested at 56 days of casting

$^c$ One each destructively tested at three different ages of accelerated curing
The mix design for both reactive and non-reactive concrete was based on ASTM C1293 [17]. Reactive concrete was made with reactive coarse aggregate and non-reactive fine aggregate. The reactive coarse aggregate was Spratt aggregate (crushed siliceous limestone) from Ontario, Canada [18]. Natural sand from Orillia, Ontario was used as the non-reactive fine aggregate. (Test results for Orillia sand are shown in Appendix A). Non-reactive concrete was made with non-reactive coarse and non-reactive fine aggregate. The non-reactive coarse aggregate was crushed limestone aggregate from Milton, Ontario. The cube specimens, after stress application, were eventually conditioned inside an acceleration chamber maintained at 50±0.5 °C temperature and > 95% relative humidity. (Construction of the chamber is discussed in Appendix B).

2.2.2 Specimen preparation

2.2.2.1 Formwork

The specimens were cast by using specially designed wooden formwork as shown in Figure 2.2. The formwork included four small holes on each face for placing expansion measurement studs (to be discussed in Section 2.2.2.3), four circular slots on each face for holding the ends of the sheaths, and a 100 mm diameter hole at the top for placing concrete. The cube specimens were demolded at 24 hours.

2.2.2.2 Stainless steel sheaths

In concrete undergoing expansive reaction, any interior space may absorb the expansive pressure of the reaction. Therefore, no interior space should exist inside the concrete specimen. However, there should be an annular space between the bolt and the sheath to facilitate the restraint-free movement of a bolt affixed with a strain gauge. Thus, a rigid sheath was chosen to withstand the likely pressure coming from the expanding concrete. Irrespective of the stress state, all the specimens were cast with twelve steel sheaths, four along each of the three mutually perpendicular directions (Figure 2.1). A minimum concrete cover of 40 mm was provided for the outermost sheaths.

In order to prevent corrosion, stainless steel tubing was chosen as the sheath material. As shown in Figure 2.3, two sizes of sheaths were used: the larger for the 19.0 mm bolts and the smaller for the 12.7 mm bolts as well as for the no-stress directions. The sheaths provided an annular space of approximately 2 mm for the bolts, which was sufficient for passing the bolts affixed with a
strain gauge. The capacity of the sheaths against collapse pressure ranged from 19 to 60 MPa, as calculated from various references [19–22]. This confirmed that the sheaths would be safe for use in the specimens in which the external compressive pressure from concrete was expected to remain below 10 MPa.

To prevent a sheath from sharing any stress applied to concrete, two measures were adopted. First, the sheath was cut 13 mm shorter than the size of a cube so that it would remain countersunk by 6.5 mm on either surface of concrete (Figure 2.1). Specially designed plastic stoppers, as shown in Figure 2.3, were used to hold the sheaths in position during the casting of concrete and to maintain the countersunk holes on the concrete surface upon removal of the stoppers. (During conditioning of the cube specimens, the plastic stoppers also acted as the footings to rest the specimen so that an air space for moisture circulation was always ensured to the bottom surface). The countersunk holes ensured that the sheath did not provide support to the bearing plates. Countersunk holes were not, however, made for the sheaths that would not house any bolts. Second, all the sheaths were covered with a thin self-adhesive plastic sheet so that they would not develop any bond to concrete. No-bond between the sheath and concrete was evidenced as the sheath could actually slide during unloading of the cube specimens for destructive testing.

Figure 2.2: Wooden formwork for concrete cube specimens of 254 mm size
2.2.2.3 Arrangement for expansion measurement

In order to measure the expansion of the cube specimens in the X, Y and Z directions, four studs were embedded in each face of the specimens. The studs were made from threaded stainless steel rods. Studs were cut to a length of 30 mm and were machined on the outer end for a smooth round surface suitable for a micrometer measurement. For the cube specimen of 254 mm length, the studs were 30 mm long, yielding the gauge length for the expansion measurement to be 194 mm. It was important to prevent the studs from taking any impact or providing any bearing to the specimen when resting on a support. Thus, the studs needed not to have any projection on the concrete surface. This was accomplished by having specially designed conical stud holders. A stud holder was bolted into a small hole made in the formwork as shown in Figure 2.2, and a stud was screwed into it. During demolding, the stud holder was first unbolted from the formwork, and then, gently unscrewed to expose the machined end of the stud along with approximately 8 mm length of the stud. A pair of studs on opposite faces offered one measurement using the micrometer, thus the four pairs offered four measurements in each direction.
For the measurements to be taken across the body of the specimen, a 250–275 mm range digital micrometer, with a resolution of 1 µm, was acquired from Mitutoyo Inc., Japan. The manufacturer-specified accuracy of the micrometer was ±4 µm. Before taking the measurements of a cube specimen, the micrometer was calibrated to the standard reference invar bar. The initial (reference) micrometer measurement was taken immediately after demolding. The expansion at an age was taken as the difference between the measurement at the age and the reference measurement. The expansion in each direction was measured as the average of the four measurements. This way, the average value was a measure of the axial expansion free from the strain, if any, due to bending.

2.2.3 High-strength bolts

2.2.3.1 Bolt material

Cubes were post-tensioned by ASTM A193 Grade B7 [23] type high-strength all-thread rods. A typical triaxially loaded cube specimen is shown in Figure 2.4. The all-thread rods were cut into lengths of 380 mm and were used as bolts (thus referred to as bolts) with a pair of nuts. Grade B7 steel was chosen for several reasons. First, it is suitable for high temperature application and has been specified for the components in contact with water. Second, its thermal expansion coefficient is similar to that of the concrete used in this study. Third, it is easily available in the market. Fourth and most importantly, it has high strength and ductility that are needed for this study.

Grade B7 steel has a specified minimum yield strength of 720 MPa and an ultimate strength of 860 MPa. The minimum elongation of B7 material at failure is 16%, and the reduction in the cross-sectional area at failure is at least 50%. The actual strengths of the bolts were determined by a tension test discussed in Section 2.2.3.3. The bolts, as well as compatible hex nuts of 2H grade [24], washers, and bearing plates, were purchased from Williams Form Hardware and Rockbolt (Canada) Ltd. The fasteners and the bearing plates were hot-dip galvanized to prevent corrosion.

2.2.3.2 Neck formation

One problem with a bolt is that the net cross-sectional area can vary to some extent from thread to thread. Thus, a neck was formed in all the bolts by removing the threads from a 40 mm long
section at the mid-length (Figure 2.5). Necks were formed in the bolts at the workshop of the supplier, prior to galvanization. The actual neck diameter was measured at the laboratory for several bolts by using a digital caliper. Table 2.2 shows the average diameter at the neck for the bolts used in this study. As shown, the coefficient of variation (COV) is extremely low for each diameter category. Thus, the average diameter and the average cross-sectional area were adopted.

By making the cross-sectional area at the neck smaller than the specified minimum net area, the neck was designed to always become the section having maximum stress and strain corresponding to an applied load. This arrangement allowed the neck to be utilized for affixing strain gauges. This was necessary for two reasons: firstly, to read the strain in the bolt during a tension test; and secondly, to monitor the stress level in the bolt during and after loading the cube specimens.

![Figure 2.4: A typical triaxially loaded cube specimen](image)

<table>
<thead>
<tr>
<th>Nominal bolt diameter (mm)</th>
<th>Specified minimum net area (mm²)</th>
<th>Average measured neck diameter (mm)</th>
<th>Standard deviation (mm)</th>
<th>Coefficient of variation</th>
<th>Average cross-sectional area at neck (mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12.7</td>
<td>91.0</td>
<td>9.60</td>
<td>0.012</td>
<td>0.13%</td>
<td>72.4</td>
</tr>
<tr>
<td>19.0</td>
<td>215.0</td>
<td>15.14</td>
<td>0.025</td>
<td>0.17%</td>
<td>180.1</td>
</tr>
</tbody>
</table>
2.2.3.3 Tension test of the galvanized bolts

Two bolts were tested for tensile strength from each diameter category. Two strain gauges were glued on the diametrically opposite faces of the bolts at the neck region. The tensile strength results are shown in Table 2.3 and the stress-strain curves are presented in Figure 2.6. The stress-strain curves for the two bolts of each diameter category practically overlap.

Figure 2.6 shows that the bolts exhibited an upper yield point that could be traced from the recorded data of stress and strain. The corresponding stress and strain were referred to as the yield stress and yield strain, respectively. The ultimate failure of the bolts (not seen in Figure 2.6) occurred well after the strain gauges failed. Figure 2.6 shows that the post-yield portion of the stress-strain curve is almost horizontal for strain exceeding 5000 µm/m.

Table 2.3: Tensile strength results of the galvanized bolts

<table>
<thead>
<tr>
<th>Bolt number</th>
<th>Bolt diameter (mm)</th>
<th>Measured net dia. at the neck (mm)</th>
<th>Cross-sectional area (mm²)</th>
<th>Yield strain (µm/m)</th>
<th>Yield load (kN)</th>
<th>Yield stress (MPa)</th>
<th>Peak load (kN)</th>
<th>Ultimate stress (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>12.7</td>
<td>9.60</td>
<td>72.4</td>
<td>4850</td>
<td>66.7</td>
<td>921</td>
<td>68.2</td>
<td>943</td>
</tr>
<tr>
<td>2</td>
<td>12.7</td>
<td>9.59</td>
<td>72.2</td>
<td>4700</td>
<td>65.9</td>
<td>913</td>
<td>66.5</td>
<td>921</td>
</tr>
<tr>
<td>3</td>
<td>19.0</td>
<td>15.15</td>
<td>180.3</td>
<td>5050</td>
<td>167.4</td>
<td>929</td>
<td>175.4</td>
<td>973</td>
</tr>
<tr>
<td>4</td>
<td>19.0</td>
<td>15.15</td>
<td>180.3</td>
<td>4950</td>
<td>165.3</td>
<td>917</td>
<td>172.9</td>
<td>959</td>
</tr>
</tbody>
</table>
2.2.3.4 Bearing plates

The tension in the prestressing bolts was developed by bearing on thick plates resting on two opposite surfaces of a cube specimen. Stress along one direction was applied through prestressing with four identical bolts with a diameter either of 12.7 or 19 mm. Corresponding to the four bolts, each loaded surface of a cube specimen consisted of four bearing plates as shown in Figure 2.4.

Ideally, the size of the bearing plates should be no less than the surface area of concrete in order to distribute the stress uniformly. However, this was not possible considering the space required for the expansion and ultrasonic pulse velocity measurements, and for the ingress of moisture into the specimen. Determining the size of the plates further required to consider the convenience of obtaining the required size and the complexity of handling a large plate. Thus, the bearing plates were chosen to partially cover the concrete surface by anticipating some non-uniformity of the stress distribution at the near-surface concrete. As the gauge length of concrete for expansion measurement excluded a thickness of 30 mm of concrete cover on each side, the influence of the near-surface concrete was expected to be minimum. However, the stress state could be more
accurately obtained by performing a numerical analysis. Such an analysis will be performed in Chapter 3 where the measured expansion results are interpreted in terms of the stress state.

Two sizes of plates were used for the two types of bolts. The characteristic compressive strength \(f'_c\) of the reactive concrete was 39.3 MPa (based on a test of 26 companion cylinders for which average strength was 43.4 MPa and the standard deviation was 2.9 MPa). The \(f'_c\) for the non-reactive concrete was 40.5 MPa (based on a test of 7 companion cylinders for which average strength was 49.4 MPa and the standard deviation was 2.5 MPa). The design bearing strength would be \(0.85 \phi f'_c\) where \(\phi\) is the strength factor taken as 0.65. Remaining on the conservative side in assuming the degradation of compressive strength over time would be 20%, the design bearing strength for the reactive concrete was 17.4 MPa. The chosen sizes of the plates were 76 mm by 76 mm with a central hole of 14 mm diameter for the 12.7 mm bolt, and 102 by 102 mm with a central hole of 21 mm for the 19.0 mm bolt. For the chosen sizes, the maximum bearing stresses in concrete were 11.4 MPa and 15.2 MPa for the smaller and the larger plates, respectively.

The thickness of the plates was designed to withstand the bending moment at the critical section when the plate carried the maximum bearing stress. A calculation showed the required thicknesses to be 11 mm and 16 mm corresponding to the 12.7 mm and 19.0 mm bolts, respectively. However, the thicker (19 mm thick) bearing plates were chosen to remain on the conservative side.

### 2.2.4 Preparatory work before loading

#### 2.2.4.1 Adhesion of the strain gauges

A strain gauge was required to monitor the strain of the bolt during loading. For this purpose, a strain gauge, integrated with a 5 m long leadwire (FLA-5-11-5LT type strain gauge from Tokyo Sokki Kenkyujo Co. Ltd., Japan) was glued at the neck of each bolt. After affixing, the strain gauge and its surrounding were covered with two layers of water repelling coating called M-coat. The M-coat layer was further protected by waxing which was finally covered with an aluminum tape. The tape was extended to cover the entire neck portion of the bolts. The leadwire was protected by passing it above a cushion formed by a duct tape as shown in Figure 2.5. Also, it
was wrapped by a duct tape for a length of approximately 30 cm so that it would neither be cut by the threads of the bolt nor be damaged by getting squeezed in between concrete and a bearing plate.

2.2.4.2 Surface finishing

While the five formed surfaces of a cube specimen were smooth, the finished surface had some high points and some bugholes. The finished surface was left unloaded for the no-stress, uniaxial and biaxial specimens. However, for the triaxial specimens, the surface was smoothened before placing bearing plates on it. The high points were trimmed using a surface grinder. Cement slurry was, then, applied to make the surface smooth and level.

2.2.4.3 Groove cutting

The cube specimens needed grooves in order to pass the leadwires for the strain gauges located underneath the bearing plates. Approximately 5 mm deep grooves were cut in the concrete surface by a circular diamond saw blade. The grooves started at the ends of the sheaths, and were aligned diagonally to the rectangle formed by the four sheaths as shown in Figure 2.7.
2.2.5 Loading of cube specimens

2.2.5.1 Strain monitoring during stress application

A 16-channel data logging system was used for logging the strain in the bolts during loading. Before being used for logging the strain, the data logger was checked for its accuracy by monitoring the strain of several strain gauges for 3 days at a sampling frequency of one reading per minute. The standard deviation of the observed strain was less than 10 microstrain (µm/m), which is equivalent to a fluctuation of ±2 MPa (±0.25%) stress in the high-strength bolt, and thus negligible. The data logger was connected to a computer. The load application was controlled by observing the live strain on a computer monitor.

2.2.5.2 Stress application maneuver

The stress in the specimens was applied at an age of 52 to 56 days. A cube specimen was placed on a flat wooden platform above a concrete floor. The bolts were placed and hand tightened first by carefully bringing the leadwire of the strain gauges underneath the bearing plates. Loading was performed by two people using two wrenches – one on a nut on the either end of a bolt. A torque wrench was used to tighten a nut while the other was used to provide reactive force. It was extremely important to prevent any rotation of the bolt since this would break the strain gauge.

The stress application maneuver took approximately one hour for a specimen with uniaxial stress and approximately five hours for a specimen with triaxial stress. The stress application in the four bolts in each stress direction (X, Y or Z) was performed in three cycles and in one bolt at a time. The first cycle included loading up to about 50% of the target strain, and the second cycle included loading up to about 100% of the target strain. The third cycle comprised of fine adjustment needed to bring the final strain as close to the target strain as possible.

A number of trial stress applications were made before performing the stress application in the actual specimens. A target strain of 4400 microstrain was initially considered. However, the galvanized fasteners were found to possess high friction. Sometimes, frictional force held the applied force for a while and released it abruptly, thus yielding a bolt. Nuts were oiled which helped to reduce the friction during loading. However, some bolts did yield and required replacement. Consequently, new minimum target strains were set as 4200 microstrain for the
12.7 mm bolts, and 4000 microstrain for the 19.0 mm bolts. The average strains that were actually achieved for the 19.0 mm and 12.7 mm bolts were 4280 and 4150 with the coefficients of variation as 2.6% and 3.5%, respectively. (Strain readings in the bolts are presented in Appendix C). The actual average stress in the cube specimens was calculated accordingly as shown in Table 2.4.

2.3 Concrete Stresses

2.3.1 Applied stresses

This study aimed to maintain reasonably constant stress while the concrete specimens underwent ASR. Since ASR can cause expansion of the stressed specimens, the tension in the bolts and also the compressive stress in the concrete specimens increase accordingly. Therefore, the bolts were stressed close to yielding. Once yielded, the stress level in the bolt would remain constant at the yield plateau for large strains in excess of 5000 microstrain (0.5% strain). This strain is far beyond the maximum likely expansion of the concrete used in this study.

The bolts were loaded to approximately 90% of the yield load of the bolts. Table 2.4 shows the loads applied to the two types of bolts and the corresponding stresses applied to the concrete specimens. Even when the bolts yielded, the stress level in the concrete specimens remain within about 110% of the initially applied stress. While shrinkage was not expected for the specimens stored in humid conditions, the expansive strain would compensate for the likely creep strain and any losses due to stress relaxation. Moreover, the control specimens, with no expansive strain, are useful to assess the effect of creep and shrinkage.

2.3.2 Effect of elevated temperature

The stress was applied to the specimens at room temperature (23±3 °C). However, the specimens were conditioned at 50±0.5 °C. Thus, the effect of temperature on the strain of the bolts was monitored by introducing a triaxially stressed room-temperature (23±3 °C) specimen to the 50±0.5 °C chamber. The strain in the bolts decreased by an average of 90 microstrain for four hours, and then remained constant. In order to analyze this observation theoretically, a calculation was made. The coefficient of thermal expansion for grade B7 steel at 50 °C is $10.8 \times 10^{-6}/^\circ{\text{C}}$ [27] and for concrete with limestone aggregate is $7.8 \times 10^{-6}/^\circ{\text{C}}$ [28]. For a difference in temperature of 50-23 = 27 °C, the differential thermal strain is 81 microstrain, a
close match with the observed value of 90 microstrain. The reduction in strain by 90 microstrain is about 2% of the target strain. The stress in concrete will correspondingly be about 2% less than the applied stress.

Outcomes of the proposed loading setup is intended for concrete structures. The stresses in concrete in real structures is often associated with a marked degree of variability. Therefore, the proposed practical setup is expected to be adequate for structural applications.

Table 2.4: Average stresses applied to the cube specimens by prestressing with high-strength bolts

<table>
<thead>
<tr>
<th>Particulars</th>
<th>19.0 mm bolts</th>
<th>12.7 mm bolts</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nominal concrete stress, MPa</td>
<td>10</td>
<td>4</td>
</tr>
<tr>
<td>Side of cube (mm)</td>
<td>254</td>
<td>254</td>
</tr>
<tr>
<td>Gross area of concrete, mm$^2$</td>
<td>64,516</td>
<td>64,516</td>
</tr>
<tr>
<td>Sheath outer diameter, mm</td>
<td>25.4</td>
<td>19.0</td>
</tr>
<tr>
<td>Area of concrete less sheaths, mm$^2$</td>
<td>62,489</td>
<td>63,376</td>
</tr>
<tr>
<td>Net area of a bolt at neck, mm$^2$</td>
<td>180.1</td>
<td>72.4</td>
</tr>
<tr>
<td>Target strain in a bolt, microstrain</td>
<td>4150</td>
<td>4280</td>
</tr>
<tr>
<td>Stress in a bolt, MPa</td>
<td>830.0</td>
<td>856.0</td>
</tr>
<tr>
<td>Force per bolt, kN</td>
<td>149.5</td>
<td>62.0</td>
</tr>
<tr>
<td>Total force in a concrete face, kN</td>
<td>598.0</td>
<td>247.8</td>
</tr>
<tr>
<td>Average stress in concrete, MPa</td>
<td>9.6</td>
<td>3.9</td>
</tr>
</tbody>
</table>

2.4 Results and Discussion of Triaxial Expansions

All cube specimens were measured for expansion (or contraction) in the three directions (X, Y and Z) at different ages. Following the initial measurement, which was taken the day after casting, the specimens were cured at room temperature (23±3 °C) with frequently watered wet burlap covered by a plastic sheet for 6 months (180 days). The specimens were then subjected to the acceleration chamber (maintained at 50±0.5 °C temperature and > 95% relative humidity) to encourage and accelerate ASR. However, prior to taking the expansion measurements, the specimens were removed from the acceleration chamber and acclimatized at room temperature (23±3 °C) for approximately 24 hours. During this acclimatization period, drying of the samples was prevented by covering them with wet burlap and a plastic sheet. The specimens were placed back in the acceleration chamber the next day. Any influence associated with the orientation of
the specimens during storing/conditioning was minimized by changing the orientation of the specimens after each measurement.

Expansion measurements were taken at 1, 28, 52, 57, 120, 210, 235, 270, 315, 360, 430, 485, and 545 days. As stress was applied to the specimens at the age of 52 to 56 days, a pre-loading measurement was taken at 52 days, and a post-loading measurement was taken at 57 days. The expansion of cubes until loading was insignificant for both control and reactive specimens. The control (non-reactive) concrete specimens showed a small average longitudinal expansion of 0.011%; the reactive specimens underwent 0.004% contraction. The maximum observed expansion for the control specimens was 0.037%. The X-, Y- and Z-direction expansions of concrete with reactive aggregate, for all stress states, are presented, respectively, in Figures 2.8, 2.9, and 2.10. (The error bar in these and many other figures in this thesis represents ± one standard deviation). The expansions reached their ultimate values indicating the exhaustion of the reaction during 8 to 12 months of accelerated curing (age of 430 to 545 days). After 3, 8 and 12 months of accelerated curing (at the ages of 270, 430 and 545 days, respectively), one reactive cube specimen from each stress state was unloaded for destructive testing. Therefore, while each data point in Figures 2.8 to 2.10 is an average of three specimens up to 270 days, each data point is an average of two specimens from 315 to 430 days. For 485 and 545 days, each data point represents only one cube specimen.

Figures 2.8 to 2.10 showed some marked differences in expansions among the different stress states. The no-stress specimens showed reasonably similar expansions in the three directions (0.139%, 0.136% and 0.161% in the X-, Y- and Z-directions, respectively). Slightly larger expansion in the Z-direction is attributed to the anisotropy associated with casting direction [29,30,31]. When stress was applied in the X-direction as in uniaxially stressed specimens, the expansion was reduced in the X-direction. The remaining two directions had identical expansions, which were however, greater than the corresponding expansions of the no-stress specimens. The level of reduction of the expansion in the X-direction increased with increased stress level.

When the specimens were restrained biaxially in the X- and Y-directions, the expansion was reduced in both of the stressed directions. Consequently, increased expansion was observed in the stress-free (Z) direction. As shown in Figure 2.10, the expansion in the Z-direction not only
was significantly greater than the free expansion of the no-stress specimen, but also increased with an increased stress level in the X-direction. These results demonstrate a clear trend of expansion transfer from stressed to stress-free directions.

The expansion was markedly reduced for the triaxially stressed specimens. Expansion was reduced in all three directions. A larger reduction in expansion was observed for T (9.6, 3.9, 3.9) specimen than for t (3.9, 3.9, 3.9) specimen.

The expansion results presented in this chapter not only demonstrate that ASR expansion is strongly influenced by the stress state but also provide a basis to formulate an expansion-stress relationship for the three dimensional domain. The expansion results will be analyzed in Chapter 3 with reference to the stress states of the specimens obtained from a numerical analysis.

### 2.4.1 Measurement variation

**Intra-specimen variation**: The average expansion of a specimen along an axis is based on four measurements on the four pair of studs. The COV associated with the expansion measured on the four studs was less than 20% for the reactive specimens along the directions without any stress. For the reactive specimens along the directions with 3.9 MPa stress, a relatively larger COV was observed in the range of 30 to 80%. The relatively larger COV must have been partly contributed by the influence of experimental variation in stress. For the reactive specimens along the direction with 9.6 MPa stress (only the X-direction), and for the control (non-reactive) specimens along all directions, not all the studs showed expansion, some actually showed contraction. Thus, the scatter cannot be described in terms of COV.

**Inter-specimen variation**: The variation in expansion measurements from multiple reactive specimens, inter-specimen variation, was comparable to that reported for the intra-specimen variation. For the reactive specimens along the directions without any stress, the COV was within 20%. For the reactive specimens along the directions with 3.9 MPa stress, the COV was up to 50%. For the reactive specimens along the direction with 9.6 MPa stress, and for the control (non-reactive) specimens along all directions, the change in length was extremely small, and hence, the scatter cannot be described in terms of COV.
Figure 2.8: Longitudinal expansion of reactive cube specimens along X-direction at different ages

Figure 2.9: Longitudinal expansion of reactive cube specimens along Y-direction at different ages
Instrument variation: The variation in measurements should be interpreted by considering the accuracy of the measuring device, which was ±4 μm. The change in length was extremely small for the control specimens along all directions and for the reactive specimens along the direction with 9.6 MPa stress. The all-time measurements for the former ranged from +76 to −39 μm and for the latter ranged from +84 to −65 μm. The COV is not a good representation of the variability due to the presence of negative expansion values (contraction), and/or, owing to the extremely small measurements in comparison to the accuracy of the device. The accuracy of the micrometer was a limitation for measuring the extremely small changes in the length of the cube specimens. A measuring device with greater accuracy could be beneficial. However, for the type and purpose of the study aimed at investigating concrete structures, such an accuracy is neither feasible nor essential.

2.5 Summary and Concluding Remarks

Both public and private institutions are increasingly interested in addressing the deteriorating problems in concrete structures and extending the life of the structures. Expansive reactions, such as ASR, have been recognized as an urgent problem for many important concrete structures,
such as dams, bridges and nuclear power plants. Often, concrete in such structures is subjected to stresses in one or more directions.

With the advancement of computational technology, it is now possible to make a thorough analysis of a structure numerically. However, the crucial input for such an analysis is missing, owing to the lack of proper experimental studies that elucidate the ASR expansion, cracking and degradation of mechanical properties of concrete subjected to multiaxial stress state. Thus, it has become essential to investigate the evolution and consequences of the reaction in more realistic specimens that can simulate the multiaxial stress state of the real concrete structures.

To investigate ASR evolution in a multiaxially loaded concrete specimen, the greatest challenge is perhaps the unavailability of a proper experimental setup. Even though triaxial tests (including polyaxial system in geotechnical engineering) have been practiced for years, no suitable method is available for applying and maintaining triaxial stress in concrete specimens for an extended duration sufficient for the reaction to evolve and exhaust. In this study, a new experimental method was developed for applying and sustaining multiaxial compressive stresses in concrete specimens undergoing ASR until the reaction is exhausted.

The proposed method utilized the post-tensioning method of prestressing to apply stresses in concrete. By stressing high-strength bolts close to yielding, a reasonably constant stress was sustained for a long duration (> 1 year) in the concrete specimens that were conditioned in a hot (50±0.5 °C) and humid (> 95% relative humidity) chamber. Multiple specimens were loaded for uniaxial, biaxial or triaxial stress states. The specimens were conditioned under the accelerated conditions since the age of 6 months and were periodically measured for the triaxial expansions. The ultimate expansion was achieved between 8 to 12 months of accelerated curing.

This chapter presents the expansion results which not only demonstrate that ASR expansion is strongly influenced by the stress state but also provide a basis to formulate an expansion-stress relationship for the three dimensional domain.

Considering the potential interest to replicate a similar setup in future experimental studies, this chapter was devoted to elucidate the new method in detail. The method is customizable, scalable and practical to simulate the practical stress state, and will be useful to perform experimental studies that can advance the understanding about the evolution and consequences of ASR on
concrete structures. Moreover, the setup could inspire similar studies to investigate other expansive problems in concrete, such as the sulfate attack.

Finally, the following suggestions are made for the possible improvements in the future:

• Use of a mechanical torque wrench could prove beneficial in tightening the bolts.

• Galvanization of the fasteners was necessary considering the hot and humid environment that encourages corrosion. However, galvanization increased the friction of the bolts. Non-corrosive grades of steel could be explored to avoid the galvanization process.

• A wide range of stress levels can be considered. Methods should be explored to improve the stress distribution in concrete. Numerical analysis should be performed in order to analyze the stress distribution.

• Compression is the predominant stress type for concrete in most of the concrete structures. Also, the secondary stress due to ASR mostly produces tension in the reinforcement steel and compression in concrete. Thus, the method discussed here should be adequate for most structures. However, a new setup involving other stresses, such as tension, could be considered for future development in the field.

• Protection of strain gauges and their leadwires could be improved. Post-yield type strain gauges could be used. Additional protective measures could be applied for longevity of the strain gauges subjected to a harsh environment. Also, the strain gauges could be monitored continuously, if needed.

• In this study, the grooves were cut in the concrete specimens for passing the leadwires underneath the bearing plates. In the future, an extrusion could be designed in the formwork so that the groove is formed in the concrete surface.

2.6 References


Chapter 3
Expansion–Stress Relationship for Concrete with Alkali–Silica Reaction

Abstract

Many concrete structures around the world are experiencing expansion of concrete due to alkali-silica reaction (ASR). ASR expansion is reduced in the direction of compressive stress and is transferred to the unstressed directions. However, the relationship between ASR expansion and a multiaxial stress state in concrete, which is often the case of concrete structures, is inadequately understood. This chapter presents a relationship between ASR expansion and the stress state of concrete which is based on experimentally determined triaxial expansions of unstressed and uniaxially, biaxially and triaxially stressed concrete cube specimens. It should be noted that the expansion-stress relationship is not coupled with reaction kinetics and is isolated from the effect of creep and shrinkage by subtracting the expansions measured on non-reactive control specimens. The proposed relationship was implemented in a finite element program to predict the measured expansion values on concrete cube specimens. The predicted expansions are in reasonable agreement with the measured expansions.

Keywords: alkali-silica reaction; multiaxial stress; uncoupled axial expansion; maximum possible axial expansion; expansion-stress relationship

3.1 Introduction

Expansion due to alkali-silica reaction (ASR) has caused serious consequences on many concrete structures around the world [1]. ASR is a complicated reaction that is influenced by several factors such as temperature, humidity, alkali content of concrete, type and content of reactive aggregate and stress state. The reaction usually takes many years to develop in concrete structures. Studies on ASR over the past several decades have clarified most of the mechanisms of ASR. However, the effect of stress state on ASR expansion is not yet well understood.

Several studies have examined the influence of stress on the ASR expansion of concrete by maintaining uniaxial compressive stress in concrete that was undergoing ASR [2–6]. Such
studies have revealed that expansion is reduced in the direction of compressive stress and is transferred to the unstressed directions. The phenomena of both reduction and transfer complicate the relationship between the ASR expansion and the stress state. Studies limited to uniaxial stress are inadequate for understanding the relationship between the ASR expansion that occurs volumetrically and the stress state that can be multiaxial.

Attempts have been made to experimentally investigate the effect of multiaxial stress on the ASR expansion of concrete [7–10]. In order to experimentally investigate the effect of multiaxial stress on ASR expansion, Gravel et al. [7] measured triaxial expansion of a cube specimen subjected to biaxial stress. Stress in the range of 3.3 to 6 MPa was shown to prevent expansion in the stressed direction. However, it should be noted that (i) the experiment involved only one specimen, (ii) the specimen had moisture supply only on two of the six surfaces, (iii) the curing temperature was non-uniform ranging from 34 to 45 °C, and (iv) the load was applied after the expansion reached 0.05% while the ultimate expansion in the free direction was less than 0.07%. Multon and Toutlemonde [8] investigated the mechanism of ASR expansion when concrete is subjected to triaxial restraints. Axial stress in cylinder specimens was applied through a load cell and the lateral restraint was provided through passive confinement by steel rings. The authors [8] concluded that “the ASR volumetric imposed strain can be considered as constant whatever the stress state”. In contrast, the work by Gautam and Panesar [9] have reported that the volumetric strains vary with stress state, in particular in the triaxially stressed condition. Nevertheless, the study [8] established that an analysis of ASR-affected concrete structures should consider ASR expansion at the volumetric level and should acknowledge the expansion transfer phenomenon.

Numerical studies have been performed by considering the influence of stress on ASR expansion [11–17]. The reaction kinetics has been thoroughly elucidated [12] and has been coupled into the relationship between ASR expansion and stress state in some analyses [15,17]. While ASR expansion in each direction has been independently assessed in some studies [13,14], the interaction of expansion in three mutually perpendicular directions has also been considered [15,17]. However, “due to lack of sufficient experimental data”, the relationship between expansion and stress had to rely on “engineering common sense” [17]. Moreover, in an attempt to phenomenologically incorporate the reaction kinetics [15,17] and the creep of concrete [15], models have become extremely complex, and several assumptions have to be made in analyzing
concrete structures with such models. In essence, the models lack an experimental basis involving ASR expansion of concrete under multiaxial stress state.

An experimental investigation was performed by Gautam and Panesar [9] that measured triaxial ASR expansion in several unrestrained and restrained concrete specimens subjected to uniaxial, biaxial or triaxial stresses. The experimental results showed: (i) the reduction of ASR expansion due to stress, (ii) the transfer of ASR expansion from stressed to unstressed direction, and (iii) the reduction in volumetric expansion due to triaxial stress. The above three outcomes and observations form the basis for developing a relationship between the ASR expansion and the stress state of concrete. This chapter is focused on the influence of stress state on the expansion of concrete due to ASR, and a relationship is proposed. The reaction kinetics component is decoupled from the effect of stress based on experimental observations. Also, the effect of creep and shrinkage is isolated by subtracting expansion measurements on non-reactive specimens from the measurements on reactive specimens. The proposed relationship is expected to be a simple and relatively more accurate tool for the analysis of ASR-affected concrete structures.

3.2 Research Significance

ASR expansion is influenced by the stress state in concrete. Various models have been proposed to relate the ASR expansion with the stress state. However, the models cannot be substantiated in the absence of an experimental investigation in a three-dimensional stress state. This study proposes an expansion-stress relationship for ASR expansion which is rooted in an experimental study involving stresses in three directions. The relationship has been validated by comparing the experimental results with results from numerical analyses. The relationship provides a simple and reasonably accurate basis for the numerical simulation of ASR-affected concrete structures.

3.3 Experimental Investigation

3.3.1 Concrete specimens

The experimental study involved twenty-eight concrete cube specimens of 254 mm size subjected to multiaxial stresses [9]. Table 3.1 details the seven stress states that include a no-stress condition, uniaxial stress, biaxial stress and triaxial stress. Two concrete mix designs were used: (i) a reactive concrete mix with reactive coarse aggregate and non-reactive fine aggregate, and (ii) a non-reactive control mix with non-reactive coarse and non-reactive fine aggregate. The
concrete mix designs were based on the proportions detailed in ASTM C1293 [18]. The cement content of concrete was 420 kg/m$^3$ and the alkali level of concrete was 5.25 kg/m$^3$ Na$_2$O$_{eq}$. Natural sand from Orillia, Ontario was used as the non-reactive fine aggregate. Spratt aggregate (crushed siliceous limestone) from Stittsville, Ontario was used as the reactive coarse aggregate for the reactive concrete mix. Crushed limestone from Milton, Ontario was used as the non-reactive coarse aggregate for the non-reactive mix. No admixtures or supplementary cementitious materials were used. The water-to-cement ratio of both mix designs was 0.44.

Table 3.1: Average applied compressive stress in concrete cube specimens for the seven stress states

<table>
<thead>
<tr>
<th>Stress state</th>
<th>Designation</th>
<th>Average applied stress, MPa</th>
<th>Number of reactive cube specimens</th>
<th>Number of control cube specimens</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$f_c$</td>
<td>$f_y$</td>
<td>$f_z$</td>
</tr>
<tr>
<td>No-stress</td>
<td>n (0, 0, 0)</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Uniaxial</td>
<td>u (3.9, 0, 0)</td>
<td>3.9</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>U (9.6, 0, 0)</td>
<td>9.6</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Biaxial</td>
<td>b (3.9, 3.9, 0)</td>
<td>3.9</td>
<td>3.9</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>B (9.6, 3.9, 0)</td>
<td>9.6</td>
<td>3.9</td>
<td>0</td>
</tr>
<tr>
<td>Triaxial</td>
<td>t (3.9, 3.9, 3.9)</td>
<td>3.9</td>
<td>3.9</td>
<td>3.9</td>
</tr>
<tr>
<td></td>
<td>T (9.6, 3.9, 3.9)</td>
<td>9.6</td>
<td>3.9</td>
<td>3.9</td>
</tr>
</tbody>
</table>

3.3.2 Loading and conditioning of specimens

Stress application in the cube specimens was performed at an age of 52 to 56 days. The post-tensioning method of prestressing was used to apply and sustain the desired compressive stresses in the concrete specimens, and is detailed in Gautam and Panesar [9]. In any given direction, stress was applied by using four high-strength bolts tightened against a pair of bearing plates for each bolt. Reasonably constant stress state was maintained by stressing the bolts close to yielding. Any likely expansion due to ASR would yield the bolts, and the stress level in the bolt would remain reasonably constant at the yield plateau for large strains in excess of 5000 microstrain (0.5% strain). Figure 3.1a shows the arrangement of multiaxial stress application in a specimen in a 3D view. Figures 3.1b, c and d detail the YZ, XZ and XY planes, respectively, for 3.9 MPa stress each in the X-, Y- and Z-direction. Figure 3.1b also shows the dashed line representing the relatively larger bearing plate for 9.6 MPa stress in the X-direction. Figure 3.2
shows a triaxially stressed T (9.6, 3.9, 3.9) specimen. Since stress was applied through bearing plates by prestressing with high-strength bolts, some stress concentration effect prevailed in the specimens. The stress distribution was obtained through a numerical analysis in this study.

Figure 3.1: Multiaxial stress application arrangement in the cube specimens and studs for expansion measurement (Note: the plates in the XY plane were rotated for making space for performing ultrasonic pulse velocity test)

From the day of demolding to the age of 180 days, the specimens were cured at room temperature (23±3 °C) with frequently watered wet burlap further covered with a plastic sheet.
At the age of 180 days, the specimens were subjected to the acceleration chamber, maintained at 50±0.5 °C temperature and > 95% relative humidity (RH). Figure 3.3 shows a photograph of the specimens stored inside the acceleration chamber.

Figure 3.2: A triaxially stressed T (9.6, 3.9, 3.9) concrete cube specimen (254 mm size)

Figure 3.3: Concrete cube specimens being stored in the acceleration chamber
3.3.3 Expansion measurement

Expansion of the cube specimens was measured in the X-, Y- and Z-directions. In each direction, expansion was taken as the average of four measurements which were taken on four pairs of studs embedded in concrete during casting, as shown in Figure 3.1. For the cube specimen of 254 mm length, the studs were 30 mm long, thus the gauge length for the expansion measurement was 194 mm.

Expansion was measured by using a digital micrometer, with a resolution of 1 µm, and a manufacturer-specified accuracy of ±4 µm. Before taking the measurements of a cube specimen, the micrometer was calibrated to the standard reference invar bar. The initial (reference) micrometer measurements were taken immediately after demolding at the age of one day.

Following the initial measurement, expansion measurements were taken at 28, 52, 57, 120, 210, 235, 270, 315, 360, 430, 485, and 545 days. As stress was applied to the specimens at the age of 52 to 56 days, the 52-day measurement served as a pre-loading measurement and the 57-day measurement served as a post-loading measurement. After subjecting the specimens to accelerated curing (50±0.5 °C and > 95% RH) at the age of 180 days, expansion measurements were performed by taking the specimens out of the chamber and acclimatizing at room temperature for approximately 24 hours.

3.4 Experimental Results and Analysis

Table 3.2 presents the axial expansions of the reactive concrete cube specimens for all stress states. It should be noted that the specimens were introduced to the accelerated curing at 50±0.5 °C beginning 180 days of casting. The results in Table 3.2 are the average of multiple specimens for a given stress state. Each expansion value in Table 3.2 is an average of three specimens up to 270 days. Each expansion value is an average of two specimens from 315 to 430 days. Between 485 and 545 days, each expansion value represents only one cube specimen.

Table 3.3 presents the axial expansions (or contractions if negative) of the non-reactive concrete cube specimens for all stress states. The maximum axial expansion for the no-stress non-reactive specimen was 0.016% at 545 days. The maximum observed expansion for the non-reactive specimens was 0.037%. Only the Z-direction of the B (9.6, 3.9, 0) specimen and the Y- and Z-directions of the U (9.6, 0, 0) specimen showed a relatively larger expansion of the non-reactive
specimens in the range of 0.03 to 0.037%. This must have been contributed by the stress in the other directions. Moreover, when comparing the 3.9 MPa stress to the 9.6 MPa stress (for uniaxial, biaxial and triaxial non-reactive specimens) in the X-direction, the specimens contracted along the X-direction with an increase in stress (from u (3.9, 0, 0) to U (9.6, 0, 0), from b (3.9, 3.9, 0) to B (9.6, 3.9, 0), and from t (3.9, 3.9, 3.9) to T (9.6, 3.9, 3.9) specimens). The average contraction at 545 days was 0.003% for the u (3.9, 0, 0), b (3.9, 3.9, 0) and t (3.9, 3.9, 3.9) specimens and was 0.014% for the U (9.6, 0, 0), B (9.6, 3.9, 0) and T (9.6, 3.9, 3.9) specimens. These contractions reflect the effect of stress.

The effect of stress, other than on ASR expansion, was assumed identical for the reactive and the non-reactive specimens for a given direction for a given stress state. Moreover, the effect of creep and shrinkage (and swelling other than ASR expansion) was assumed similar for the reactive and the non-reactive specimens for a given direction for a given stress state. Therefore, the expansion measurement of the non-reactive specimens was subtracted from the corresponding measurement of the reactive specimen to obtain the net ASR expansion as presented in Table 3.4. As can be seen in Table 3.2 and 3.3, the stress application, during 52 to 56 days of age, caused some elastic shortening in the cube specimens in the direction of the applied stress. Also, the ASR expansion until the age of stress application was insignificant as shown in Table 3.2. In order to exclude the effect of elastic shortening on the measure of ASR expansion, the net ASR expansion can thus be considered with reference to the post-loading measurement at 57 days as shown in Table 3.5. A scenario of triaxial expansions for all the stress states at the age of 545 days (180 days at 23±3 °C plus 365 days at 50±0.5 °C) is presented in Figure 3.4.
Table 3.2: As measured axial expansion (%) of the reactive specimens

<table>
<thead>
<tr>
<th>Axis</th>
<th>Age (days)</th>
<th>28</th>
<th>52</th>
<th>57</th>
<th>120</th>
<th>210</th>
<th>235</th>
<th>270</th>
<th>315</th>
<th>360</th>
<th>430</th>
<th>485</th>
<th>545</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>52</td>
<td>57</td>
<td>120</td>
<td>210</td>
<td>235</td>
<td>270</td>
<td>315</td>
<td>360</td>
<td>430</td>
<td>485</td>
<td>545</td>
<td></td>
</tr>
<tr>
<td>Stress state ↓</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>n (0, 0, 0)</td>
<td>-0.004</td>
<td>-0.005</td>
<td>-0.005</td>
<td>0.002</td>
<td>0.054</td>
<td>0.083</td>
<td>0.110</td>
<td>0.114</td>
<td>0.132</td>
<td>0.140</td>
<td>0.140</td>
<td>0.139</td>
<td></td>
</tr>
<tr>
<td>u (3.9, 0, 0)</td>
<td>-0.001</td>
<td>-0.006</td>
<td>-0.005</td>
<td>-0.002</td>
<td>0.028</td>
<td>0.044</td>
<td>0.062</td>
<td>0.077</td>
<td>0.092</td>
<td>0.093</td>
<td>0.098</td>
<td>0.098</td>
<td></td>
</tr>
<tr>
<td>U (9.6, 0, 0)</td>
<td>-0.009</td>
<td>-0.010</td>
<td>-0.028</td>
<td>-0.027</td>
<td>-0.015</td>
<td>-0.009</td>
<td>0.000</td>
<td>-0.003</td>
<td>0.004</td>
<td>0.004</td>
<td>0.008</td>
<td>0.009</td>
<td></td>
</tr>
<tr>
<td>b (3.9, 3.9, 0)</td>
<td>0.000</td>
<td>0.001</td>
<td>-0.002</td>
<td>0.003</td>
<td>0.037</td>
<td>0.057</td>
<td>0.072</td>
<td>0.083</td>
<td>0.091</td>
<td>0.095</td>
<td>0.095</td>
<td>0.095</td>
<td></td>
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<tr>
<td>B (9.6, 3.9, 0)</td>
<td>0.001</td>
<td>0.001</td>
<td>-0.016</td>
<td>-0.017</td>
<td>0.007</td>
<td>0.010</td>
<td>0.018</td>
<td>0.022</td>
<td>0.028</td>
<td>0.031</td>
<td>0.024</td>
<td>0.027</td>
<td></td>
</tr>
<tr>
<td>t (3.9, 3.9, 3.9)</td>
<td>-0.004</td>
<td>-0.003</td>
<td>-0.010</td>
<td>-0.002</td>
<td>0.018</td>
<td>0.038</td>
<td>0.063</td>
<td>0.079</td>
<td>0.095</td>
<td>0.101</td>
<td>0.098</td>
<td>0.099</td>
<td></td>
</tr>
<tr>
<td>T (9.6, 3.9, 3.9)</td>
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<td>-0.006</td>
<td>-0.021</td>
<td>-0.020</td>
<td>-0.016</td>
<td>-0.010</td>
<td>-0.003</td>
<td>-0.002</td>
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<td>0.010</td>
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<tr>
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<td>-0.011</td>
<td>-0.011</td>
<td>0.000</td>
<td>0.048</td>
<td>0.074</td>
<td>0.099</td>
<td>0.109</td>
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<td>-0.005</td>
<td>0.009</td>
<td>0.061</td>
<td>0.092</td>
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<td>-0.007</td>
<td>0.000</td>
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<td>0.074</td>
<td>0.105</td>
<td>0.140</td>
<td>0.160</td>
<td>0.181</td>
<td>0.189</td>
<td>0.195</td>
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<tr>
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<td>-0.001</td>
<td>-0.007</td>
<td>-0.007</td>
<td>0.009</td>
<td>0.018</td>
<td>0.028</td>
<td>0.031</td>
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<tr>
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<td>0.000</td>
<td>-0.007</td>
<td>-0.007</td>
<td>0.015</td>
<td>0.019</td>
<td>0.029</td>
<td>0.032</td>
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<td>-0.006</td>
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<td>-0.008</td>
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<td>0.003</td>
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Table 3.3: As measured axial expansion (%) of the non-reactive specimens

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**Table 3.4:** Net ASR axial expansion (%) obtained after subtracting the measurement of the non-reactive specimens from that of the reactive specimens

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<td>0.023</td>
<td>0.028</td>
<td>0.037</td>
<td>0.031</td>
<td>0.029</td>
</tr>
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</table>
Table 3.5: Net ASR axial expansion (%) with reference taken as the measurement immediately after stress application

<table>
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<tr>
<th>Axis</th>
<th>Age (days)</th>
<th>57</th>
<th>120</th>
<th>210</th>
<th>235</th>
<th>270</th>
<th>315</th>
<th>360</th>
<th>430</th>
<th>485</th>
<th>545</th>
</tr>
</thead>
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<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td></td>
<td>Stress state ↓</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>X</td>
<td>n (0, 0, 0)</td>
<td>0.000</td>
<td>0.004</td>
<td>0.046</td>
<td>0.085</td>
<td>0.115</td>
<td>0.118</td>
<td>0.130</td>
<td>0.139</td>
<td>0.141</td>
<td>0.140</td>
</tr>
<tr>
<td></td>
<td>u (3.9, 0, 0)</td>
<td>0.000</td>
<td>0.011</td>
<td>0.043</td>
<td>0.057</td>
<td>0.078</td>
<td>0.092</td>
<td>0.111</td>
<td>0.111</td>
<td>0.112</td>
<td>0.114</td>
</tr>
<tr>
<td></td>
<td>U (9.6, 0, 0)</td>
<td>0.000</td>
<td>0.006</td>
<td>0.017</td>
<td>0.028</td>
<td>0.039</td>
<td>0.039</td>
<td>0.043</td>
<td>0.043</td>
<td>0.048</td>
<td>0.050</td>
</tr>
<tr>
<td></td>
<td>b (3.9, 3.9, 0)</td>
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<td>0.006</td>
<td>0.032</td>
<td>0.062</td>
<td>0.079</td>
<td>0.090</td>
<td>0.101</td>
<td>0.103</td>
<td>0.102</td>
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<td>B (9.6, 3.9, 0)</td>
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<td>0.005</td>
<td>0.023</td>
<td>0.039</td>
<td>0.053</td>
<td>0.057</td>
<td>0.061</td>
<td>0.065</td>
<td>0.059</td>
<td>0.061</td>
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<td>0.016</td>
<td>0.045</td>
<td>0.071</td>
<td>0.089</td>
<td>0.098</td>
<td>0.113</td>
<td>0.110</td>
<td>0.109</td>
</tr>
<tr>
<td></td>
<td>T (9.6, 3.9, 3.9)</td>
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<td>0.001</td>
<td>0.020</td>
<td>0.031</td>
<td>0.036</td>
<td>0.038</td>
<td>0.045</td>
<td>0.038</td>
<td>0.035</td>
</tr>
<tr>
<td>Y</td>
<td>n (0, 0, 0)</td>
<td>0.000</td>
<td>0.011</td>
<td>0.047</td>
<td>0.084</td>
<td>0.109</td>
<td>0.122</td>
<td>0.132</td>
<td>0.140</td>
<td>0.144</td>
<td>0.144</td>
</tr>
<tr>
<td></td>
<td>u (3.9, 0, 0)</td>
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<td>0.014</td>
<td>0.067</td>
<td>0.098</td>
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<td>0.182</td>
<td>0.186</td>
<td>0.190</td>
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<td>0.011</td>
<td>0.072</td>
<td>0.101</td>
<td>0.136</td>
<td>0.154</td>
<td>0.174</td>
<td>0.179</td>
<td>0.185</td>
<td>0.189</td>
</tr>
<tr>
<td></td>
<td>b (3.9, 3.9, 0)</td>
<td>0.000</td>
<td>0.005</td>
<td>0.013</td>
<td>0.037</td>
<td>0.051</td>
<td>0.053</td>
<td>0.056</td>
<td>0.057</td>
<td>0.055</td>
<td>0.057</td>
</tr>
<tr>
<td></td>
<td>B (9.6, 3.9, 0)</td>
<td>0.000</td>
<td>0.005</td>
<td>0.014</td>
<td>0.029</td>
<td>0.043</td>
<td>0.045</td>
<td>0.048</td>
<td>0.054</td>
<td>0.050</td>
<td>0.050</td>
</tr>
<tr>
<td></td>
<td>t (3.9, 3.9, 3.9)</td>
<td>0.000</td>
<td>0.006</td>
<td>0.003</td>
<td>0.021</td>
<td>0.039</td>
<td>0.042</td>
<td>0.048</td>
<td>0.058</td>
<td>0.052</td>
<td>0.053</td>
</tr>
<tr>
<td></td>
<td>T (9.6, 3.9, 3.9)</td>
<td>0.000</td>
<td>0.007</td>
<td>0.004</td>
<td>0.024</td>
<td>0.038</td>
<td>0.042</td>
<td>0.044</td>
<td>0.056</td>
<td>0.052</td>
<td>0.049</td>
</tr>
<tr>
<td>Z</td>
<td>n (0, 0, 0)</td>
<td>0.000</td>
<td>0.008</td>
<td>0.048</td>
<td>0.089</td>
<td>0.119</td>
<td>0.136</td>
<td>0.150</td>
<td>0.158</td>
<td>0.162</td>
<td>0.162</td>
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<tr>
<td></td>
<td>u (3.9, 0, 0)</td>
<td>0.000</td>
<td>0.013</td>
<td>0.069</td>
<td>0.100</td>
<td>0.139</td>
<td>0.166</td>
<td>0.195</td>
<td>0.201</td>
<td>0.204</td>
<td>0.205</td>
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<td></td>
<td>U (9.6, 0, 0)</td>
<td>0.000</td>
<td>0.017</td>
<td>0.078</td>
<td>0.107</td>
<td>0.143</td>
<td>0.167</td>
<td>0.189</td>
<td>0.195</td>
<td>0.203</td>
<td>0.207</td>
</tr>
<tr>
<td></td>
<td>b (3.9, 3.9, 0)</td>
<td>0.000</td>
<td>0.013</td>
<td>0.072</td>
<td>0.135</td>
<td>0.181</td>
<td>0.214</td>
<td>0.236</td>
<td>0.250</td>
<td>0.249</td>
<td>0.251</td>
</tr>
<tr>
<td></td>
<td>B (9.6, 3.9, 0)</td>
<td>0.000</td>
<td>0.005</td>
<td>0.105</td>
<td>0.165</td>
<td>0.221</td>
<td>0.263</td>
<td>0.294</td>
<td>0.314</td>
<td>0.314</td>
<td>0.314</td>
</tr>
<tr>
<td></td>
<td>t (3.9, 3.9, 3.9)</td>
<td>0.000</td>
<td>0.006</td>
<td>0.000</td>
<td>0.015</td>
<td>0.030</td>
<td>0.042</td>
<td>0.045</td>
<td>0.055</td>
<td>0.053</td>
<td>0.052</td>
</tr>
<tr>
<td></td>
<td>T (9.6, 3.9, 3.9)</td>
<td>0.000</td>
<td>0.008</td>
<td>0.003</td>
<td>0.025</td>
<td>0.037</td>
<td>0.048</td>
<td>0.053</td>
<td>0.062</td>
<td>0.056</td>
<td>0.054</td>
</tr>
</tbody>
</table>
The results in Figure 3.4 show that ASR axial expansion is reduced due to compressive stress and is transferred to free directions. For the specimens with uniaxial stress, the expansion was reduced in the X-direction, and the Y- and Z-directions had approximately identical expansions, which were however, greater than the corresponding Y- and Z-direction expansions of the no-stress specimens. For biaxially stressed specimens, the expansions were reduced in both of the stressed directions (X and Y), and significantly greater expansion was observed in the unstressed (Z) direction. Moreover, comparison of the Z-direction (free direction) expansions of the biaxially stressed specimens b (3.9, 3.9, 0) and B (9.6, 3.9, 0) shows that the Z-direction expansion increased with an increase in the stress level in the X-direction. For the triaxially stressed specimens, the axial expansions were markedly reduced in all three directions, resulting in lower volumetric expansion compared to the no-stress, uniaxial and biaxial stress conditions.

3.4.1 Evolution of ASR expansion

The net ASR axial expansion results presented in Table 3.5 were utilized to calculate volumetric ASR expansion of the reactive specimens. The ratio of the volumetric expansion of concrete for each stress state to the volumetric expansion of concrete for no-stress state is presented in Figure 3.5. The ratio for the triaxially stressed specimens was relatively smaller for the first month of
accelerated curing (until an age of 210 days) than for the successive ages. Except that, the ratio of volumetric expansion remained fairly constant with age for all stress states. Volumetric expansion for uniaxially stressed specimens was mostly 90 to 118% of the volumetric expansion of the no-stress specimens. For biaxially stressed specimens, the volumetric expansion ranged from 90 to 99% of the volumetric expansion of no-stress specimens. However, the volumetric expansion was markedly reduced for the triaxially stressed specimens by approximately 50 to 70%. Based on these observations, this study assumes that volumetric expansion due to ASR is conserved as long as there is at least one unstressed direction. This assumption is consistent with the understanding that ASR expansion is caused by the fluid pressure of the reaction products [5,12,15,19–21].

![Graph showing volumetric expansion ratio](image)

**Figure 3.5:** Ratio of volumetric expansion for different stress states to the volumetric expansion for no-stress state at different ages (Note “x” represents the stress state n, u, U, b, B, t, or T)

Concrete samples were cut from the cube specimens and were examined microscopically after 56, 270, 430 and 545 days of casting. Figure 3.6 shows a digital microscope image of a polished surface of concrete from the t (3.9, 3.9, 3.9) specimen after 270 days of casting (90 days of accelerated curing). As exemplified in Figure 3.6, reactive concrete specimens from all stress states experienced extensive ASR cracking after 90 days of accelerated curing. This illustrates
that even though expansion was reduced, the reaction was not suppressed by the triaxial stress. Ichikawa and Miura [19] have shown that the expansive pressure generated by ASR is approximately 400 MPa. Thus, the stress state in concrete structures (approximately 2 orders of magnitude smaller) is unlikely to suppress the chemical reaction but can reduce ASR expansion. This mechanism can be convincingly explained by the theory of pressure-dependent filling of ASR gel in concrete pores [5]. Therefore, an increase in stress causes ASR gel to penetrate into the increasingly smaller micro-pores in concrete. Even though the authors [5] proposed the theory even for the case of uniaxial stress, the experimental results in this study showed that the theory is applicable for the case of triaxial confinement. In the case of uniaxial or biaxial confinement, as the reaction product can preferentially flow towards the unstressed direction, the pressure gradient is perhaps inadequate to make it penetrate into the micro-pores.

Figure 3.6: ASR cracks on a polished surface of concrete from the t (3.9, 3.9, 3.9) specimen after 90 days of accelerated curing

Figure 3.7 shows the evolution of volumetric expansion with age for the seven stress states. The volumetric expansion is plotted as a ratio of its value at an age to that at 545 days. The evolution of volumetric expansion with age appears identical irrespective of the stress state with the exception of two observations. First, the evolution of volumetric expansion in the triaxially stressed specimens was delayed by approximately one month. And second, the triaxially stressed
specimens experienced some contraction after the peak expansion at 430 days. The evolution in volumetric expansion in Figure 3.7 indicates a characteristic sigmoidal curve for ASR expansion as proposed by Larive et al. [4,12]. This observation suggests that the effect of stress state on reaction kinetics can be practically ignored. Thus, this paper postulates that reaction kinetics can be decoupled from the effect of stress state.

Figure 3.7: Evolution of volumetric expansion with age for the seven stress states (Note: the volumetric expansion is plotted as the ratio of its value at an age “x” days to that at 545 days)

3.4.2 Stress distribution and its effect on expansion

Table 3.6 presents the net ASR axial expansion of cubes along X-, Y- and Z-directions as the percentage of volumetric expansion of the no-stress specimens at corresponding ages. The data is shown for the age range of 315 to 545 days, in which the ratio of volumetric expansion for a stress state to that for no-stress state remained relatively constant as shown in Figure 3.5.

As shown in Table 3.6, for an average stress of 9.6 MPa in the X-direction, the X-direction expansions of the U (9.6, 0, 0), B (9.6, 3.9, 0) and T (9.6, 3.9, 3.9) specimens are comparable from 315 to 545 days. For an average stress of 3.9 MPa in the Y-direction, the Y-direction
expansions of the b (3.9, 3.9, 0) and B (9.6, 3.9, 0) specimens are approximately similar. Similarly, for an average stress of 3.9 MPa in the Y- and Z-directions, the Y- and Z-direction expansions of the t (3.9, 3.9, 3.9) and T (9.6, 3.9, 3.9) specimens are approximately identical. For an average stress of 3.9 MPa in the X-direction, the X-direction expansions of the u (3.9, 0, 0), b (3.9, 3.9, 0) and t (3.9, 3.9, 3.9) specimens are approximately identical; however, they are approximately double than the Y-direction expansions of the b (3.9, 3.9, 0) and B (9.6, 3.9, 0) specimens with an average stress of 3.9 MPa in the Y-direction, and of the Y- and Z-direction expansions of the t (3.9, 3.9, 3.9) and T (9.6, 3.9, 3.9) specimens with an average stress of 3.9 MPa each in the Y- and Z-directions.

Table 3.6: Net ASR axial expansion as percentage of volumetric expansion of no-stress specimens

<table>
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<tr>
<th>Axis</th>
<th>Age (days) →</th>
<th>315</th>
<th>360</th>
<th>430</th>
<th>485</th>
<th>545</th>
</tr>
</thead>
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<tr>
<td></td>
<td>Stress state ↓</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>X</td>
<td>n (0, 0, 0)</td>
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<td>31.6</td>
<td>31.7</td>
<td>31.6</td>
<td>31.3</td>
</tr>
<tr>
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<td>26.9</td>
<td>25.3</td>
<td>25.1</td>
<td>25.6</td>
</tr>
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<td>10.5</td>
<td>9.9</td>
<td>10.7</td>
<td>11.3</td>
</tr>
<tr>
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<td>23.2</td>
<td>22.9</td>
<td>22.8</td>
</tr>
<tr>
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<td>14.9</td>
<td>14.9</td>
<td>13.2</td>
<td>13.7</td>
</tr>
<tr>
<td></td>
<td>t (3.9, 3.9, 3.9)</td>
<td>23.6</td>
<td>23.8</td>
<td>25.9</td>
<td>24.6</td>
<td>24.5</td>
</tr>
<tr>
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<td>9.3</td>
<td>10.4</td>
<td>8.6</td>
<td>7.9</td>
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<tr>
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<td>32.1</td>
<td>32.1</td>
<td>32.2</td>
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<td>42.7</td>
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<td>41.0</td>
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<td>13.0</td>
<td>12.3</td>
<td>12.7</td>
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<td>11.7</td>
<td>12.4</td>
<td>11.2</td>
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<tr>
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<td>t (3.9, 3.9, 3.9)</td>
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<td>11.5</td>
<td>13.2</td>
<td>11.7</td>
<td>11.9</td>
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<tr>
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<td>10.8</td>
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<td>11.6</td>
<td>10.9</td>
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<td>36.3</td>
<td>36.2</td>
<td>36.2</td>
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<td>45.9</td>
<td>44.7</td>
<td>45.5</td>
<td>46.5</td>
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<td>57.3</td>
<td>57.2</td>
<td>55.7</td>
<td>56.3</td>
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<td>71.3</td>
<td>71.8</td>
<td>70.3</td>
<td>70.4</td>
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<td>11.0</td>
<td>12.6</td>
<td>11.8</td>
<td>11.6</td>
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<td>T (9.6, 3.9, 3.9)</td>
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<td>12.7</td>
<td>14.2</td>
<td>12.5</td>
<td>12.0</td>
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</table>
The relatively larger expansion in the X-direction of the u (3.9, 0, 0), b (3.9, 3.9, 0) and t (3.9, 3.9, 3.9) specimens is attributed to the non-uniform stress distribution along the X-direction. Since the expansion measurement studs in the X-direction were relatively farther from the bearing plates as shown in Figure 3.1, the measured expansion at the studs was contributed by both the uniformly stressed core concrete and the low-stressed near-surface concrete. The stress distribution along the axes of expansion measurement studs of the u (3.9, 0, 0), b (3.9, 3.9, 0) and t (3.9, 3.9, 3.9) specimens were obtained by a finite element analysis performed in VecTor3 program. (A preliminary analysis was performed in ANSYS as discussed in Appendix D). VecTor3 is a non-linear finite element analysis software developed at the University of Toronto for the analysis of three dimensional reinforced concrete structures [22].

Based on VecTor3 analysis, Figure 3.8 shows the distribution of normal stress in the X-direction (\(f_x\)) on the two planes XY and XZ of the t (3.9, 3.9, 3.9) specimen passing through the axis of the expansion measurement stud “X1” (X1 is one of the four expansion measurement studs in the X-direction as shown in Figure 3.1b). The stress concentration caused by the bearing plates is evident. Figure 3.9 shows a plot of the stress distribution in the X-direction along the stud axis (gauge length) for the u (3.9, 0, 0), b (3.9, 3.9, 0) and t (3.9, 3.9, 3.9) specimens. The average stress in the X-direction along the stud axis was 2.6 MPa, 2.2 MPa and 1.7 MPa, respectively, for the u (3.9, 0, 0), b (3.9, 3.9, 0) and t (3.9, 3.9, 3.9) specimens. Lower stress was observed from u (3.9, 0, 0) to b (3.9, 3.9, 0) to t (3.9, 3.9, 3.9) specimens. This was due to the stress in the Y-direction in the b (3.9, 3.9, 0) specimen and due to the stresses in the Y- and Z-directions in the t (3.9, 3.9, 3.9) specimen. In addition to the stress in the X-direction, stresses along the stud axes in the Y- and Z-directions of the b (3.9, 3.9, 0) and t (3.9, 3.9, 3.9) specimens were also analyzed. The average stress in the Y-direction along the stud axis of the b (3.9, 3.9, 0) and t (3.9, 3.9, 3.9) specimens was 2.9 MPa and the average stress in the Z-direction along the stud axis of the t (3.9, 3.9, 3.9) specimen was approximately 3.1 MPa.
a) Section along XY plane

b) Section along XZ plane

Figure 3.8: Distribution of the normal stress $f_x$ (MPa) in the one-eighth of the t (3.9, 3.9, 3.9) specimen in the XY and XZ planes containing the expansion measurement stud “X1” in the X-direction.

Figure 3.9: Stress distribution along the stud axis in the X-direction for the u (3.9, 0, 0), b (3.9, 3.9, 0) and t (3.9, 3.9, 3.9) specimens
3.5 Expansion-Stress Relationship

Based on the experimental results and the stress analysis of the specimens as presented in Section 3.4, a relationship between expansion and stress is proposed in this study. Since ASR expansion can transfer from one direction to another depending on the stress state, ASR expansion in a direction is governed not only by the stress in the given direction but also by the stresses in the other two directions. Thus, a given direction can have a range of ASR expansion corresponding to a free volumetric expansion. The range is defined as $[\epsilon_{un}^{e}, \epsilon_{\text{max}}^{e}]$ where $\epsilon_{un}^{e}$ represents the “uncoupled axial expansion” in the direction and $\epsilon_{\text{max}}^{e}$ represents the “maximum possible axial expansion” in the direction. (It should be noted that the term “uncoupled” here means the axial expansion is not coupled with the stresses in the other directions).

Fairly identical expansions in the X-direction for the u (3.9, 0, 0), b (3.9, 3.9, 0) and t (3.9, 3.9, 3.9) specimens indicate that no additional expansion due to transfer is possible (no coupling of expansion in the X-direction with stresses in the Y- and Z-directions) for the level of stress in the X-direction in the u (3.9, 0, 0), b (3.9, 3.9, 0) and t (3.9, 3.9, 3.9) specimens. Thus, additional expansion in a given direction due to transfer of expansion from other directions is assumed to be possible only for no stress or a relatively lower stress (<1.7 MPa, as obtained in Section 3.4.2) in the direction.

3.5.1 Uncoupled axial expansion

A slightly greater Z-direction expansion than the X- and Y-direction expansions in the no-stress specimen was attributed to the anisotropy associated with casting direction [23,24,25] and was ignored for modeling purposes. Thus, the uncoupled axial expansion for no-stress specimen was considered as one-third of the volumetric expansion of no-stress specimen. The volumetric expansion of the no-stress specimen is hereafter referred to as free volumetric expansion. The expansion in the X-direction of the u (3.9, 0, 0), b (3.9, 3.9, 0) and t (3.9, 3.9, 3.9) specimens was identical as shown in Table 3.6, and the average value was 24.46% of the free volumetric expansion. This expansion was attributed to the 1.7 MPa stress, as obtained in Section 3.4.2. The expansion in the Y-direction of the b (3.9, 3.9, 0), B (9.6, 3.9, 0), t (3.9, 3.9, 3.9) and T (9.6, 3.9, 3.9) specimens and the expansion in the Z-direction of the t (3.9, 3.9, 3.9) and T (9.6, 3.9, 3.9) specimens, all have an average stress of 3.0 MPa, was comparable, and the average uncoupled
axial expansion, as obtained from Table 3.6, was 12.12\% of the free volumetric expansion. The expansion in the X-direction of the U (9.6, 0, 0), B (9.6, 3.9, 0) and T (9.6, 3.9, 3.9) specimens, which have an average stress of 9.6 MPa in the X-direction, was comparable, and the average uncoupled axial expansion, as obtained from Table 3.6, was 11.36\% of the free volumetric expansion. Hence, the experiment provided four points for the expansion-stress curve, namely, four stresses (0, 1.7, 3.0 and 9.6 MPa) with corresponding uncoupled axial expansions as (33.33, 24.46, 12.12 and 11.36) percentage of the free volumetric expansion. The change in expansion corresponding to the stresses of 3.0 MPa and 9.6 MPa was extremely small. On the other hand, the difference in expansion corresponding to the stresses of 1.7 MPa and 3.0 MPa was quite large. These observations indicate that the expansion-stress curve has a reverse S-shaped pattern in which ASR expansion is extremely sensitive to stress in a range around 1.7 MPa and is less sensitive for stresses greater than that. A relatively smaller stress, in the range of 0.3 MPa, has been considered as insignificant to reduce any ASR expansion [3,14]. Moreover, a uniaxial test by Kagimoto et al. [2] indicated that the expansive pressure generated by ASR gel is stabilized at a stress level of approximately 2.6 MPa. Based on the experimental results and considering the literature observations [2,3], curve fitting was performed to obtain a relationship between the stress and expansion in a given direction. The model that best suited the data in this study was a four-parameter logistic curve as below:

\[
\begin{align*}
\text{pe}_{\text{un}} = a + \frac{b - a}{1 + \left( \frac{f}{c} \right)^d} \quad \text{for } f \geq 0
\end{align*}
\]  

(3.1)

where \( \text{pe}_{\text{un}} \) is the uncoupled axial expansion in a given direction expressed as the percentage of free volumetric expansion; \( a, b, c \) and \( d \) are constants of curve-fitting; and \( f \) is the stress in the given direction where compression is positive. The constants \( a, b, c \) and \( d \) were obtained to be 11.360, 33.333, 1.804 and 6.546, respectively. Eq. (3.1) indicates that an axial expansion of 11.36\% of the free volumetric expansion will occur despite a large compressive stress. This observation should, however, be interpreted considering the experimental setup in this study.
3.5.2 Maximum possible axial expansion

For the range of stress from 0 to 1.7 MPa, ASR expansion in a given direction is influenced by the stresses in the other directions. Thus, the expansion can have a range from an uncoupled axial expansion to the maximum possible axial expansion. Since volumetric expansion is assumed to conserve for a biaxial stress state, the axial expansion in the unstressed direction in a restrained specimen can be substantially larger than in a no-stress specimen. Based on Eq. (3.1) for a biaxial stress state, considering the two stressed directions will have an expansion of at least 11.36% of the free volumetric expansion, the maximum possible axial expansion in the free direction can be 77.3% of the free volumetric expansion. This value was considered as the maximum possible axial expansion in the free direction. The maximum possible axial expansion was considered the same as the uncoupled axial expansion for stresses exceeding 1.7 MPa. No experimental expansion data was available corresponding to stress levels between 0 and 1.7 MPa. Therefore, a logistic function was assumed for the maximum possible axial expansion as for the uncoupled axial expansion. Thus, a four-point logistic curve was fitted for four points, namely, four stresses (0, 1.7, 3.0 and 9.6 MPa) with corresponding maximum possible axial expansions as (77.30, 24.46, 12.12 and 11.36) percentage of the free volumetric expansion. The equation for the maximum possible axial expansion \( (P_{e_{\text{max}}}) \), expressed as the percentage of free volumetric expansion, is given as,

\[
P_{e_{\text{max}}} = A \cdot \frac{B - A}{D + f(C)} \quad \text{for } f \geq 0
\]

\[
(3.2)
\]

where constants \( A, B, C, \) and \( D \) are 11.360, 77.300, 1.312, and 5.379, respectively.

The uncoupled and the maximum possible axial expansions as obtained from Eq. (3.1) and (3.2), respectively, are plotted in Figure 3.10. Since the two curves overlap for stress exceeding 1.7 MPa, Eq. (3.2) need only be used for stress range from 0 to 1.7 MPa.

The stress states involved in the experimental study were only compressive. Thus, the scenario of ASR expansion for tensile stress needs further research. For this study, ASR expansion in a direction with tensile stress was considered same as for unstressed direction.
3.5.3 Expansion distribution along the three directions

If the sum of maximum possible axial expansion in three directions is less than 100% (i.e. equal to the free volumetric expansion), the expansion in each direction will be equal to the maximum possible axial expansion in the respective direction. If the sum of maximum possible axial expansion in the three directions is greater than the free volumetric expansion, the volumetric expansion will not be reduced and will be limited to the free volumetric expansion. The difference between 100% and the sum of the uncoupled axial expansion in the three directions needs to be distributed along the three directions. The relative weights of the distribution will depend on the stress state in each direction. The ASR expansion that needs to be distributed is proposed to be distributed proportional to the difference in the maximum possible axial expansion and the uncoupled axial expansion ($P_{\text{max}} - P_{\text{un}}$) for each direction. The main steps for implementing the above-mentioned expansion-stress relationship in a numerical program are:

- Step 1: Obtain three principal stresses. Assume compression as positive and tension as negative.
• Step 2: Consider the free volumetric expansion for the structure being analyzed for the state considered in the numerical analysis.

• Step 3: Calculate the uncoupled axial expansion ($p_{\text{un}}$) in each direction using Eq. (3.1).

• Step 4: Calculate the maximum possible axial expansion ($p_{\text{max}}$) in each direction using Eq. (3.2).

• Step 5: Calculate the sum of maximum possible axial expansions ($p_{\text{max}}$) in three directions and determine the expansion to be redistributed, if applicable.

• Step 6: For the volumetric expansion that needs to be distributed, distribute in the three directions in the proportion of the difference between the maximum possible axial expansion and the uncoupled axial expansion ($p_{\text{max}} - p_{\text{un}}$) in each direction.

• Step 7: Calculate the axial expansion in each direction by adding the uncoupled axial expansion and the distributed component of expansion in the same direction.

3.6 Validation

The proposed expansion-stress relationship was implemented in the finite element analysis program VecTor3 by Jurcut [22]. ASR expansion in VecTor3 program is modeled as elastic strain offsets such that the total strain at a point is the summation of i) the net stress-induced strain; ii) the elastic strain offset due to mechanisms such as thermal expansion, shrinkage, and ASR expansion; iii) the plastic concrete strain offsets due to cyclic loading or material nonlinearity such as creep; and iv) the strains due to shear slip along the crack [22]. The ASR induced strain, which can depend on the stress state of concrete, is evaluated following an iterative procedure [22]. Selected cube specimens from the experiment were analyzed in this study to validate the proposed expansion-stress relationship. Before performing an actual analysis, trial analyses were performed on simple one-element and multi-element models in VecTor3 to confirm that the expansion-stress relationship implemented in the program behaved reasonably.

Figure 3.11 shows a typical model of the t (3.9, 3.9, 3.9) specimen in VecTor3. The dark color represents the steel plate, the light color represents concrete and the white color represents a dummy material with extremely low strength and stiffness. Because of symmetry, only one-
eighth of the cube specimen was modeled. Brick elements were used to model concrete and steel plate. Concrete was modeled with fourteen elements along a direction. Based on the results from a test of concrete cylinders and cores, compressive strength, modulus of elasticity, and Poisson’s ratio of concrete were given as 44.2 MPa, 29 GPa, and 0.2, respectively. Steel plates were modeled with two elements through the thickness, and the load was applied on top of the bearing plates distributed at eight nodes, approximately representing the washer used in the experiment. Due to a simplification in modeling, a bearing plate had a slight discrepancy in its position (maximum 4 mm) and orientation relative to the experimental configuration. The experiment involved twelve friction-less steel sheaths which were also not modeled in VecTor3 program.

The proposed expansion-stress relationship was validated against the experimental results on the u (3.9, 0, 0), b (3.9, 3.9, 0) and t (3.9, 3.9, 3.9) specimens. Figure 3.12 compares the numerical results with the experimental results for the u (3.9, 0, 0), b (3.9, 3.9, 0) and t (3.9, 3.9, 3.9) specimens. Since the experimental results are net ASR expansion, the numerical results for ASR expansion also consider net ASR expansion. For this, two sets of numerical analysis were performed for each case: one with ASR expansion and the other without ASR expansion. Figure 3.12 shows that the results from the numerical analysis are in good agreement with the experimental results. Moreover, Figure 3.12 also compares the numerical results obtained using Saouma and Perotti [17] model for an identical free volumetric expansion. The results from the model presented in this study are markedly closer to the experimental results for the uniaxial, biaxial and triaxial stress states compared to those from Saouma and Perotti [17] model.
3.7 Discussion

- The proposed empirical model was validated against the experimental measurements for the cube specimens having multiaxial stress states. A small discrepancy might have occurred due to the simplifications made in the numerical analysis namely, no consideration of sheath and a slight dislocation of bearing plate relative to the experimental case. However, the experimental results were simulated with reasonable accuracy.

- Based on the proposed model, a stress level of 1.49 MPa corresponds to an uncoupled axial expansion of 28.4% and the maximum possible axial expansion of 33.33% of the free volumetric expansion. Thus, based on the model, a hydrostatic stress state lower than 1.5 MPa cannot reduce volumetric ASR expansion.

- Based on the experimental observation, the present model decouples the effect of stress on reaction kinetics. This consideration greatly simplifies the simulation of ASR-affected concrete structures. The present model can be utilized for a given free volumetric expansion and should be adequate for most structural applications. However, if the evolution of structural performance is to be examined, the present model may still be
utilized by giving the free volumetric expansion in several increments and by considering the stress state at each increment.

- ASR is a long-term process and so is creep. Thus, the effect of creep is an important consideration in the analysis of concrete structures. While ASR has been coupled with creep by Grimal et al. [15], the proposed model decouples ASR from creep simply based on experiments involving non-reactive concrete. More complicated experimental investigation to decouple the ASR and creep strains in reactive concrete can be a topic of future research interest.

- The proposed model indicates that a minimum axial expansion of 11.36% of the free volumetric expansion will occur despite a large compressive stress. This appears unreasonable for a relatively larger compressive stress. However, for the typical stress level in concrete structures, which is mostly below 10 MPa, this level of expansion may not be unreasonable and is conservative. A small amount of ASR expansion in concrete structures might be compensated by elastic shortening, creep and shrinkage strain due to compressive stress.

- The degradation of concrete properties due to ASR has not been incorporated in the proposed expansion-stress relationship. Appropriate degradation parameters corresponding to the free volumetric expansion and the stress state of concrete can be specified in an analysis program, as required.

- Even though the experiment showed the expansion transfer effect in the uniaxial and biaxial specimens, the number of cases were still insufficient to clearly define transfer as a function of stress state. Further studies are recommended to focus on the stress level, particularly in the range of 0.5 to 2 MPa.

- The ASR expansion transfers from stressed direction to a relatively less stressed direction. The expansion transfer mechanism as observed in the experimental study revealed a tendency approximately analogous to the expansion of fluid. When the fluid inside a relatively rigid container expands, the expansion is directed almost entirely towards the free direction with no pressure development inside the container. In case of ASR in concrete, except for triaxial stresses, ASR expansion was thus conserved volumetrically. However, when the container with expanding fluid is sealed, the fluid will develop a pressure inside the container. Depending on the stiffness of the container
and the seal, some expansion may occur. In the case of ASR in concrete with triaxial stresses, some pressure would be developed and expansion would be reduced volumetrically. The pressure development would, however, be limited to a certain extent, which is mostly attributed to the consumption of pressure in concrete pores. Thus, any attempts to interpret ASR expansion based on the concept of the first invariant of the stress tensor is not valid. For instance, the first invariant of the stress tensor for the B (9.6, 3.9, 0) specimen is greater than that for the t (3.9, 3.9, 3.9) specimen. However, the volumetric expansion of B (9.6, 3.9, 0) was practically not reduced but the volumetric expansion of the t (3.9, 3.9, 3.9) specimens was reduced by 50%.

- The arrangement for stress application in the experiment caused non-uniform stress distribution in the cube specimens. However, the stress distribution was obtained by a numerical analysis, and the stress condition helped in formulating and validating the expansion-stress relationship.

### 3.8 Conclusion

A relationship between ASR expansion and stress state was proposed based on an experimental study that measured the ASR expansion in the three mutually perpendicular directions in a number of unrestrained and restrained concrete cube specimens subjected to uniaxial, biaxial or triaxial stresses. The experimental results showed that the reaction kinetics is not coupled with the expansion-stress relationship. Therefore, the proposed empirical relationship is not coupled with the reaction kinetics. The relationship is simple to implement in a numerical analysis program. It was implemented in a finite element analysis software, VecTor3. The relationship was validated with the experimental results in cube specimens with uniaxial, biaxial and triaxial stress states. Rooted in an experimental investigation involving the measurement of ASR expansion for the case of multiaxial stress states, this model is expected to be an accurate basis for reliably estimating the ASR expansion of concrete structures in multiple directions.

### 3.9 References


Chapter 4
The Effect of Multiaxial Stresses on the ASR Damage of Concrete

Abstract
Alkali-silica reaction (ASR) causes expansion, cracking and degradation of the mechanical properties of concrete. While ASR performance of unrestrained concrete specimens is relatively well understood, the ASR performance of concrete structures is complicated by co-acting stresses. This chapter investigates the influence of sustained multiaxial stresses on the response of concrete undergoing ASR. Reactive concrete cube specimens (with three replicates) were subjected to different stress states from no-stress to triaxial stress and were subjected to accelerated curing until the exhaustion of the reaction. The specimens were periodically core-drilled and the cores were tested for compressive strength and modulus of elasticity. Stress state influenced the degradation of mechanical properties and the ASR-affected concrete behaved as an orthotropic material. ASR cracking was portrayed along three mutually perpendicular planes of multiaxially loaded concrete by performing damage rating index analysis. Surface cracking was also monitored. Triaxial confinement contributed to having reduced volumetric expansion and less cracking.

Keywords: alkali-silica reaction; multiaxial stresses; damage rating index; expansion; mechanical properties

4.1 Introduction
Alkali-silica reaction (ASR) is a serious deterioration problem in many concrete structures around the world [1]. The reaction causes expansion, cracking and degradation of mechanical properties of concrete. Although generally it is now possible to construct new structures safe against ASR, the greatest challenge remains to ensure the performance of existing ASR-affected structures by assessing ASR-damage.

Damage assessment of ASR-affected concrete usually relies on measuring expansion, monitoring cracking, and testing mechanical properties. Measurements on unrestrained specimens have established expansion as a characteristic feature of ASR. However, concrete in structures is often
subjected to stresses in one or multiple directions, and hence, expansion measured on unrestrained laboratory specimens does not appropriately reflect the concrete behavior in structures. ASR expansion is the external manifestation of the damage occurring inside the concrete and is thus largely influenced by the stress state of concrete (e.g. [2–5]). Apart from the expansion measurement, an understanding of the cracking and the degradation of mechanical properties is essential in evaluating the performance of ASR-affected concrete structures.

Cracks have been monitored as a means of assessing damage of existing concrete structures. Surface cracks have been mapped to assess the ASR damage experienced by concrete structures, sometimes in terms of “cracking index” [6,7]. However, the cracking index method involves monitoring cracks along the four sides and two diagonals of a square area on a concrete surface, and does not differentiate the cracks by their orientation, which can be influenced by the stress state of concrete [8]. Another method to assess the damage due to ASR by monitoring cracking is the damage rating index (DRI). The DRI method was developed in the 1990s as a means of quantitatively assessing the damage of the concrete microstructure due to ASR. The DRI is rooted on cracking that is characteristic of ASR damage and it accounts for seven damage features. For concrete structures, the DRI method can be applied to concrete samples taken from the structures even without a reference measurement. While the DRI method has been extensively applied to laboratory concrete specimens [9–11] and to the cores from existing structures [12], the influence of stress state on damage has not been studied. Grattan-Bellew [12] observed widely varying surface cracking and DRI values of cores from different parts of an ASR-affected dam and viewed the variation as “difficult to explain”. While variation in the reactivity of aggregate and moisture content were indicated as possible causes, it is possible that the influence of stress state might also have a role in those variations. Stress state may cause spatial and directional variations in the cracking and the DRI values of concrete in the ASR-affected structures.

Testing mechanical properties is another method of assessing ASR damage. Many studies have reported on the degradation of mechanical properties of concrete due to ASR [13–17]. However, such studies mostly test unrestrained specimens that do not truly represent the ASR damage of structural concrete having a variety of stresses, as indicated by Giaccio et al. [18]. Degradation of mechanical properties in the context of concrete structures is sometimes assessed by testing cores taken from ASR-affected concrete structures [12,19,20]. Such tests have been the basis of the
“stiffness damage test” that was proposed as a tool to evaluate the ASR damage [21,22]. However, the focus of such studies was to relate stiffness damage test results to the ASR expansion. Even though testing of over 1000 cores from several ASR-affected structures was reported [23], the influence of stress-state on the evolution and consequence of ASR has been inadequately acknowledged. Since ASR damage can be influenced by the concrete’s stress state, the information provided by two cores at the same location but along two directions represent two widely different scenarios. Large variations have been observed in the mechanical properties of cores from reinforced concrete members taken parallel and perpendicular to the longitudinal reinforcement [19,24]. The prestressing effect produced by the reinforcing steel due to the expansive pressure from ASR was attributed for such directional variations [20,24]. However, the stress level and its relationship to the degradation of mechanical properties of concrete due to ASR has not been explored. Since the effect of restraint is essentially through the prestressing stress it applies on concrete, a clear understanding of the effect of stress state on the degradation of mechanical properties of concrete due to ASR can help in reliably evaluating the performance of ASR-affected concrete structures.

This study is focused on the damage of ASR concrete subjected to multiaxial stresses. Twenty-one reactive cube specimens with three replicates of seven stress states from no-stress to triaxial stress and an additional reference specimen were subjected to high temperature (50±0.5 °C), high humidity (> 95% relative humidity) accelerated curing until the exhaustion of the reaction. The specimens were periodically measured for triaxial expansion from the day of demolding to the exhaustion of the reaction [2]. To further understand the evolution and consequences of ASR, the cube specimens were periodically core-drilled, and the cores were tested for compressive strength and modulus of elasticity. In addition, degradation of mechanical properties was studied through a number of companion cylinders. The cubes were analyzed for the DRI along three mutually perpendicular planes. This chapter reports the results from such tests. Also reported is the surface cracking of cube specimens.

4.2 Research Significance

ASR damage of concrete is a serious problem in existing structures. While concrete in structures is subjected to various stress states, the mechanism of ASR has been mostly investigated for unrestrained concrete specimens. This study investigates the influence of uniaxial, biaxial and
triaxial stresses on the ASR-affected concrete by analyzing triaxial expansion, cracking along three planes, and degradation of the mechanical properties along stressed and unstressed directions. By integrating the understanding of triaxial expansion, cracking, and degradation of mechanical properties, this study is anticipated to expand the material level understanding of ASR to the structural response of ASR-affected concrete structures.

4.3 Experimental Investigation

4.3.1 Concrete ingredients and mix design

The concrete mix design was based on the proportions detailed in ASTM C1293 [25]. High-alkali (0.99% Na$_2$O equivalent by mass of cement) general use cement was used in the concrete mix. The cement content of the mix was 420 kg/m$^3$. The chemical composition of cement used in this study is shown in Table 4.1. The alkali level of the concrete was boosted to 5.25% kg/m$^3$ Na$_2$O equivalent by dissolving NaOH pellets in the mixing water. Spratt aggregate from Stittsville, Ontario, Canada was used as the reactive coarse aggregate. Natural sand from Orillia, Ontario, Canada was used as the non-reactive fine aggregate. No admixtures or supplementary cementitious materials were used. The water-to-cement ratio of the mix was 0.44. A similar but non-reactive mix was used to cast control cylinder specimens. The non-reactive mix was designed by substituting the reactive siliceous limestone coarse aggregate by non-reactive limestone coarse aggregate.

4.3.2 Casting of cube specimens

In total, twenty-two concrete cube specimens of 254 mm size were cast. Seven stress states were considered from no-stress to triaxial stress as shown in Table 4.2; all stresses are in compression. For each stress state, three replicates were cast so that each set could also undergo destructive testing at three different ages of accelerated curing. One specimen was cast as a reference to be destructively tested before stress application. The cube specimens were demolded after one day of casting.

Overall, the cube specimens were cast from nine batches of the same concrete mix design. The average 28-day strength for the nine batches was 43.4 MPa with the standard deviation of 2.9 MPa. The average 28-day strength of the non-reactive mixtures was 49.4 MPa with the standard deviation of 2.5 MPa.
Table 4.1: Chemical composition of cement

<table>
<thead>
<tr>
<th>Constituents</th>
<th>LOI</th>
<th>SiO₂</th>
<th>Al₂O₃</th>
<th>Fe₂O₃</th>
<th>CaO</th>
<th>MgO</th>
<th>SO₃</th>
<th>Alkali (Na₂O eq)</th>
<th>Free Lime</th>
<th>Insoluble residue</th>
</tr>
</thead>
<tbody>
<tr>
<td>Percentage</td>
<td>2.27</td>
<td>19.25</td>
<td>5.33</td>
<td>2.41</td>
<td>62.78</td>
<td>2.36</td>
<td>4.01</td>
<td>0.99</td>
<td>1.29</td>
<td>0.52</td>
</tr>
</tbody>
</table>

Table 4.2: Seven stress states in the concrete cube specimens

<table>
<thead>
<tr>
<th>Stress state</th>
<th>Designation</th>
<th>$f_x$</th>
<th>$f_y$</th>
<th>$f_z$</th>
<th>Number of cube specimens</th>
</tr>
</thead>
<tbody>
<tr>
<td>No-stress</td>
<td>n</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>3+1</td>
</tr>
<tr>
<td>Uniaxial</td>
<td>u</td>
<td>3.9</td>
<td>0</td>
<td>0</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>U</td>
<td>9.6</td>
<td>0</td>
<td>0</td>
<td>3</td>
</tr>
<tr>
<td>Biaxial</td>
<td>b</td>
<td>3.9</td>
<td>3.9</td>
<td>0</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>9.6</td>
<td>3.9</td>
<td>0</td>
<td>3</td>
</tr>
<tr>
<td>Triaxial</td>
<td>t</td>
<td>3.9</td>
<td>3.9</td>
<td>3.9</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>T</td>
<td>9.6</td>
<td>3.9</td>
<td>3.9</td>
<td>3</td>
</tr>
</tbody>
</table>

4.3.3 Loading and conditioning of specimens

Stress application in the cube specimens was performed at an age of 52 to 56 days. The post-tensioning method of prestressing was chosen as the technique to apply and sustain the compressive stresses in the concrete specimens, and is detailed in Gautam and Panesar [2]. In any given direction, stress was applied by using four high-strength bolts tightened against a pair of bearing plates for each bolt. The bolts had a yield strength in excess of 900 MPa. Reasonably constant stress state was maintained by stressing the bolts close to yielding. Any likely expansion due to ASR would yield the bolts and the stress level in the bolt would remain reasonably constant at the yield plateau for large strains in excess of 5000 microstrain (0.5% strain).

After the stress application, the cube specimens were stored on carts, and air space was maintained for moisture circulation on all the surfaces. Figure 4.1 shows a cart with the cube specimens. Any influence associated with the orientation of the specimens during storing/conditioning was minimized by changing the orientation of the specimens after each measurement. From the day of demolding to the age of 6 months, the specimens were cured at room temperature ($23\pm3$ °C) with frequently watered wet burlap further covered with a plastic sheet. At the age of 6 months, the specimens were relocated to the acceleration chamber,
maintained at 50±0.5 °C temperature and > 95% relative humidity (RH). The specimens were periodically measured for expansion in the three directions from one day to 18 months of casting. All the measurements on specimens being cured at 50±0.5 °C and > 95% RH were taken only after removing the carts out from the chamber and acclimatizing the specimens at room temperature for approximately 24 hours.

Figure 4.1: Cube specimens with different stress states being stored in a cart

4.3.4 Core testing

Cube specimens were core drilled by using a 75 mm diameter diamond drill bit cooled by water circulation. To avoid moisture loss, the drilled cores were wrapped with cling-film. Each core was trimmed to discard a length of approximately 35 mm on either end. The remaining cylindrical piece was cut to a length of 150 mm, and the ends were surface ground. The core was affixed with concrete strain gauges (surface remained air-dried for approximately two hours during this process) and was tested for compressive strength as per ASTM C39 [26] and static modulus of elasticity as per ASTM C469 [27].

Core drilling was performed at four stages of ASR based on expansion measurements, namely, the first stage at 2 months of casting, the second stage at 3 months of accelerated curing (9 months after casting) when ASR was actively taking place, the third stage at 8 months of accelerated curing when the reaction was nearing to the completion, and the fourth stage at 12 months of accelerated curing when the reaction had exhausted. In each of the latter three stages,
one cube specimen from each of the seven stress states was unloaded and core drilled. However, this test regime was modified in the case of the biaxial specimens. The coring of the biaxial specimens from the second stage was postponed until the third stage so that two identical cubes were available at the third stage to simultaneously acquire in-situ properties of one core along the X-direction and another core along the Z-direction. It was intended to minimize the time between unloading the cube specimens and core testing. The duration between unloading the cube specimens and coring was limited to 48 hours at most. Then, the cores were always tested the next day of coring. It should be noted that the specimens were measured for any change in length due to stress release during unloading for coring. Expansion due to stress release was up to 0.025 mm (0.013%) along the stressed direction for the 9.6 MPa stress that had caused 0.035 mm (0.018%) contraction during loading.

4.3.5 Damage rating index (DRI)

The portion of each cube specimen after core drilling was dissected for DRI analysis along the three mutually perpendicular planes as shown in Figure 4.2. The DRI method as outlined by Villeneuve et al. [28] was adopted in this study. Further reference was made to the guidelines published by Fournier et al. [29].

Figure 4.2: Dissection of a no-stress cube specimen (254 mm size) along the three mutually perpendicular planes (YZ, ZX and XY) for DRI analysis and core drilled along the Z-axis
Slices from the cube specimens, with area ranging from 90 to 250 cm², were polished to a surface roughness of 1500 grit and were divided into 1 cm² grids as shown in Figure 4.3. Each grid was observed in a stereo binocular microscope to count the number of seven petrographic features listed in Table 4.3. The number of counts were multiplied by the respective weighting factors and then summed. The sum was normalized to an area of 100 cm² to provide the DRI value for each plane. The DRI value for a cube specimen was obtained as the average for the three planes.

Table 4.3: Features counted in an individual grid as per the DRI method [28]

<table>
<thead>
<tr>
<th>Petrographic features</th>
<th>Weighting factors</th>
</tr>
</thead>
<tbody>
<tr>
<td>Closed/tight cracks in coarse aggregate particle</td>
<td>0.25</td>
</tr>
<tr>
<td>Opened cracks or network cracks in coarse aggregate particle</td>
<td>2</td>
</tr>
<tr>
<td>Cracks or network cracks with reaction product in coarse aggregate particle</td>
<td>2</td>
</tr>
<tr>
<td>Debonded coarse aggregate</td>
<td>3</td>
</tr>
<tr>
<td>Disaggregated / corroded aggregate particle</td>
<td>2</td>
</tr>
<tr>
<td>Cracks in cement paste</td>
<td>3</td>
</tr>
<tr>
<td>Cracks with reaction product in cement paste</td>
<td>3</td>
</tr>
</tbody>
</table>

Figure 4.3: DRI slice from a biaxial specimen along the XY plane at 8 months of accelerated curing (grid size is 1 cm by 1 cm; length is 254 mm)
4.4 Experimental Results and Discussion

4.4.1 Expansion

The evolution of volumetric expansion for the cube specimens over an 18-month duration is presented in Figure 4.4. (The error bar in this and many other figures in this thesis represents ± one standard deviation). The rate of expansion with time was identical for the specimens except for the triaxially stressed specimens. However, the triaxially stressed specimens had a relatively lower rate of expansion until about 4.5 months of accelerated curing and a relatively higher rate of expansion between 4.5 to 8 months of accelerated curing (10.5 to 14 months of age). This indicated some delay in expansion contributed by the triaxial confinement. However, for all stress states, the ultimate expansion was achieved between 8 to 12 months of accelerated curing.

The ultimate longitudinal and volumetric expansions for the various stress states are presented in Figure 4.5. The specimens underwent elastic shortening because of the stress application. This was indicated by the two sets of expansion measurements taken just before and after loading the specimens at an age of 8 weeks, as seen in Figure 4.4. In order to exclude the elastic strain, the expansion due to ASR as shown in Figure 4.5, was considered with reference to the post-loading measurement. A strong influence of stress state was observed on the ASR expansion, which was also reported previously [2]. Expansion in a given direction was reduced in the presence of a stress applied in the same direction. Expansion was transferred from the stressed to the unstressed directions. In general, a larger stress has a larger influence on the extent of reduction of the expansion and also on the transfer of ASR expansion.

As shown in Figures 4.4 and 4.5, the volumetric expansion due to ASR remained approximately constant irrespective of the stress state with the exception of the triaxial stress state. The volumetric expansion was markedly reduced by 50 to 70% for the triaxially stressed specimens. These observations indicate that the volumetric expansion due to ASR is conserved as long as there is at least one unstressed direction. Accordingly, longitudinal expansion, measured by the concrete prism test for example, may be un-conservative if regarded as the extent of expansion of concrete in a structure. For instance, a biaxially stressed concrete structure may show a relatively smaller expansion along the stressed directions; however, a relatively larger expansion, far greater than the linear expansion, might occur in the out-of-plane direction.
Figure 4.4: Evolution of volumetric expansion of the cube specimens with different stress states

Figure 4.5: Longitudinal and volumetric expansion (X+Y+Z) at 12 months of accelerated curing
4.4.2 Cracking

4.4.2.1 DRI along the three planes for different stress states

The DRI values for the reference cube specimen tested at the age of 2 months were 74, 67 and 67, respectively, along YZ, ZX and XY planes. Table 4.4 presents the DRI values for the cube specimens corresponding to the seven stress states at 3, 8 and 12 months of accelerated curing. DRI values increased with age for all the stress states, with an increase of 30 to 40% from 3 to 12 months of accelerated curing.

Even though the extent of damage varied for different stress states, the reaction was not prevented by the stresses. As shown in Table 4.4, the DRI values for the triaxially stressed specimens t (3.9, 3.9, 3.9) and T (9.6, 3.9, 3.9), respectively, are about 78% and 64% of the DRI value for the no-stress specimen at the same age. Figure 4.6 shows the contributions of the seven damage features to the DRI values of the specimens for all the stress states at 12 months of accelerated curing. All the damage features of DRI were abundantly observed in the specimens of all the stress states.

Figure 4.6 shows the variation of DRI values along the three mutually perpendicular planes YZ, ZX and XY. The DRI value for a given plane was sensitive to the stress state in the plane. Stress reduced the opening of cracks along the direction of stress, which was reflected in the DRI results. (Attempts were made to visualize ASR cracks through ultraviolet imaging of the polished DRI slices by vacuum impregnating with epoxy containing a fluorescent dye. However, the method was not effective as discussed in Appendix E). Crack opening along a particular direction generally correlated with the axial expansion in that direction. Accordingly, in a cube specimen, the DRI value was generally the greatest in the plane that contained directions with the largest expansions, and vice versa. Hence, the DRI value in the YZ plane of the uniaxially stressed specimens was considerably larger than the DRI values in the ZX and XY planes. For the biaxially stressed specimens, the DRI value in the XY plane was considerably smaller than in the YZ and ZX planes. While the YZ and ZX planes had one direction (Z) free to expand, the XY plane had stresses in both X- and Y-directions. Thus, XY was the preferential plane for the smallest damage (cracking) in the biaxially stressed specimens.
Stress in ASR-affected concrete was effective to reduce the cracking of the mortar matrix, referred to as the paste cracking in the DRI method. As shown in Figure 4.6, the contribution of paste cracks (both the “cracks in cement paste” and the “filled cracks in cement paste”) to the DRI value was markedly small in the XY plane of biaxially stressed b (3.9, 3.9, 0) and B (9.6, 3.9, 0) specimens and in all three planes of the triaxially stressed t (3.9, 3.9, 3.9) and T (9.6, 3.9, 3.9) specimens. While the combined contribution of the “cracks in cement paste” and the “filled cracks in cement paste” was approximately 15 to 20% for the no-stress, uniaxial and biaxial stress conditions, its contribution for the triaxial stress condition was less than 15% for t (3.9, 3.9, 3.9) and less than 8% for T (9.6, 3.9, 3.9) specimens. Thus, while the stresses are unlikely to prevent the reaction, they can still be effective in reducing the ASR damage of concrete.

Figure 4.6: Contributions of the seven damage features to the DRI along the three planes of the cube specimens at 12 months of accelerated curing
Table 4.4: DRI results for the cube specimens

<table>
<thead>
<tr>
<th>Stress state</th>
<th>Plane for DRI analysis</th>
<th>3 months of accelerated curing</th>
<th>8 months of accelerated curing</th>
<th>12 months of accelerated curing</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>DRI value (plane)</td>
<td>Average DRI value (specimen)</td>
<td>Ratio with n (0, 0, 0)</td>
</tr>
<tr>
<td>n (0, 0, 0)</td>
<td>YZ</td>
<td>677</td>
<td>630</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>ZX</td>
<td>604</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>XY</td>
<td>608</td>
<td></td>
<td></td>
</tr>
<tr>
<td>u (3.9, 0, 0)</td>
<td>YZ</td>
<td>812</td>
<td>696</td>
<td>1.1</td>
</tr>
<tr>
<td></td>
<td>ZX</td>
<td>572</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>XY</td>
<td>704</td>
<td></td>
<td></td>
</tr>
<tr>
<td>U (9.6, 0, 0)</td>
<td>YZ</td>
<td>820</td>
<td>686</td>
<td>1.09</td>
</tr>
<tr>
<td></td>
<td>ZX</td>
<td>575</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>XY</td>
<td>664</td>
<td></td>
<td></td>
</tr>
<tr>
<td>b (3.9, 3.9, 0)</td>
<td>YZ</td>
<td>-</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>ZX</td>
<td>-</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>XY</td>
<td>-</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B (9.6, 3.9, 0)</td>
<td>YZ</td>
<td>-</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>ZX</td>
<td>-</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>XY</td>
<td>-</td>
<td></td>
<td></td>
</tr>
<tr>
<td>t (3.9, 3.9, 3.9)</td>
<td>YZ</td>
<td>504</td>
<td>493</td>
<td>0.78</td>
</tr>
<tr>
<td></td>
<td>ZX</td>
<td>493</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>XY</td>
<td>481</td>
<td></td>
<td></td>
</tr>
<tr>
<td>T (9.6, 3.9, 3.9)</td>
<td>YZ</td>
<td>526</td>
<td>407</td>
<td>0.65</td>
</tr>
<tr>
<td></td>
<td>ZX</td>
<td>327</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>XY</td>
<td>369</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

4.4.2.2 Expansion-DRI relationship

In general, for all stress states, the DRI value of the cube specimen was proportional to the volumetric expansion as shown in Figure 4.7. However, a DRI value of approximately 70 had been observed at the age of 2 months even before ASR expansion had occurred. The DRI versus volumetric expansion data fit well to a power equation with $R^2=0.904$. By taking into account the DRI value that was observed even before the ASR expansion had occurred, the curve indicates that a significant level of microscopic damage can occur inside concrete before any macroscopic expansion is evident.
Axial (longitudinal) expansion of 0.04% is often accepted as a safe expansion [25] that can be accommodated by concrete without undergoing visible surface cracking. However, Figure 4.7 shows that a DRI value of 500 is possible corresponding to a volumetric expansion of 0.12%, which is equivalent to a uniform linear expansion of 0.04% in all three directions. Since ASR expansion can transfer from one to another direction depending on the stress state, the interpretation of ASR expansion and damage should consider the consequences in all three directions.

\[ \text{DRI} = 5273.44 \times (\text{Vol. Exp})^{0.3503} \]

\[ R^2 = 0.904 \]

![Graph showing relationship between DRI and volumetric expansion](image)

Figure 4.7: Average DRI versus volumetric expansion for all cube specimens with different stress states and at different test ages, and a curve fitting

### 4.4.2.3 Surface cracking

Figure 4.8 shows four photographs of the selected cube surfaces exhibiting surface cracking at 12 months of accelerated curing. Even though the cracks were generally fine with a maximum width of approximately 0.5 mm, the photographs reveal the influence of stress states on surface cracking. Figure 4.8a shows that the no-stress n (0, 0, 0) specimens exhibited a map cracking pattern, a characteristic of ASR in concrete structures. Figure 4.8b shows cracking along the centrelines of the cube surface of the uniaxially stressed u (3.9, 0, 0) specimen. This was the characteristic cracking pattern in the YZ plane of the uniaxially stressed specimens. Since stress was along the X-direction, larger expansion was observed in the Y- and Z-directions.
Accordingly, the cracks were clearly oriented such that the crack openings were in the Y- and Z-directions.

A similar effect of stress in aligning the cracks was further exhibited by the cracking patterns in the biaxially stressed specimens. Figure 4.8c shows a central crack covering the entire surface in the YZ plane of the b (3.9, 3.9, 0) specimen. The crack occurred along the Y-direction such that the crack opening was directed along the unrestrained Z-direction. Figure 4.8d shows the photograph of the XY plane of the biaxially stressed B (9.6, 3.9, 0) specimen. Relatively finer cracks were aligned along the X-direction. Even though both X- and Y-directions were restrained, the stress in the Y-direction was smaller than that in the X-direction. Thus, the cracks were aligned along the X-direction.

Figure 4.8: Cracks on some of the cube specimen (254 mm size) surfaces at 12 months of accelerated curing
4.4.3 Effect of ASR on mechanical properties

4.4.3.1 Core test results

The compressive strength results of the cores at 2 months of casting and at 3, 8 and 12 months of accelerated curing are presented in Figure 4.9. (Data of core testing results are presented in Appendix F). Each data point in the figure represents the result of one core sample. The figure compares the test results along the X-direction (the direction of maximum stress) for all the specimens. Stress along the X-direction appears to improve the compressive strength even though a clear relationship of compressive strength with stress state could not be observed. Compressive strength of cores generally increased with age for all the stress states and was unable to reflect the damage due to ASR. The evolution of compressive strength is further discussed in the next section in relation to the cylinder test results.

The static modulus of elasticity of the cores at 2 months of casting and at 3, 8 and 12 months of accelerated curing is presented in Figure 4.10. Each data point in the figure represents the result of one core sample. The figure compares the test results along the X-direction of the specimens. After 8 months of accelerated curing, the modulus of elasticity for the no-stress specimen dropped by a maximum of 27% of the value at 2 months of casting. However, stress along the X-direction alleviated the loss in the modulus of elasticity. The maximum degradation was limited to 15% when a stress of 3.9 MPa was present along the X-direction. Biaxial and triaxial stresses further alleviated the loss in the modulus of elasticity. These observations suggest that the stress state could be a benefit to many concrete structures in resisting any likely loss in the modulus of elasticity due to ASR. The evolution of static modulus of elasticity is further discussed in the next section in relation to the cylinder test results.

Figure 4.11 compares the modulus of elasticity in the biaxially stressed specimens along the direction of maximum stress (X) and along the direction of no-stress (Z) tested after 8 months of accelerated curing. The static modulus of elasticity was larger in the X-direction than in the Z-direction by 16% and 25%, respectively, for the b (3.9, 3.9, 0) and B (9.6, 3.9, 0) specimens. This suggests that the crack closure effect (for the cores in the Z-direction) contributes to reduce the static modulus of elasticity. Stress turned the ASR-affected concrete into an orthotropic material.
Figure 4.9: Compressive strength of the cores taken along the X-direction from the cube specimens

Figure 4.10: Static modulus of elasticity of the cores taken along the X-direction from the cube specimens
4.4.3.2 Cylinder test results

Control (non-reactive) cylinders and reactive cylinders, cast from the same concrete batches as of the cube specimens, were tested for compressive strength, modulus of elasticity, and splitting tensile strength. The results are presented in Figures 4.12 to 4.14. Compressive strength increased for both the reactive and non-reactive cylinders from 1 month of casting to 12 months of accelerated curing (18 months of casting). The increase was approximately 24% for reactive and 58% for the non-reactive cylinders. The reactive cylinders had visible network of random cracks as illustrated by Giaccio et al. [18]. The relatively smaller increase in the compressive strength of the reactive specimens compared to the non-reactive specimens reflects the combined effects of ASR and hydration reaction. Curing at 50±0.5 °C and > 95% relative humidity promoted the continued hydration of concrete, which offset the damage caused by ASR.

With accelerated curing, the modulus of elasticity of the non-reactive concrete increased by 5-8% compared to the value at 2 months of casting. However, a loss was observed in the modulus of elasticity of the reactive concrete. The maximum loss of 23% (compared to the value at 2 months of casting) was observed after having 3 months of accelerated curing. The level of
degradation was comparable to that from the core test (27%). Partial recovery was observed in the modulus of elasticity of the reactive cylinders after undergoing the maximum degradation at the age of 3 months of accelerated curing. The modulus of elasticity of the cores as presented in Figure 4.10 also indicate partial recovery, particularly from 8 to 12 months of accelerated curing. The mechanism of partial recovery is discussed in detail in Chapter 7.

Figure 4.12: Compressive strength of the non-reactive and reactive cylinders

Figure 4.13: Static modulus of elasticity of the non-reactive and reactive cylinders
While the splitting tensile strength (Figure 4.14) for the non-reactive specimens increased by 93%, it was degraded by a maximum of 53% for the reactive specimens, before a partial recovery was observed.

In addition to the test of cylinders and cores for investigating the degradation of mechanical properties due to ASR, the reactive and control cube specimens were measured for the ultrasonic pulse velocity (UPV) along the three directions. However, UPV was not sensitive to the ASR damage. The UPV results are presented in Appendix G.

4.5 Summary and Conclusions

This study investigated the expansion, cracking, and degradation of the mechanical properties of ASR-affected concrete subjected to seven different compressive stress states, namely no-stress and two variations each of uniaxial, biaxial and triaxial stresses. The following are the key findings:

1. ASR axial expansion was highly sensitive to the stress state of concrete and behaved as a directional property. ASR axial expansion was reduced due to stress and transferred to a
no-stress or relatively lower stress direction. ASR attempts to conserve the volumetric expansion in concrete as long as one unstressed direction is available.

2 The expansion transfer mechanism was confirmed by the DRI results. The comparison of DRI values along different planes demonstrated that even the biaxial confinement was not adequate to suppress ASR damage. However, triaxial specimens showed markedly smaller DRI values compared to the other stress conditions suggesting that the triaxial confinement can reduce ASR damage. Even though the triaxial confinement did not prevent the reaction, the triaxial confinement was effective to largely reduce cracking, particularly the mortar matrix cracking.

3 The stress state influenced the opening of cracks. Cracks were opened preferentially along the unstressed direction or the direction with a relatively smaller compressive stress.

4 Similar to the ASR axial expansion, the cracking of concrete and the change in mechanical properties showed a directional behavior with the stress state. The loss in static modulus of elasticity was the greatest for the no-stress specimens compared to the stressed specimens. The modulus of elasticity was observed to be anisotropic as a direct result of the directional stress state.

5 Triaxial compressive stress in ASR concrete contributed to having reduced volumetric expansion, lower DRI value, less cracking, and less overall degradation of the mechanical properties.

This study integrates the understanding of the three main effects of ASR, namely expansion, cracking and degradation of mechanical properties of concrete in the context of concrete structures. The findings from this study will be useful in the analysis of reinforced and multiaxially stressed concrete structures, in particular, suggesting the necessity of treating ASR-affected concrete as an orthotropic material. This is anticipated to increase the accuracy of numerical analysis. Similar studies with various types of reactive aggregates and various mix designs will improve the understanding of the consequences of ASR in the concrete structures. The findings from this study can further help in performing field investigation programs, in particular, to ascertain the direction of expansion measurement and coring, and to interpret the orientation of cracks.
4.6 References


[29] B. Fournier, P.-L. Fecteau, V. Villeneuve, S. Tremblay, L.F.M. Sanchez, Description of petrographic features of damage in concrete used in the determination of the damage rating index (DRI), Laval University, Quebec City, Quebec, Canada, 2015.
Chapter 5
The Effect of Elevated Temperature on the Expansion, Damage Rating Index and Mechanical Properties of ASR-Affected Concrete

Abstract

Experimental studies on alkali-silica reaction (ASR) are mostly performed at an elevated temperature to accelerate the reaction. However, the conditioning temperature can vary depending on the purpose and duration of a study, and the damage may not be linearly proportional to the temperature. In order to evaluate the effect of an elevated accelerating temperature on the performance of ASR-affected concrete, this study compared expansion, damage rating index and mechanical properties of concrete prism specimens conditioned at 38 °C and 50 °C. A 12 °C rise in temperature accelerated the expansion by 3.22 times. The trend of loss in mechanical properties was similar at both temperatures even though the extent of loss and the microstructural cracking increased slightly at 50 °C. Increasing the temperature to 50 °C from 38 °C can shorten the test duration by more than three times with little effect on the response of concrete.

Keywords: Alkali-aggregate reaction; temperature, acceleration; expansion; damage rating index

5.1 Introduction

Alkali-silica reaction (ASR) is an undesirable chemical reaction in concrete that causes expansion, cracking and degradation of mechanical properties. Even though ASR has now been adequately understood to ensure new concrete structures are safe against it, ASR is still a major deterioration problem in many existing structures. The reaction is slow and usually takes 10 years or more to show its effects in field concrete structures [1]. In an experimental study that involved large concrete specimens subjected to an outdoor condition, ASR expansion was monitored and it continued for 20 years [2]. However, the urgent need for diagnosis, prognosis and necessary repair measures of existing structures cannot afford such a long duration for an experimental study. Therefore, experimental studies on concrete specimens are typically performed under an accelerated condition. While most of the studies involve small specimens...
such as concrete prisms (75 × 75 × 285 mm) and cylinders (φ100 × height 200 mm), the study of larger specimens (1 m or longer) such as beams and walls are preferred to understand the performance of concrete structures affected by ASR [3–5].

The relatively slow rate of ASR is a challenge for testing of larger specimens. While small specimens can be accommodated in existing laboratory facilities, maintaining the conditions necessary for accelerating the rate of reaction in large structural specimens and conditioning them for an extended duration can be logistically challenging. This is one of the reasons why large specimens are rarely studied. One such study [3] involved 3 m long concrete beams (cross-section of 0.25 × 0.5 m) that were subjected to an elevated temperature of 38 °C. The measurements in the beams were reported for 14 months. Yet, the growing expansion and beam deflection indicated that the reaction did not exhaust during the reported duration. Relatively larger thickness of the beams, when compared to the small specimens such as prisms, might have partly contributed to the lengthened duration of the reaction. A laboratory study of large structural specimens necessitates an accelerating condition that fosters a faster rate of reaction.

Accelerating the rate of reaction can be achieved by increasing the exposure temperature, relative humidity, or alkalinity, or combinations thereof. Most laboratory studies employ the combination of all three accelerating measures. Nevertheless, increase in temperature appears to be the most common accelerating variable to increase the rate of reaction.

The most common temperature for accelerating ASR on concrete specimens is 38 °C based on the concrete prism test (CPT), which is regarded as a reliable reference test method for assessing the potential alkali-silica reactivity of aggregates as per ASTM [6], CSA [7] and RILEM [8] standards. However, CPT takes 1 to 2 years for the test and is regarded as slow for many purposes. Practical requirements have encouraged shortening the test duration by further increasing the temperature [9–11]. For instance, the accelerated concrete prism test, usually performed at 60 °C [12], has emerged as a faster alternative to CPT. Nevertheless, the 60 °C temperature method is reported to cause reduced expansion compared to that achieved when exposed to 38 °C temperature. Some reasons for the reduced expansion at 60 °C are reported [10,13] as (i) increased leaching of alkalies; (ii) change in pore solution chemistry owing to the reduced concentration of hydroxyl ions; and (iii) drying of prisms at higher temperature. Another unusual observation reported in the study [13] of ASR conducted at 60 °C was that non-reactive
fine aggregate was attributed to demonstrate a “dramatic effect” of reduced expansion and large variability in expansion [13]. While the mechanism was not clearly understood, the combination of a particular reactive coarse aggregate with a number of non-reactive aggregates resulted in widely varying expansions [13].

Considering the complexity associated with interpretation of the response of ASR-affected concrete specimens conditioned at 60 °C, Folliard et al. [14] performed a series of concrete prism tests at 49 °C, an intermediate value between 38 and 60 °C. While the ultimate expansion decreased with an increase in temperature, this effect was more pronounced for the temperature increment from 49 to 60 °C than from 38 to 49 °C. The ultimate expansions corresponding to 60 °C (0.09%) were markedly less than those corresponding to 49 °C (0.17%) and 38 °C (0.20%). A non-linear effect of the two temperature increments was also observed in terms of alkali leaching. Alkali leaching was more pronounced for the temperature increment from 49 to 60 °C than from 38 to 49 °C. These findings indicate that the accelerating temperature for investigating the performance of large specimens should be limited to 49 °C (say 50 °C).

An increase in temperature accelerates the rate of ASR by thermoactivating the two processes involved in ASR [15]. Firstly, an increase in temperature accelerates the dissolution of silica in aggregates. For instance, the rates of dissolution of three types of silica particles measured at 25 °C in 3 M NaOH solution for 3 days were 71%, 8% and 7%, whereas these amounts were, respectively, 100%, 66% and 31% when measured at 80 °C [16]. In another study [17], the dissolution of silica at 80 °C was reported to be 6 to 7 times more than at 60 °C. Secondly, increased temperature accelerates the rate of formation of reaction products [15]. Results from an experimental study [18] on concrete conditioned at temperatures ranging from 23 to 58 °C confirmed that both the processes follow the Arrhenius law of chemical reaction [15]. Based on the formulas for two time constants for the two processes [15], the rate of expansion is approximately doubled when the temperature is increased from 38 to 50 °C. As per the formulas proposed in another experimental study [19], the rate of reaction can be increased by approximately 1.7 times when the temperature is increased from 38 to 50 °C. These findings indicate that the test duration can be significantly reduced by choosing 50 °C instead of 38 °C as the accelerating temperature for large concrete specimens.

Recently, an accelerating temperature of 50 °C was chosen in a large-scale project aimed at investigating the consequences of ASR on existing structures [20]. The project involved several
large unreinforced and reinforced concrete specimens with thickness ranging from 75 to 254 mm and length of up to 1.5 m. Even though the accelerating temperature was chosen as 50 °C, the study had to rely on the vast knowledge of ASR gained mainly through experiments performed at 38 °C. For example, the reactivity of aggregate to be used for casting the specimens was decided based on the expansion of concrete prisms tested at 38 °C. To evaluate the effect of temperature increment from 38 to 50 °C on expansion, which is one of the key performance indicators for reactive concrete, a study was necessary to compare the expansion results at the two temperatures. Moreover, the specimens were designed to be monitored for the performance of concrete in terms of cracking and degradation of mechanical properties. However, most of the performance indicators for ASR damage are either unavailable or inadequate for tests at 50 °C. The damage rating index (DRI) is one such indicator that quantitatively assesses the damage due to ASR based mainly on the cracks it develops in concrete. The DRI method has been extensively applied on field samples and on specimens conditioned at 38 °C [21–24]. In order to assess DRI as a performance indicator for ASR-affected concrete, Sanchez [25] performed DRI analysis of twenty concrete mixtures made with a variety of reactive aggregates. The study discussed the influence of concrete strength and different types of reactive aggregates on DRI. However, all the experiments were performed at 38 °C, and hence, the effect of temperature on damage was not investigated. An increase in temperature can lead to larger damage despite having identical ultimate expansion as indicated by a recent numerical study [26]. Thus, a study comparing DRI of identical concrete specimens conditioned at 38 °C and at 50 °C is essential before DRI can be used as a performance indicator for the concrete specimens conditioned at 50 °C. Furthermore, to better understand the influence of increasing temperature on the damage of concrete, the degradation of mechanical properties should be compared at the two temperatures.

This study was performed to compare the performance of ASR-affected concrete conditioned at 50±0.5 °C and at 38±2 °C. This chapter compares the consequences of ASR as assessed on identical concrete prism specimens tested at 38±2 °C and at 50±0.5 °C. Both tests include reactive and control (non-reactive) mixes of concrete. Longitudinal expansion of the prism specimens from the two tests is compared. Prism specimens from the two tests are analyzed by the DRI method. The results are discussed to compare the damage between the two tests, to elucidate the evolution of ASR damage, and also to highlight some limitations of the DRI method. The degradation of dynamic modulus of elasticity and modulus of rupture of the prism
specimens is measured. Based on the findings from the tests, a conclusion is made regarding the implication of increasing the accelerating temperature from 38 to 50 °C in consideration to accelerating the rate of ASR in large concrete specimens.

5.2 Materials and Methods

5.2.1 Concrete mix design and materials

The mix design was based on the concrete prism test as per ASTM C1293 [6], and is shown in Table 5.1. The water-to-cement ratio was 0.44. High alkali general use (GU) cement was used with a total alkali content of 0.99% Na₂O equivalent by mass of cement. The chemical composition of cement is shown in Table 5.2. The alkali content of the mix was boosted to 1.25% Na₂O equivalent of cement by adding NaOH pellets to water prior to concrete mixing. No other admixtures were used.

Two types of concrete were considered, namely reactive and control (non-reactive). The reactive concrete was made with reactive coarse aggregate and non-reactive fine aggregate. The control concrete consisted of non-reactive fine and non-reactive coarse aggregates.

The reactive coarse aggregate was Spratt aggregate from Stittsville near Ottawa, Ontario, Canada [27]. The Spratt limestone containing reactive silica minerals (9% SiO₂) is classified as highly reactive aggregate, and has been widely studied in Canada and abroad [28]. It is crushed aggregate from a quarry. The non-reactive coarse aggregate for the control concrete was crushed limestone aggregate from Milton, Ontario, Canada. Non-reactive fine aggregate was natural sand from Orillia, Ontario, Canada. The accelerated mortar bar test was performed to confirm the non-reactivity of sand as per ASTM C1260 [29].

<table>
<thead>
<tr>
<th>Mix</th>
<th>Cement (kg/m³)</th>
<th>Water (kg/m³)</th>
<th>Coarse aggregate (kg/m³)</th>
<th>Sand (kg/m³)</th>
<th>Alkali pellets (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reactive</td>
<td>420</td>
<td>184.8</td>
<td>1115.0</td>
<td>719.2</td>
<td>1.41</td>
</tr>
<tr>
<td>Control</td>
<td>420</td>
<td>184.8</td>
<td>1138.8</td>
<td>691.0</td>
<td>1.41</td>
</tr>
</tbody>
</table>

Note: The dry-rodded density of reactive coarse aggregate was 1585 kg/m³ and that of non-reactive coarse aggregate was 1604 kg/m³.

The mass of aggregates is shown for saturated surface-dry condition.
Table 5.2: Chemical composition of GU cement

<table>
<thead>
<tr>
<th>Constituents</th>
<th>LOI</th>
<th>SiO₂</th>
<th>Al₂O₃</th>
<th>Fe₂O₃</th>
<th>CaO</th>
<th>MgO</th>
<th>SO₃</th>
<th>Alkali (Na₂Oₑq)</th>
<th>Free Lime</th>
<th>Insoluble residue</th>
</tr>
</thead>
<tbody>
<tr>
<td>Percentage</td>
<td>2.27</td>
<td>19.25</td>
<td>5.33</td>
<td>2.41</td>
<td>62.78</td>
<td>2.36</td>
<td>4.01</td>
<td>0.99</td>
<td>1.29</td>
<td>0.52</td>
</tr>
</tbody>
</table>

5.2.2 Casting and conditioning of the specimens

5.2.2.1 Concrete prism specimens conditioned at 38 °C (CPT)

The first set of tests performed on concrete prisms conditioned at 38±2 °C has been referred to as the concrete prism test (CPT) in this study. Concrete prisms were cast as per ASTM C1293 [6]. Eighteen reactive and twelve control prisms were cast. The size of the prism specimens was 285 mm by 75 mm by 75 mm. Initial reading for longitudinal expansion was taken after demolding at one day of casting. The specimens were stored in sealed plastic pails in the vertical orientation and flipped at successive measurement intervals. Since the day of demolding, the prisms were cured under a constant temperature of 38±2 °C and relative humidity > 95%. The humidity was maintained by having a water layer at the bottom of the pails that was separated by an air space from the bottom end of the prisms. The pails were lined with a layer of water-absorbent paper that dipped to the bottom of the pail. Tests on the specimens were performed at the ages of 7, 28, 56, 91, 182, 273, 365 and 730 days. All test measurements were taken at room temperature by acclimatizing the pails beforehand for 16 to 20 hours.

5.2.2.2 Concrete prism specimens conditioned at 50 °C (ACPT)

The second set of tests performed on concrete prisms conditioned at 50±0.5 °C has been referred to as the accelerated concrete prism test (ACPT) in this study. Nine concrete prisms were cast for each of the reactive and the control mixes. The specimen casting, demolding and initial measurements were similar to those mentioned in Section 5.2.2.1. Since the day of demolding, pails containing three prism specimens each were stored under a constant temperature of 50±0.5 °C. The relative humidity was maintained by a method as mentioned in Section 5.2.2.1. Furthermore, the pails were stored inside an acceleration chamber maintained at > 95% relative humidity. Tests on the specimens were performed at the ages of 7, 28, 56, 91, 127, 159 and 187 days. All test measurements were taken at room temperature by acclimatizing the pails beforehand for 16 to 20 hours.
5.2.3  Test details

5.2.3.1  Expansion measurement

Longitudinal expansion was measured on the concrete prisms. The expansion was measured using a comparator against the steel studs embedded in the ends of the prisms with a gauge length of 250 mm. The comparator has a resolution of 1 µm, and was calibrated with a reference invar bar. The expansion at a given age was calculated as the difference between the comparator reading at a given age and the initial comparator reading taken at the day of demolding. (Four prism specimens were also measured for transverse expansions along the two directions of the cross-section as discussed in Appendix H).

5.2.3.2  Modulus of rupture

Two prisms were tested by third-point loading as outlined in ASTM C78 [30]. The span length \( L \) was maintained as 225 mm, which is three times the depth of the prism \( d = 75 \text{ mm} \).

The modulus of rupture \( (R) \) was calculated as,

\[
R = \frac{PL}{bd^2}
\]

where \( P \) is the maximum applied load and \( b \) is the average width of the prism. The values of \( b \) and \( d \) used in the calculation were taken by measuring each prism with a caliper.

The fracture always occurred in the tension surface within the middle third of the span. The cracked middle portion of a prism was discarded while the remaining portion of the prism was utilized for DRI analysis as described in Section 5.2.3.5.

5.2.3.3  Transverse resonant frequency

Fundamental transverse resonant frequency of the prisms was measured as per ASTM C215 [31], as a non-destructive method of estimating the degradation of dynamic modulus of elasticity of the prisms. A test prism was supported at its two nodal points located at a distance of 0.224 times the length of the prism from either end of the prism. The resonant frequency was observed as the maximum frequency obtained by changing the frequency of the driving force until resonance
would occur. Dynamic modulus of elasticity was calculated by using the formula given in ASTM C215 [31] as,

$$E_d = CMn^2$$  \hspace{1cm} (5.2)

where $M$ is the mass of the prism specimen, $n$ is the fundamental transverse resonant frequency and $C$ is a parameter that depends on the size, shape and the dynamic Poisson's ratio of the concrete specimen.

### 5.2.3.4 Poisson’s ratio

The dynamic Poisson’s ratios for the reactive and control concrete were estimated by measuring the pulse velocities of P- and S-waves in cube specimens (254 mm) by using a “Pundit Lab” ultrasonic equipment from Proseq USA, Inc. Dynamic Poisson’s ratio ($\mu_d$) was calculated by using the formula given in Proseq [32] as,

$$\mu_d = \frac{V_p^2 - 2V_s^2}{2(V_p^2 - V_s^2)}$$  \hspace{1cm} (5.3)

where $V_p$ and $V_s$ are the pulse velocities of P- and S-waves, respectively.

The values of $\mu_d$ were obtained as 0.276 and 0.267, respectively, for the reactive and non-reactive concrete specimens conditioned at 23±3 °C for 6 months of casting. These values were used to calculate the dynamic modulus of elasticity of concrete by resonant frequency method.

### 5.2.3.5 Damage rating index (DRI)

Damage Rating Index (DRI) method has emerged as a technique of quantifying the extent of damage in concrete [33]. The method was originally proposed by Grattan-Bellew in 1995 [24]; a revised version was proposed by Villeneuve et al. in 2012 [34], which was adopted in this study. Further reference was made to Sanchez et al. [23] and to the detailed guidelines on DRI published by Fournier et al. [35].

Two prisms were tested for DRI at each test age. For each prism, one slice near the middle and one slice near the end were taken and polished for the microscopic observation. The slices were
prepared by successively polishing to a surface roughness of approximately 5 µm (#1500 grit). Forty-nine 1 cm$^2$ grids were prepared as shown in Figure 5.1. Each grid was examined in a stereo binocular microscope at ~16× magnification to count the seven petrographic features as listed in Table 5.3. The numbers of counts were multiplied by the respective weighting factors and summed. The sum was normalized to an area of 100 cm$^2$ to provide the DRI value for a slice. DRI value for concrete at an age was obtained as the average for four slices from two prisms.

Figure 5.1: Polished section of a concrete prism (age 3 months, CPT) for DRI analysis (each grid is 1 cm by 1 cm)

<table>
<thead>
<tr>
<th>Petrographic features</th>
<th>Weighting factors</th>
</tr>
</thead>
<tbody>
<tr>
<td>Closed/tight cracks in coarse aggregate particle</td>
<td>0.25</td>
</tr>
<tr>
<td>Opened cracks or network cracks in coarse aggregate particle</td>
<td>2</td>
</tr>
<tr>
<td>Cracks or network cracks with reaction product in coarse aggregate particle</td>
<td>2</td>
</tr>
<tr>
<td>Debonded coarse aggregate</td>
<td>3</td>
</tr>
<tr>
<td>Disaggregated / corroded aggregate particle</td>
<td>2</td>
</tr>
<tr>
<td>Cracks in cement paste</td>
<td>3</td>
</tr>
<tr>
<td>Cracks with reaction product in cement paste</td>
<td>3</td>
</tr>
</tbody>
</table>
5.3 Results and Discussions

5.3.1 Expansion

Average longitudinal expansions of the concrete prisms in CPT (38 °C) and ACPT (50 °C) are presented in Figure 5.2. (The error bar in this and other figures in this thesis represents ± one standard deviation). Since two prisms were destructively tested at different test dates, the number of prisms gradually decreased over time. Thus, each data point shown in the figure represents the average of a varying number of prisms. For the reactive concrete prisms tested under CPT condition, the expansion at 14 days and 730 days represents the average of 18 prisms and four prisms, respectively. For the control (non-reactive) concrete prisms tested under CPT condition, the expansion at 28 days and 365 days represents the average of 12 and two prisms, respectively. For the prisms tested under ACPT condition, the expansion at 7 days and 187 days represents the average of nine and two prisms, respectively. The ACPT test was discontinued after 187 days.

The expansion of control prisms was less than 0.04%, a limit for non-reactivity as specified by ASTM C1293 [6]. For CPT, the expansion of reactive concrete prisms was 0.23% in 365 days. It further increased to 0.26% in 2 years (730 days). For ACPT, an expansion of 0.25% was
observed in 6 months (187 days) for the reactive prisms. The total expansion and the trend of expansion for the reactive prisms was similar in both tests; however, the rate of expansion was significantly faster in the ACPT.

Contrary to the literature [10,13,14], no reduction in expansion was observed due to an increase in temperature. One possible explanation could be that at 50 °C, the sulfate ions had minimal effect on reducing the pH of the concrete pore solution unlike at 60 °C at which the effect was reported as a possible reason for having reduced expansion [10]. However, this phenomenon needs to be verified experimentally. Furthermore, in this study, the ACPT specimens were kept in sealed buckets and the buckets were stored inside a > 95% relative humidity chamber. This would eliminate the effect of drying of prisms (drying of prisms was given as a possible reason for the reduced expansion at 60 °C [13,14]) compared to the tests in which buckets are stored in relatively drier environment and the buckets might not achieve perfectly sealing condition.

Figure 5.3 presents the days required by the reactive concrete for the identical expansions in CPT and ACPT. The number of days in CPT and ACPT for identical expansions correlate with the $R^2 = 0.99$. When the data was fit to a straight line passing through the origin, the slope of the line was 3.22, indicating that the rate of expansion in ACPT was 3.22 times faster than in CPT. The expansion vs. age data were re-plotted in Figure 5.4 with different horizontal scales for the ages for CPT and ACPT such that 3.22 days for CPT corresponded to 1 day for ACPT. When the scale for ACPT was larger by a factor of 3.22 than in CPT, the expansion curves from CPT and ACPT overlapped. The expansion results suggested that the trend and the total expansion due to ASR remain unaffected by changing the accelerating temperature from 38±2 to 50±0.5 °C. The rate of expansion increased by 3.22 times.
Figure 5.3: Days required by reactive prisms in CPT and ACPT for identical expansion

Figure 5.4: Longitudinal expansion of concrete prisms in CPT and ACPT using different time scales (note: top horizontal scale for age of ACPT and bottom horizontal scale for age of CPT)
5.3.2 Modulus of rupture

Figure 5.5 shows the modulus of rupture of concrete prism specimens for CPT and ACPT at different test ages. Each data point is an average for two prisms. Modulus of rupture for the non-reactive specimens varied between 6 and 8 MPa. Relatively larger scatter was observed in the non-reactive specimens than in the reactive specimens. This observation is consistent with a previous experimental study [36] that involved testing of several specimens. The study [36] showed that the coefficient of variation in modulus of rupture increases significantly with increasing flexural strength. Based on the finding of the study [36], ASTM standards for the flexural strength of concrete [30,37] mention that the coefficient of variation associated with the modulus of rupture results is dependent on the flexural strength of the specimens.

The modulus of rupture for the reactive prisms in CPT increased initially to attain its peak at 28 days. By 91 days, the modulus of rupture reduced to approximately 50% of its 28-day value. After 91 days, it reduced at a relatively slower rate until 365 days to reach 42% of its 28-day value. Closer examination of the data revealed that a partial recovery was observed from 365 to 730 days.

The modulus of rupture for the reactive specimens in ACPT followed a similar trend as in CPT except that the trend occurred faster. The trend comprised of a sharp reduction in modulus of rupture from 7 to 28 days, a relatively slower reduction from 28 to 91 days and a partial recovery from 91 to 187 days. The modulus of rupture at 28 days was 58% of its 7-day value and at 91 days, it was approximately 41% of its 7-day value. The modulus of rupture at 187 days was 55% of its 7-day value showing a partial recovery from 91 to 187 days.

A partial recovery in the mechanical properties of ASR-affected concrete has also been observed in the literature [38–41]. Hydration of cement has been generally attributed for such partial recovery. Irrespective of the underlying mechanism, an identical trend of partial recovery was observed in both CPT and ACPT specimens. Nevertheless, the mechanism of partial recovery was investigated, which is discussed in detail in Chapter 7.

The reduction in modulus of rupture appeared slightly greater in ACPT than in CPT. Based on the observed maximum value of modulus of rupture (7.85 MPa), which occurred at 28 days in CPT, and the observed minimum values (3.31 MPa in CPT at 365 days and 2.78 in ACPT at 91
days), modulus of rupture for the reactive concrete reduced by a maximum of 58% and 65% in CPT and ACPT, respectively.

Figure 5.5: Variation in modulus of rupture of concrete prism specimens over time using different time scales

5.3.3 Dynamic modulus of elasticity

Figure 5.6 shows the dynamic modulus of elasticity for the prism specimens in CPT and ACPT as calculated from the transverse resonant frequency test. Each data point is an average of two prisms. In order to illustrate the effect of ASR, the results in Figure 5.6 are presented relative to the control (non-reactive) prisms. The dynamic modulus of elasticity for CPT was normalized to the 28-day value of control prisms in CPT. For ACPT, the dynamic modulus of elasticity was normalized to the 7-day value of control prisms in ACPT. For the control prisms, the dynamic modulus of elasticity showed a slight and gradual increase over time. At 365 days, dynamic modulus of elasticity of the control prisms in the CPT increased by 5% compared to its 28-day value. Likewise, at 187 days, dynamic modulus of elasticity of the control prisms in the ACPT
increased by 7% compared to its 7-day value. The slight and gradual increase in dynamic modulus of elasticity of control concrete must be due to the continued hydration of cement.

For the reactive concrete, the dynamic modulus of elasticity of the prisms in the CPT increased from 14 to 28 days and reduced thereafter until 273 days. The maximum reduction (18%) in the dynamic modulus of elasticity was observed at 273 days, with respect to the dynamic modulus of elasticity at 28 days. A partial recovery in the dynamic modulus of elasticity was observed from 273 to 365 days. In the ACPT, the dynamic modulus of elasticity reduced by 12% from 7 to 28 days. The maximum reduction of 24% was observed at 56 days, with respect to the dynamic modulus of elasticity at 7 days. After 56 days, a partial recovery was observed in the dynamic modulus of elasticity, a mechanism similar to that in CPT.

Figure 5.6: Variation in dynamic modulus of elasticity over time (using different time scales) from the resonant frequency method
5.3.4 Damage rating index (DRI)

Table 5.4 presents the number of counts of the seven petrographic features for DRI per 100 grids for the reactive concrete specimens in both the CPT and the ACPT. The DRI examination was performed on the slices taken along the transverse direction of the prism specimens. (For comparison purpose, some prisms were examined for DRI along the longitudinal section as discussed in Appendix I). The DRI value was calculated based on the number of counts by following the procedure outlined in Villeneuve et al. [34]. The DRI values for the reactive prisms in CPT and ACPT are plotted against the longitudinal expansion as shown in Figure 5.7. The DRI value increased with an increase in expansion and showed a fairly linear trend with expansion.

Table 5.4: Number of counts of the seven petrographic features per 100 cm$^2$

<table>
<thead>
<tr>
<th>Test</th>
<th>Age (days)</th>
<th>Long. Exp. (%)</th>
<th>DRI value</th>
<th>Closed/tight cracks in coarse aggregate (1)</th>
<th>Opened cracks (or networks) (2)</th>
<th>Filled cracks (or networks) (3)</th>
<th>Debonded coarse aggregate (4)</th>
<th>Deposited/corroded aggregate (5)</th>
<th>Cracks in cement paste (6)</th>
<th>Filled cracks in cement paste (7)</th>
<th>Sum (2+3+6+7)</th>
<th>Crack density (count/cm$^2$) [23]</th>
</tr>
</thead>
<tbody>
<tr>
<td>CPT</td>
<td>14</td>
<td>0.006</td>
<td>83</td>
<td>148</td>
<td>3</td>
<td>15</td>
<td>3</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>18</td>
<td>0.18</td>
</tr>
<tr>
<td></td>
<td>28</td>
<td>0.013</td>
<td>123</td>
<td>236</td>
<td>4</td>
<td>24</td>
<td>3</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>28</td>
<td>0.28</td>
</tr>
<tr>
<td></td>
<td>56</td>
<td>0.048</td>
<td>209</td>
<td>273</td>
<td>16</td>
<td>48</td>
<td>2</td>
<td>1</td>
<td>2</td>
<td>0</td>
<td>66</td>
<td>0.66</td>
</tr>
<tr>
<td></td>
<td>91</td>
<td>0.101</td>
<td>320</td>
<td>180</td>
<td>36</td>
<td>96</td>
<td>1</td>
<td>0</td>
<td>2</td>
<td>1</td>
<td>135</td>
<td>1.35</td>
</tr>
<tr>
<td></td>
<td>182</td>
<td>0.171</td>
<td>468</td>
<td>197</td>
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<td>153</td>
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<td>4</td>
<td>196</td>
<td>1.96</td>
</tr>
<tr>
<td></td>
<td>273</td>
<td>0.210</td>
<td>604</td>
<td>157</td>
<td>63</td>
<td>172</td>
<td>2</td>
<td>2</td>
<td>20</td>
<td>9</td>
<td>264</td>
<td>2.64</td>
</tr>
<tr>
<td></td>
<td>365</td>
<td>0.223</td>
<td>614</td>
<td>125</td>
<td>66</td>
<td>166</td>
<td>1</td>
<td>2</td>
<td>23</td>
<td>14</td>
<td>269</td>
<td>2.69</td>
</tr>
<tr>
<td></td>
<td>730</td>
<td>0.261</td>
<td>892</td>
<td>109</td>
<td>86</td>
<td>190</td>
<td>18</td>
<td>30</td>
<td>28</td>
<td>39</td>
<td>343</td>
<td>3.43</td>
</tr>
<tr>
<td>ACPT</td>
<td>28</td>
<td>0.085</td>
<td>508</td>
<td>182</td>
<td>31</td>
<td>167</td>
<td>3</td>
<td>7</td>
<td>12</td>
<td>3</td>
<td>213</td>
<td>2.13</td>
</tr>
<tr>
<td></td>
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<td>10</td>
<td>15</td>
<td>5</td>
<td>290</td>
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</tr>
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<td></td>
<td>91</td>
<td>0.214</td>
<td>805</td>
<td>141</td>
<td>88</td>
<td>231</td>
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<td></td>
<td>187</td>
<td>0.248</td>
<td>832</td>
<td>74</td>
<td>127</td>
<td>187</td>
<td>5</td>
<td>10</td>
<td>35</td>
<td>15</td>
<td>364</td>
<td>3.64</td>
</tr>
</tbody>
</table>

5.3.4.1 Detailed discussion on the effect of elevated temperature

The DRI values were greater for the ACPT specimens than for the CPT specimens at the same expansion level as shown in Figure 5.7. Until an expansion of 0.21%, the DRI value for the
ACPT specimens was greater by approximately 200 than for the CPT specimens. Thereafter, despite a relatively slower increase in CPT expansion from 0.21% to 0.26%, the DRI value did indeed increase significantly. The DRI value for the CPT specimens approached closer to the ACPT specimens at an identical expansion value (Figure 5.7). Debonding, disintegration of particles and paste cracks had an increased contribution to the DRI value for the CPT specimens corresponding to the increase in expansion from 0.21% (365 days) to 0.26% (730 days). The combined contribution of these features increased from 19% at 365 days to 35% at 730 days.

![Figure 5.7: DRI value versus longitudinal expansion for the reactive prisms in CPT and ACPT](image)

Whether greater DRI values in ACPT actually indicated more cracks was further analyzed by comparing the number of cracks without the weighting factors. Figure 5.8 compares the number of counts of the seven petrographic features in CPT and ACPT for two different levels of expansion (~0.1% and 0.21%). For an identical expansion, the faster rate of reaction in ACPT (compared to CPT) induced more cracks in the aggregate particles. Also, for the expansion of ~0.1%, the number of paste cracks was greater in ACPT than in CPT. Figure 5.9 shows a digital microscope image of a polished surface of concrete cured at 50±0.5 °C for 91 days. An abundance of reaction product can be seen inside the aggregate particle. The pressure of the reaction product inside the aggregate is attributed to the cracking of the aggregate particle which
was followed by paste cracking. This mechanism is in agreement with the cracking of the aggregate particles due to the expansive pressure of “gel pockets” and the propagation of cracking to the paste matrix as discussed by Dunant and Scrivener [42]. Moreover, the type of cracks in aggregate has been attributed to influencing the amount of paste cracking; “sharp” cracks in aggregate contribute to increased paste cracking [43]. Increased temperature in ACPT is expected to have formed increased number of sharp cracks in aggregate particles, thus, resulting in extensive paste cracking at the early age. A parameter called “crack density” was defined by Sanchez et al. [23] to better explain the cracking of concrete specimens due to ASR. Crack density is defined as the sum of the numbers of filled and opened cracks in aggregate and all the paste cracks per unit area of a polished surface of concrete. The crack density for the CPT and the ACPT specimens is compared in Table 5.4; it shows that ACPT caused more cracks in concrete than CPT did.

![Figure 5.8: Comparison of the number of counts (i.e. no weighting factors) of the damage features in CPT and ACPT (for 0.1 and 0.21% expansions)](image)

The relatively faster rate of reaction in ACPT, analogous to a faster rate of loading, resulted in a greater number of cracks within the aggregate particles and a greater number of paste cracks (at the moderate level of expansion of ~0.1%) compared to the CPT. A recent numerical study [26]
demonstrated that despite having identical expansion, the concrete conditioned at a higher temperature (50±0.5 °C) may experience more cracks compared to the concrete conditioned at a lower temperature (38±2 °C). Owing to the inherent heterogeneity of concrete and the viscoelastic behavior of the paste matrix, the response of concrete to expansive stresses varies as a function of the rate of stress application. As observed in this study, the paste (actually the mortar matrix) cracking followed the aggregate cracking. Once the reaction product swelled, it applied pressure to the mortar matrix, which in turn accommodated the pressure by either expanding or cracking. The rate of application of expansive pressure was influenced by the rate of reaction, which was markedly faster in ACPT than in CPT. Due to the viscoelastic nature of the matrix, the faster reaction (in ACPT) resulted in increased paste cracking; in contrast, the relatively slower reaction (in CPT) accommodated more expansive pressure through expansion by undergoing less paste cracking. Figure 5.10 shows the contribution of aggregate cracks and paste cracks to DRI value for different levels of expansion in CPT and ACPT. From Figure 5.10, after having approximately 0.14% expansion of concrete, it is expected that the expansive pressure gradually begins to surpass the tensile strength of the paste matrix even in CPT and results in increasing levels of paste cracking.

Figure 5.9: Reaction product in a coarse aggregate particle on a polished surface of concrete cured at 50±0.5 °C for 91 days, as observed under a digital microscope.
The prism surfaces that exhibited the most surface cracking were scanned in a flatbed scanner. The portion of the scanned images are shown in Figure 5.11. The chosen surfaces are among the ones that exhibited the most surface cracking to naked eyes. The prism surface in ACPT in 6 months showed a visible network of hairline cracks. The prism surface in CPT in 2 years showed a few hairline cracks. These images suggest that an increase in temperature causes an increased cracking of the concrete microstructure.

5.3.4.2 Evolution of damage as measured by DRI method

DRI method in this study was useful to understand the evolution of ASR damage in reactive concrete. Figure 5.12 shows the evolution of the ASR damage with age as measured by DRI method for the CPT specimens from 14 to 730 days. It illustrates the contributions of the seven petrographic features to the DRI value.
The damage was initiated in the reactive aggregate particles. The contribution to DRI value was dominated by closed/tight cracks until 56 days (Figure 5.12). The closed/tight cracks contributed up to 48% of the DRI value at 28 days but their contribution decreased significantly after 56 days. The number of counts of closed/tight cracks (Table 5.4) increased from 14 to 56 days but decreased thereafter indicating that damage initiation was predominantly in the form of closed/tight cracks in the aggregates. On the other hand, the number of counts of filled and opened cracks increased with an increase in age. An increase in the number of filled and opened cracks but a decrease in the number of closed/tight cracks indicated that most of the closed/tight cracks were the precursor of the filled or the opened cracks. Closed/tight cracks have been viewed as not only due to ASR but also due to weathering or crushing operations before being used in concrete [23,25,35]. However, the two types can be differentiated if DRI is examined in a reference concrete slice that is prepared before the evolution of ASR. For instance, even if aggregates had pre-existing closed/tight cracks, these were included in the number of counts at 14 days, which was 148 per 100 cm$^2$ (Table 5.4). After multiplying the number of counts with the corresponding weighting factor of 0.25, the maximum possible contribution of such cracks was thus limited to a 37 DRI value. Had a slice been prepared at the age of one day, the number of closed/tight cracks must have been even lower. The feature “closed/tight cracks” indeed serves as an important parameter to capture the onset of damage due to ASR.

The contribution to DRI value was dominated by the aggregate cracks throughout the study duration. The closed/tight cracks gradually transformed into filled or opened cracks. The paste cracks were insignificant until 91 days in CPT indicating that they initiated only after significant reaction took place in the aggregate particles. Figure 5.10 shows the contribution of aggregate cracks and paste cracks to DRI for different levels of expansion in CPT and ACPT. The contribution and its trend appear identical in both cases except that the paste cracking is pronounced in ACPT even at a moderate expansion, of say 0.1%. The contribution of aggregate cracks varied from 96% to 64% to dominate the DRI value. The contribution of paste cracks gradually increased from 0% to a maximum of 23% up to 730 days.
Figure 5.11: Flatbed scan images of the middle portion of concrete prisms (width 75 mm). The a) prism in ACPT exhibited a visible network of cracks compared to the b) prism in CPT.

![Figure 5.11](image)

Figure 5.12: DRI of reactive prisms in CPT at different ages with contributions of the seven petrographic features.
5.3.4.3 Some remarks on the DRI method [34]

The DRI is a comprehensive method to quantitatively determining the degree of damage due to ASR. However, the DRI method has some limitations [23, 34, 35]. First, the method is a semi-quantitative method that can have a large variability on the results based on the experience of an operator. In order to reduce the variability between operators in the DRI method, Villeneuve et al. [34] proposed a revision in the definition of the damage features and in their weighting factors. Second, even though the weighting factors were chosen in a logical basis, the relative weights assigned to several damage features are still arbitrary [23]. Third, the DRI value can have high variability based on the minimum size of the aggregate particle that is accounted for counting the damage features. A minimum size of 2 mm has been recommended by Fournier et al. [35].

This study highlights three additional limitations, particularly in the context of characterizing the evolution of damage, including: the ambiguity associated with the thickness of a crack in classifying it as either tight or opened crack; the length of cracks; and the coalescence of multiple cracks into a single damage feature of disaggregated particle. These three aspects are discussed further.

i) Classification of an aggregate crack into a closed/tight crack or a filled or opened crack is based on the judgment of the operator as to the crack thickness. As many of the closed/tight cracks grew in width to be transformed into filled or opened cracks, a polished surface of concrete at a particular instant did contain cracks with a wide range of thicknesses (and also cracks with varying thickness along their length). This resulted in an ambiguous situation in which a particular crack could be classified either as a closed/tight crack or a filled or opened crack.

ii) The length of cracks was realized as an important aspect that is not currently sufficiently considered in the DRI method [34]. While an aggregate particle less than 2 mm on the polished section was not included in crack counting [35], there were many instances of short cracks (1–2 mm long) in larger particles. Such a short crack receives the same weighting in the calculation of a DRI value as by a crack as long as 14 mm, which is approximately the diagonal dimension of a 1 cm² grid. Also, two or three cracks of length, say 3 mm, at one stage of reaction could ultimately grow to form a single long crack, say
10-14 mm long, at a later stage. Depending on when the concrete slice was made, initially a particular damage feature may be counted as three cracks but its mature form could be counted as a single crack.

iii) A number of relatively smaller filled or opened cracks, which were discrete at an earlier stage and would be counted as multiple cracks, would later coalesce into a reacted fraction of an aggregate, which would be counted as a single disaggregated particle. Both features having the same weighting in the calculation of a DRI value, initially a particular damage feature may be counted as multiple cracks but its mature form could be counted as a single disaggregated particle.

The discrepancy in DRI corresponding to the identical expansions in CPT and ACPT could partly be attributed to these three limitations. For instance, the elevated temperature in ACPT resulted in several smaller cracks as suggested by greater total number of aggregate cracks (sum of the crack counts in columns 1, 2 and 3 of Table 5.4) in the ACPT specimens than in the CPT specimens for similar levels of expansion. However, some of the smaller cracks coalesced to form larger cracks, and this was more so in the ACPT specimens as suggested by a decrease in the total number of aggregate cracks from 460 to 388 during 91 to 187 days.

5.4 Summary and Concluding Remarks

This study investigated the influence of elevated conditioning temperature on the damage caused by ASR on reactive concrete containing one type of reactive coarse aggregate, Spratt. Two sets of tests were performed, namely (1) concrete prism test (CPT) performed at 38±2 °C and (2) accelerated concrete prism test (ACPT) performed at 50±0.5 °C. Both sets included identical reactive and control concrete prism specimens. The prisms were prepared, and tested identically except that they were conditioned in environments at different temperatures. In order to understand the evolution of damage due to ASR, all the three effects of ASR on concrete, they are expansion, cracking and degradation of mechanical properties, were monitored.

With an increase of 12 °C temperature from 38±2 to 50±0.5 °C, the expansion in ACPT was 3.22 times faster than in CPT. Except for the rate of expansion, the general trend of expansion and the ultimate expansion were similar for both the ACPT and CPT specimens.
Accounting for the 3.22 times faster rate of reaction in ACPT than in CPT, the trends of reduction and partial recovery in modulus of rupture and in dynamic modulus of elasticity were identical in both CPT and ACPT. However, the reduction was slightly greater in ACPT than in CPT. Modulus of rupture reduced by approximately 58% in CPT and 65% in ACPT. Dynamic modulus of elasticity reduced by approximately 18% in CPT and 24% in ACPT.

DRI results elucidated the evolution of damage due to ASR in the reactive concrete. The ability of the DRI method to account for tight cracks captured the onset of damage due to ASR. Despite some limitations, DRI is a comprehensive method for investigating the damage due to ASR. However, rather than the absolute DRI value, the individual petrographic features of the DRI method are equal or more instrumental to understand the evolution of damage due to ASR. The faster rate of reaction in ACPT induced greater number of cracks at the earlier stage of reaction. This resulted in larger DRI value in ACPT than in CPT for identical expansion. While the expansive pressure of a relatively slower reaction was accommodated by the CPT specimens without undergoing significant paste cracking, after having approximately 0.14% expansion of concrete, the expansive pressure gradually surpasses the tensile strength of the matrix and yields increasing levels of paste cracking. On the other hand, many smaller cracks in ACPT coalesced to form larger cracks, particularly from 91 to 187 days.

Identical expansions and similar trends of modulus of rupture and dynamic modulus of elasticity suggest that an increase in conditioning temperature from 38±2 °C to 50±0.5 °C speeds up the test with little effect on the ASR mechanism based on specimens in this study. Nevertheless, a slight increase in the extent of damage and microstructural cracking with a 12 °C rise in temperature indicates that while accelerated tests are necessary for ASR studies, an increased temperature is likely to involve an increased deviation in the response of concrete from that in the field concrete structures, which was, however, not investigated in this study.

5.5 References


[35] B. Fournier, P.-L. Fecteau, V. Villeneuve, S. Tremblay, L.F.M. Sanchez, Description of petrographic features of damage in concrete used in the determination of the damage rating index (DRI), Laval University, Quebec City, Quebec, Canada, 2015.


Abstract

Assessment of alkali-silica reactivity is an important quality control measure for concrete aggregates in preventing alkali-silica reaction (ASR) in concrete structures. However, the reactivity is often assessed, by the concrete prism test specified in ASTM C1293, for a particular grading of coarse aggregate that may deviate from the aggregate grading in field concrete. This study investigated the effects of coarse aggregate grading on the ASR expansion and damage of concrete. The results indicated that a deviation of 10% in the grading of reactive coarse aggregate could result in up to 50% larger concrete expansion compared to concrete with standard (ASTM C1293) grading. Aggregate grading also significantly influenced the ASR damage of concrete as measured by the damage rating index method. Furthermore, aggregate grading influenced the quality of concrete. The findings illustrate that aggregate grading is an important parameter in the study of the reactivity of aggregates and the ASR performance of concrete.

Keywords: Alkali-aggregate reaction; expansion; aggregate; degradation; aggregate grading

6.1 Introduction

Reactive aggregate is a necessary component required for the occurrence of alkali-aggregate reaction, which is a serious deterioration problem in concrete structures. Alkali-silica reaction (ASR), the predominant type of alkali-aggregate reaction, takes place between the alkali hydroxides available in concrete pore fluid and the reactive silica supplied by the aggregates, and causes cracking, expansion and deterioration of concrete. The type and amount of reactive silica available in aggregates largely influences their alkali-silica reactivity. The proportion and the size of a particular type of reactive aggregate have also been found to largely influence the
expansive behavior of ASR-affected concrete, which are generally explained in terms of pessimum effects [1–6].

ASR expansion of a concrete mix varies with the type, content and size of the reactive aggregate. For a particular type of reactive aggregate, a pessimum proportion is the certain proportion of reactive aggregate that corresponds to the maximum expansion [1]. Similarly, the pessimum size corresponds to the aggregate size exhibiting the maximum expansion [1]. Most of the studies on pessimum size have focused on fine aggregate. Although the exact size may vary with the type of aggregate and also with the size of the specimen [5], the pessimum size is generally observed to be in the range of 0.5–2 mm [1,3,7].

The size effect is, however, less clearly understood for coarse aggregates [8,9]. Zhang et al. [4] concluded that expansion is reduced with an increase in size for a reactive siliceous aggregate (from China). A similar observation of reduced expansion with an increase in aggregate size was observed by Wigum [10] for a highly reactive aggregate (from Iceland). On the other hand, Dunant and Scrivener [11] reported that the maximum expansion among the size ranges of 0–2 mm, 2–4 mm, 4–8 mm and 8–16 mm (for chloritic schist type Alpine aggregate) was observed for the size range of 4–8 mm. French [12] reported that the size range for the harshest ASR damage varies for different types of aggregates. For instance, reactive chert aggregate was most damaging in the size range of 3–7 mm, and recrystallized sandstone was most damaging in the size range of 10–20 mm. Even though a universal pessimum size perhaps does not exist for coarse aggregates, these studies indicate that the size of the coarse aggregate can influence the ASR expansion and damage of concrete.

The gradation of coarse aggregate in concrete spans a relatively broad spectrum, usually from 5 to 25 mm. A concrete mix design usually follows a grading requirement (e. g. ASTM C33 [13]) for coarse aggregate. For instance, ASTM C1293 [14], the concrete prism test (CPT), prescribes a size range of 4.75–19.0 mm for coarse aggregate. For the size range of 4.75–19.0 mm, ASTM C1293 [14] specifies that the coarse aggregate be comprised of three equal proportions: 19.0–12.5 mm, 12.5–9.5 mm and 9.5–4.75 mm. In context with ASR, a specific coarse aggregate grading is necessary to provide a basis to compare CPT results from different researchers by eliminating the consequences such as the variation in aggregate grading. However, the
consequences of deviating from a specified grading of reactive coarse aggregate on concrete’s response to ASR have not been adequately reported in the literature.

Two aspects can be specifically identified to indicate the need for a study on the influence of the variation in aggregate grading on the ASR performance of concrete. First, most ASR studies follow a specific mix design and a specific grading of coarse aggregate, and hence, the influence of the variation in aggregate grading is inadequately understood. Concomitantly, the CPT mix as per ASTM C1293 [14] cannot represent field mixes in which the mix design and the aggregate grading are different from ASTM C1293 [14] mix design [15]. Second, maintaining a specified grading is logistically challenging for ASR studies involving large test specimens, such as slabs, walls and exposure blocks [16–19]. Large concrete test specimens require a relatively large amount of aggregate, on a scale of several tons. The challenge of adhering to a specified grading is highlighted by the fact that a study “impact of aggregate gradation on properties of Portland cement concrete” [20] was conducted recently for the South Carolina Department of Transportation with the goal to determine whether concrete containing aggregate with an out-of-specification grading should be accepted or rejected. The study [20] investigated the influence of variations in aggregate grading on selected properties of concrete, such as compressive strength, modulus of elasticity, slump, split tensile strength and rapid chloride ion permeability. The study [20] suggested that, as long as the fresh properties of concrete are acceptable, any deviation in aggregate grading by ±12% may be acceptable for the strength performance of concrete; however, the effect of aggregate grading should be considered more seriously for the cracking and durability performance of concrete. Nevertheless, no consideration was given to the influence of aggregate grading on ASR performance.

Recently, an ASR research project required constructing several large specimens requiring some cubic meters of concrete [19]. This motivated a study to investigate the influence of coarse aggregate grading on the ASR performance of concrete. This study investigates the influence of aggregate grading on the expansion and damage of concrete due to ASR. Three mixes of concrete are compared with the primary variable being the coarse aggregate grading as: a standard composition as per ASTM C1293 [14]; a fine-dominant composition; and a coarse-dominant composition.
The investigation involves concrete prisms which were measured for expansion as per ASTM C1293 [14]. The prisms were tested for modulus of rupture and damage rating index at different ages from 28 to 365 days. In addition, a number of concrete cylinders were tested for compressive strength, modulus of elasticity, ultrasonic pulse velocity (UPV), and bulk resistivity at different ages from 28 to 365 days. This chapter presents the findings from these measurements to better understand the influence of aggregate grading on the alkali-silica reactivity of concrete. The effect of grading on the quality of concrete is also discussed. The importance of grading and the possible consequences of deviating from a standard grading are highlighted.

6.2 Materials and Methods

6.2.1 Coarse aggregate grading

The grading for the CPT as specified by ASTM C1293 [14] was taken as the standard grading. This study captures practical scenarios of likely deviations of the coarse aggregate grading compared to the specified ASTM C1293 [14] grading. Approximately in line with the previous study [20] which suggested an acceptable deviation in aggregate grading of ±12% based on the strength performance of concrete, this study considered a deviation in aggregate grading of ±10%. Three different gradings were considered in this study as depicted in Figure 6.1. The three gradings are described as:

1. Standard Grading: as specified in ASTM C1293 [14], which consisted of 33.33% each of the three size ranges of coarse aggregate, namely, a) 19.0–12.5 mm, b) 12.5–9.5 mm, and c) 9.5–4.75 mm;

2. Fine-Dominant Grading: which consisted of 10% less of the coarser fraction and 10% excess of the finer fraction compared to the standard grading; and

3. Coarse-Dominant Grading: which consisted of 10% excess of the coarser fraction and 10% less of the finer fraction compared to the standard grading.

The overall size range of 19.0–4.75 mm was maintained for all gradings.
6.2.2 Materials and concrete mix design

In general, all mixes were based on the CPT as per ASTM C1293 [14]. The three concrete mix designs are shown in Table 6.1. The primary difference between the three mixtures is the coarse aggregate grading. Accordingly, the three mixes were designated as the standard mix (mix S), fine-dominant mix (mix F) and coarse-dominant mix (mix C).

Spratt aggregate was used as the reactive coarse aggregate. It is a siliceous limestone, crushed aggregate from a quarry near Ottawa, Ontario, Canada, and has been used as a reference aggregate to calibrate ASR test methods [21]. The non-reactive fine aggregate was natural sand from Orillia, Ontario, Canada. The water-to-cement ratio was 0.44 for all mixes. High alkali general use (GU) cement was used with a total alkali content of 0.99% Na₂O equivalent by mass of cement. The chemical composition of cement is shown in Table 6.2. The alkali level of the concrete mixes was increased to 5.25 kg Na₂O equivalent per m³ of concrete by adding NaOH pellets to water prior to concrete mixing.

![Figure 6.1: Coarse aggregate grading scenarios](image)
Table 6.1: Mix designs of concrete

<table>
<thead>
<tr>
<th>Mix (Coarse aggregate grading)</th>
<th>Cement (kg/m³)</th>
<th>Water (kg/m³)</th>
<th>Coarse aggregate (kg/m³)</th>
<th>Sand (kg/m³)</th>
<th>NaOH pellets (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>19.0–12.5 mm</td>
<td>12.5–9.5 mm</td>
<td>9.5–4.75 mm</td>
</tr>
<tr>
<td>S (Standard)</td>
<td>420</td>
<td>184.8</td>
<td>371.67</td>
<td>371.67</td>
<td>371.67</td>
</tr>
<tr>
<td>F (Fine-dominant)</td>
<td>420</td>
<td>184.8</td>
<td>260.17</td>
<td>371.67</td>
<td>483.17</td>
</tr>
<tr>
<td>C (Coarse-dominant)</td>
<td>420</td>
<td>184.8</td>
<td>483.17</td>
<td>371.67</td>
<td>260.17</td>
</tr>
</tbody>
</table>

Note: The dry-rodded density of coarse aggregate for the standard grading was 1585 kg/m³. The mass of aggregates is shown for saturated surface-dry condition.

Table 6.2: Chemical composition of GU cement

<table>
<thead>
<tr>
<th>Constituents</th>
<th>LOI</th>
<th>SiO₂</th>
<th>Al₂O₃</th>
<th>Fe₂O₃</th>
<th>CaO</th>
<th>MgO</th>
<th>SO₃</th>
<th>Alkali (Na₂O_eq)</th>
<th>Free Lime</th>
<th>Insoluble residue</th>
</tr>
</thead>
<tbody>
<tr>
<td>Percentage</td>
<td>2.27</td>
<td>19.25</td>
<td>5.33</td>
<td>2.41</td>
<td>62.78</td>
<td>2.36</td>
<td>4.01</td>
<td>0.99</td>
<td>1.29</td>
<td>0.52</td>
</tr>
</tbody>
</table>

6.2.3 Casting and conditioning of the specimens

Concrete prisms were cast as per ASTM C1293 [14]. Nine prisms and nine cylinders were cast from mix F and mix C. Eighteen prisms and three cylinders were cast from mix S. The size of the prism specimens was 285 mm by 75 mm by 75 mm and that of the cylinders was 100 mm (diameter) by 200 mm (height). Specimens were demolded after one day of casting. After demolding, three cylinder specimens from mix S and one each from mix F and mix C were stored in a fog room maintained at room temperature (23±3 °C). Remaining specimens were stored in hermetically sealed plastic pails and were conditioned under a constant temperature of 38±2 °C and > 95% relative humidity. Prior to performing the tests on the specimens, the pails were acclimatized at room temperature for 16 to 20 hours.

6.2.4 Test details

6.2.4.1 Prisms

The concrete prisms were measured for longitudinal expansion. The initial reading for the longitudinal expansion of the prisms was taken at the age of one day. The prisms were subsequently measured for the longitudinal expansion at the ages of 7, 28, 56, 91, 182, 273 and 365 days. The prisms were tested for modulus of rupture at the ages of 28, 91, 182 and 365 days. Two prisms were tested by third-point loading as outlined in ASTM C78 [22].
Concrete slices from the prism specimens were examined using the damage rating index (DRI) method. The DRI method was originally proposed by Grattan-Bellew [23] to quantify the extent of damage in concrete suffering from ASR. The DRI method involves the judgment of an observer in classifying the different damage features and thus can involve some variability. In an attempt to minimize such variability, a revised version of the DRI method was proposed by Villeneuve et al. [24], which was adopted in this study. The method involves counting seven petrographic features that are shown, along with their weighting factors, in Table 6.3. Further reference to DRI analysis was made to the detailed guidelines on DRI published by Fournier et al. [25].

The DRI was analyzed on two prism specimens at each test age. For each prism, one slice (75 mm × 75 mm) near the middle and one slice near the end were taken and polished for examination under a stereo binocular microscope at ~16× magnification. The DRI value for concrete at an age was obtained as the average for four slices from two prisms.

<table>
<thead>
<tr>
<th>Petrographic features</th>
<th>Weighting factors</th>
</tr>
</thead>
<tbody>
<tr>
<td>Closed/tight cracks in coarse aggregate particle</td>
<td>0.25</td>
</tr>
<tr>
<td>Opened cracks or network cracks in coarse aggregate particle</td>
<td>2</td>
</tr>
<tr>
<td>Cracks or network cracks with reaction product in coarse aggregate particle</td>
<td>2</td>
</tr>
<tr>
<td>Debonded coarse aggregate</td>
<td>3</td>
</tr>
<tr>
<td>Disaggregated / corroded aggregate particle</td>
<td>2</td>
</tr>
<tr>
<td>Cracks in cement paste</td>
<td>3</td>
</tr>
<tr>
<td>Cracks with reaction product in cement paste</td>
<td>3</td>
</tr>
</tbody>
</table>

6.2.4.2 Cylinders

The cylinders conditioned at 38 °C were non-destructively tested for UPV in accordance with ASTM C597 [26], and for bulk resistivity on a monthly basis until 1 year. UPV was measured along the longitudinal axis of the cylinders using “Pundit Lab+” equipment from Proseq USA, Inc. The cylinders were destructively tested for static modulus of elasticity in accordance with ASTM C469 [27] and compressive strength in accordance with ASTM C39 [28] at 28, 91, 182
and 365 days. The cylinders conditioned at 23 °C were tested for the static modulus of elasticity [27] and the compressive strength [28] at the age of 28 days.

Bulk resistivity was tested by using the RCON equipment from Giatec Scientific Inc., Canada. The resistivity of a concrete cylinder specimen was measured across the two flat ends of the cylinders by the uniaxial method [29]. The cylinder specimen was placed between two parallel metal plates with moist sponge contacts at the interfaces. An AC current (1 kHz frequency) was applied, and the electrical resistivity was measured in terms of impedance and phase angle. For a relatively smaller phase angle, which was measured as 0 to 3°, the impedance was approximated as the resistance of the concrete cylinder. The resistivity was then calculated as the measured resistance multiplied by the cross-sectional area of the cylinder specimen and divided by its length.

6.3 Results and Discussions

6.3.1 Longitudinal expansion

Longitudinal expansions of the concrete prisms for the three mixes are presented in Table 6.4. Since two prisms were destructively tested at each test date, the number of prisms used for expansion measurement gradually decreased over time. Therefore, each expansion result shown in Table 6.4 represents the average of a varying number of prisms. For mix S, the expansion at 7 days and 365 days represents the average of eighteen prisms and six prisms, respectively. For mix F and mix C, the concrete expansion at 7 days and 365 days represents the average of nine prisms and three prisms, respectively.

The expansion for mix F was significantly greater than that for mix S by approximately 50% at all ages ranging from 91 to 365 days. For mix C, designed with relatively coarser aggregates, the expansion was consistently slightly greater than that of mix S. The expansion for mix C ranged from 2 to 15% greater than the expansion for mix S at all ages except 7 and 28 days. Based on the Student’s t-test performed at a 95% confidence level, the expansion for mix C was statistically significantly greater than the expansion for mix S at all ages ranging from 56 to 365 days except at 273 days. The influence of aggregate grading on ASR expansion will be further discussed in Section 6.3.8.
Table 6.4 shows the coefficient of variation of the expansion measurements for the prism specimens of the three mixes. The coefficient of variation was less than 6% for the mix S specimens after 56 days. (The large coefficient of variation before 56 days is attributed to the extremely small length changes.) In comparison to the mix S specimens, larger coefficients of variation were generally observed for the mix F and mix C specimens.

Table 6.4: Mean longitudinal expansion and coefficient of variation for the concrete prisms from the three mixes

<table>
<thead>
<tr>
<th>Age (days)</th>
<th>Mean longitudinal expansion (%)</th>
<th>Coefficient of variation (%)</th>
<th>No. of specimens</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mix S</td>
<td>Mix F</td>
<td>Mix C</td>
</tr>
<tr>
<td>1</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>7</td>
<td>0.003</td>
<td>0.006</td>
<td>0.002</td>
</tr>
<tr>
<td>28</td>
<td>0.013</td>
<td>0.009</td>
<td>0.008</td>
</tr>
<tr>
<td>56</td>
<td>0.048</td>
<td>0.060</td>
<td>0.055</td>
</tr>
<tr>
<td>91</td>
<td>0.101</td>
<td>0.153</td>
<td>0.116</td>
</tr>
<tr>
<td>182</td>
<td>0.171</td>
<td>0.252</td>
<td>0.192</td>
</tr>
<tr>
<td>273</td>
<td>0.210</td>
<td>0.315</td>
<td>0.215</td>
</tr>
<tr>
<td>365</td>
<td>0.223</td>
<td>0.350</td>
<td>0.242</td>
</tr>
</tbody>
</table>

The statistically significant difference between the expansions of the mix S prisms and the mix F or mix C prisms highlights the importance of grading in performing CPT or other expansion tests. It also indicates that an error in grading could be one of the factors that may cause variations in expansion results among different studies, including multi-laboratory studies.

6.3.2 Damage rating index

Figure 6.2 shows the results of DRI analysis from 28 to 365 days for the three concrete mixes. As expected, all mixes showed an increase in DRI value with age. Statistically significantly greater values of DRI were observed for mix F and mix C in comparison to mix S. However, the greatest DRI was always observed for mix F. The possible contributing factor for the greatest DRI value for Mix F is that the DRI method involves counting the number of cracks which becomes greatest for mix F containing the greatest number of reactive coarse aggregate particles for the same mass of reactive coarse aggregate as with mix S and mix C. The DRI value for mix...
S increased from 123 at 28 days to 614 at 365 days. For mix F, DRI value increased from 238 at 28 days to 1411 at 365 days. The DRI value for mix F was close to double that of mix S. The DRI value for mix C increased from 191 at 28 days to 1157 at 365 days. DRI values for the fine-dominant and coarse-dominant concrete mixes were, respectively, 86 to 130% and 56 to 98% larger compared to the standard mix.

Figure 6.2: DRI and the contribution of the seven features of DRI from the three mixes (S – standard; F – fine-dominant, and C – coarse-dominant)
The damage of concrete for all mixes evolved from the occurrence of ‘closed/tight cracks’. The contribution of ‘closed/tight cracks’ to the DRI value decreased from 28 to 365 days for all mixes. In terms of the relative contributions of the seven features of DRI, as shown in Table 6.3, for all mixes the contribution of ‘closed/tight cracks’ to the DRI value was 37-48% at 28 days but diminished to 2-5% by 365 days. The contribution of ‘opened cracks’ was consistently larger in mix F than in the other two mixes. The contributions of ‘debonded aggregate’ and ‘disaggregated particles’ were relatively larger in mix F and mix C compared to mix S. The contribution to the DRI value by the cracks in cement paste, with and without reaction product, was greatest in mix F and least in mix S. The combined contribution of the two types of paste cracks (‘cracks in cement paste’ and ‘cracks with reaction product in cement paste’) consistently increased from 28 to 365 days for all the mixes. The DRI values associated with paste cracks increased from 0 to 113 for mix S, from 17 to 302 for mix F, and from 11 to 242 for mix C. In terms of the percentage contribution to the DRI value, the increase in paste cracks’ contribution from 28 to 365 days was from 0 to 18%, 7 to 27% and 6 to 21%, respectively, for mix S, mix F, and mix C.

A large difference was observed among the DRI values for the three mixes with three gradings of reactive coarse aggregate. Thus, the variation in aggregate grading, despite no other changes in the mix design of concrete, was seen to influence the ASR damage of concrete. Mix F exhibited greater damage than mix C.

Figure 6.3 compares the evolution of expansion and DRI in the prism specimens for the three mixes. Both the expansion and the DRI values increased with age and the overall trend of expansion and DRI were similar. Comparisons of expansion and DRI results demonstrate that any deviation in aggregate grading from the standard grading markedly increases the expansion and damage due to ASR. The DRI values were consistently the lowest for mix S and the greatest for mix F beyond an age of 91 days. Both the expansion and the DRI for mix F were larger than for mix C and mix S.
6.3.3 Modulus of rupture

Figure 6.4 shows the modulus of rupture of the concrete prism specimens from the three mixes with three different gradings of reactive coarse aggregate. The modulus of rupture at 28 days for both mix F and mix C was less than that for mix S. While the 28-day value was approximately equal (within 2%) for the mix F and mix C specimens, it was approximately 30% lower when compared to the mix S specimens. The lower modulus of rupture at 28 days for mix F and mix C compared to mix S is attributed to the effect of aggregate grading.

With exposure to 38 °C and > 95% relative humidity, the modulus of rupture was markedly affected by ASR for all mixes. Most of the reduction in the modulus of rupture occurred within 91 days when ASR expansion was less than 50% of the 1-year expansion. (The expansions for mix S, mix F and mix C were, respectively, 0.101%, 0.153% and 0.116% at 91 days and 0.223%, 0.350% and 0.242% at one year). This is attributed to the fact that the reduction in modulus of rupture is due to micro-cracking that can occur even before expansion [30]. However, there was no linear trend for the reduction in modulus of rupture with age. In fact, the maximum reductions in modulus of rupture were observed at various ages depending on the mix designs. The
maximum reduction occurred in 91 days, 182 days and 365 days, respectively, for the mix F, mix C and mix S specimens. With reference to the 28-day value, the maximum reduction in modulus of rupture was 60% for mix F, 58% for mix S, and 53% for mix C. The largest and the most rapid reduction occurred in the mix F specimens. The mix C specimens had slightly smaller reduction in the modulus of rupture perhaps due to increased interlocking provided by the coarse aggregates. The modulus of rupture values for all three mixes were approximately equal at the age of one year.

Figure 6.4: Modulus of rupture of concrete prisms for Mix S, F and C conditioned at 38 °C

### 6.3.4 Modulus of elasticity

The modulus of elasticity of mix F and mix C specimens is presented in Figure 6.5. Furthermore, the 28-day modulus of elasticity of a reference cylinder conditioned at room temperature (23 °C) was 37 GPa for mix F and 38 GPa for mix C. As shown in Figure 6.5, the 28-day modulus of elasticity for the cylinders conditioned at 38 °C was smaller than the corresponding value for the cylinder conditioned at 23 °C, thus, indicating that the static modulus of elasticity was affected by ASR since its initiation. The maximum reduction in the modulus of elasticity was observed at 182 days when the reduction with respect to the reference value at 28 days was 36% and 34%,
respectively, for mix F and mix C. The modulus of elasticity for mix F was always smaller than for mix C even though it was not statistically significant at a 95% confidence level.

![Note: The error bar represents one standard deviation.](image)

Figure 6.5: Static modulus of elasticity of concrete cylinders from mix F and mix C conditioned at 38 °C

### 6.3.5 Compressive strength

The compressive strength of the mix F and mix C specimens is presented in Figure 6.6. The compressive strength of mix F was statistically significantly smaller than the compressive strength of mix C at all ages, based on a t-test at 95% confidence level. On average, the mix C specimens were 14 to 25% stronger than the mix F specimens. Irrespective of the ASR, the consistently larger strength of mix C than mix F may be due to the strength enhancement by the larger (19.0–12.5 mm) aggregates, which had the largest proportion in mix C. This is supported also by the comparison between mix C and mix S. For the cylinders conditioned at 23 °C and tested at 28 days, the compressive strength for mix S was 38.2 MPa, significantly smaller than the compressive strength of 42.1 MPa for mix C.
Compressive strength (MPa)

Age (days)

28 (23 °C) 28 91 182 365

Note: The error bar represents one standard deviation.

Figure 6.6: Compressive strength of concrete cylinders from mix F and mix C conditioned at 38 °C

Compared to the 28-day strength of a cylinder conditioned at 23 °C, the same-aged cylinders conditioned at 38 °C showed an increased compressive strength. The increase in compressive strength continued until 91 days after which a reduction in compressive strength was observed. An initial increase in compressive strength for lower expansion followed by a reduction in compressive strength corresponding to a greater expansion has also been previously observed [31] and is further investigated in Chapter 7. With respect to the 28-day compressive strength of the reference cylinder conditioned at 23 °C, the maximum reduction of 6% for mix F and 4% for mix C was observed at 365 days. While the extent of reduction is small and the trend of reduction is similar for both mix F and mix C, both the compressive strength and its reduction appear more sensitive to the mix dominated with relatively finer aggregate than for the mix dominated with relatively coarser aggregate.

6.3.6 UPV of cylinders

Figure 6.7 compares the UPV of concrete over time as measured in the cylinders from mix F and mix C. Even though the difference in UPV was not statistically significant at 95% confidence
level, UPV was always smaller for mix F than for mix C. The decrease in velocity over time was not significant. However, a slightly larger decrease was observed for mix F than for mix C. UPV method appears not a sensitive method for monitoring the ASR damage of laboratory concrete specimens as discussed in Chapter 4 and in Appendix G.

![Graph showing UPV of concrete cylinders from mix F and mix C](image)

**Figure 6.7: UPV of concrete cylinders from mix F and mix C**

### 6.3.7 Bulk resistivity

Figure 6.8 compares the evolution of the bulk resistivity of concrete from 28 to 365 days measured in the cylinders from mix F and mix C. The measurements were taken for an input AC current frequency of 1 kHz. As shown in Figure 6.8, the effect of ASR was not evident, particularly with mix F. The bulk resistivity remained almost constant up to 273 days, and then increased by approximately 25% of the 273-day value at 365 days. The bulk resistivity of mix F was consistently less than that of mix C by approximately 20%. The bulk resistivity of mix F was statistically significantly smaller than the bulk resistivity of mix C, except at 9 and 10 months, at a 90% confidence level. While resistivity results were not sensitive to the effect of ASR on the concrete mixes, the consistently smaller resistivity values for mix F indicated that the mix dominated with relatively finer aggregate was more conductive to electrical charges. Even
though not a direct measure of permeability, the lower resistivity of mix F consequently indicated that the mix dominated with relatively finer aggregate was more permeable [29] than the mix dominated with relatively coarser aggregate.

![Figure 6.8: Bulk resistivity of concrete cylinders from mix F and mix C](image)

**Note:** The error bar represents one standard deviation.

6.3.8 **Discussion on the effect of grading**

The comparison of ASR expansions of concrete made with different gradings showed a pronounced impact of aggregate grading. The deviation towards a fine-dominant grading resulted in a substantially greater expansion in as early as 91 days and remained consistently greater thereafter to reach 0.35%. A deviation in specified grading by 10% could result in 50% deviation in the ASR expansion. Keeping the same mass of reactive aggregate but of finer size allows for a greater number of smaller aggregate particles resulting in an increased number of reaction sites available for the ASR. However, mix C, with coarse-dominant grading, also showed increased expansion and DRI value compared to mix S with standard grading. Greater expansion for both mix F and mix C than for mix S suggests that rather than the effect of the exposure of more
reaction sites, the change in concrete matrix associated with the deviation in aggregate grading from the standard grading was more influential to the expansion of concrete.

Based on the tests of concrete properties, mix F had lower mechanical properties compared to mix C. Mix F had lower modulus of elasticity, UPV, bulk resistivity and statistically significantly lower compressive strength. Similarly, the reductions in modulus of rupture, modulus of elasticity and compressive strength were larger for mix F than for mix C. For a particular level of expansive pressure, a lower modulus of elasticity allows concrete to undergo larger deformation and a lower tensile strength (indicated by the modulus of rupture) allows concrete to experience more cracking. Moreover, the relatively fewer larger particles in mix F are expected to have resulted in relatively less interlocking effect associated with the coarse aggregates, which is indicated by the lowest modulus of rupture of the mix F specimens. These mechanisms are intuitively in agreement with approximately 50% larger expansion of the mix F specimens compared to the mix C specimens.

A slightly larger expansion (for mix C than for mix S) but a markedly larger DRI value indicates that the ASR damage is revealed more effectively by the DRI method than by expansion measurement. Expansion is the external manifestation of the internal chemical reaction. It cannot effectively reveal the influence of the skeleton of concrete which is governed by the coarse aggregate. However, the DRI method assesses the interior of concrete, and thus, portrays the ASR damage more effectively.

Large variations in the expansion and damage of concrete were observed by merely varying the aggregate grading while having an identical paste volume and an identical proportion of reactive coarse aggregate. Hence, the grading of coarse aggregate should be regarded as an important parameter in the study of the reactivity of aggregate and the ASR performance of concrete.

6.4 Summary and Concluding Remarks

This study investigated the effects of variation in the grading of reactive coarse aggregate, Spratt, on the ASR expansion and damage of concrete. Three mixes of reactive concrete were studied by considering three coarse aggregate gradings, namely: 1) a standard grading with 33.33% each of the three size ranges, a) 19.0–12.5 mm, b) 12.5–9.5 mm, and c) 9.5–4.75 mm, as per ASTM C1293 [14]; 2) a fine-dominant grading which consisted of 10% less of the coarser fraction and
10% excess of the finer fraction compared to the standard grading; and 3) a coarse-dominant grading which consisted of 10% excess of the coarser fraction and 10% less of the finer fraction compared to the standard grading.

A deviation in specified grading of coarse aggregate by 10% towards the finer size resulted in 50% deviation in the expansion measurement compared to the standard mix. This was attributed partly to the weaker mechanical properties and relatively less aggregate interlocking effect in the fine-dominant mix compared to the standard mix. DRI results demonstrated that any deviation in aggregate grading from the standard grading significantly increases the damage due to ASR. DRI values for the fine-dominant and coarse-dominant concrete mixes were, respectively, 86 to 130% and 56 to 98% larger compared to the standard mix. Also, more ‘paste cracks’ were observed in the prisms made of fine-dominant mix compared to the standard mix.

Over a 1-year duration, modulus of rupture of the reactive concrete prisms was reduced due to ASR by 53–60%, irrespective of the type of coarse aggregate grading used. The fastest reduction occurred for the fine-dominant mix. The coarse-dominant mix showed a slightly smaller reduction in the modulus of rupture indicating increased interlocking provided by the coarse aggregates.

The static modulus of elasticity was affected by ASR since its initiation. The maximum reduction in the modulus of elasticity of the fine-dominant and the coarse-dominant mixes was approximately 35%.

The compressive strength for the coarse-dominant mix was the largest among the three mixes indicating that the larger aggregates helped in enhancing the strength of concrete. Compared to the UPV of the coarse-dominant mix, the UPV of the fine-dominant mix was always smaller and showed larger reduction due to ASR. Bulk resistivity of concrete indicated that the fine-dominant grading resulted in a mix that was more permeable than the coarse-dominant mix.

As revealed by the expansion measurements, DRI and various properties of the three concrete mixes, the variation of the coarse aggregate grading alone significantly influenced the ASR expansion and damage of concrete. The fine-dominant grading was worse than the coarse-dominant grading. The findings highlight the importance of grading in performing CPT or other studies investigating the ASR performance of concrete mixtures, and also in minimizing the ASR
expansion of concrete structures. The quality control in maintaining a specified grading of coarse aggregate should be stringent in a project associated with assessing the ASR performance of concrete.

Any deviation in coarse aggregate grading on either side of the standard ASTM C1293 [14] grading resulted in more expansion and more damage. This observation indicates that an aggregate that is concluded as innocuous based on the concrete prism test, such as ASTM C1293 [14], may still show marked expansion and damage in concrete members in which the aggregate grading is widely different from the ASTM C1293 [14] grading. Specifications related to concrete aggregates for field concrete should consider the influence of aggregate grading on the ASR performance of concrete. Similar studies with a variety of reactive aggregates is expected to increase the understanding of the effect of coarse aggregate grading on the ASR performance of concrete.

6.5 References


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Chapter 7
The Mechanisms Behind the Partial Recovery in the Degraded Mechanical Properties of ASR-Affected Concrete

Abstract

Alkali-silica reaction (ASR) is an undesirable chemical reaction between the alkali hydroxides from the concrete pore fluid and a particular type of aggregates that contain reactive siliceous minerals. ASR causes expansion, cracking and degradation of mechanical properties of concrete. However, the degradation of mechanical properties appears to be characteristically followed by a partial recovery despite an ongoing increase in expansion. This chapter is focused on discussing the mechanism behind the partial recovery. Firstly, the mechanisms of how ASR affects the mechanical properties, namely, compressive strength, modulus of elasticity, and tensile strength is examined. Secondly, concrete samples made with reactive aggregate are microscopically examined to compare the cracks, debonding, and reaction products at different ages of accelerated curing. Images are captured from polished surfaces of concrete samples and are analyzed to understand the changes in the microstructure of concrete. The chemical compositions of the reaction products at different stages of reaction and at different locations within concrete are investigated by scanning electron microscope (SEM) backscattered electron imaging and SEM elemental mapping. Findings from the present experimental study are synthesized with the literature findings to postulate the mechanism of partial recovery in the mechanical properties.

Keywords: alkali-silica reaction; longitudinal expansion; mechanical properties; degradation; partial recovery

7.1 Introduction

Alkali-silica reaction (ASR) is a major deterioration problem in concrete structures. It causes expansion, cracking and degradation of mechanical properties of concrete. Degradation of mechanical properties is a critical concern since loss in strength and stiffness can jeopardize the structural integrity of concrete structures. Many studies have focused on the loss in mechanical properties of concrete due to ASR and have shown that different mechanical properties have different sensitivities to ASR [1–3].
Swamy and Al-Asali [1] reported that the loss of modulus of rupture, modulus of elasticity and compressive strength can be as much as 80%, 70% and 60%, respectively, due to ASR. A peculiar trend however was noted for the compressive strength, it increased until a longitudinal expansion of approximately 0.12% was reached and decreased thereafter. In contrast, the loss in tensile strength began from the onset of the reaction and was severe. In a study of specimens exposed to alkali solution for 12 weeks at 80 °C, compressive strength of reactive normal strength (nominal 35 MPa) concrete reduced by 24% and increased by up to 23% for reactive high strength (nominal 80 MPa) concrete [4]. Multon et al. [5] observed a loss of 20% in modulus of elasticity and approximately 10% increase in compressive strength after 1 year of testing reactive concrete at 38 °C. Giaccio et al. [6] reported that compressive strength of reactive concrete cylinders generally increased until the longitudinal expansion reached 0.15% but decreased thereafter. The authors [6] reported that fine cracks were unable to influence the compressive strength but able to degrade the modulus of elasticity by as much as 50%. Jones and Clark [3] observed reduction in compressive strength only after the ASR expansion reached 0.17%. The loss in modulus of elasticity was as large as 70% and the loss in tensile strength was up to 80%. Even without noticeable opening of micro-cracks, a substantial reduction in tensile strength was observed [3].

These studies suggest that the mechanisms by which different mechanical properties of concrete are affected by ASR are different. Generally, the loss in tensile strength occurs as soon as micro-cracking develops in concrete but the loss in compressive strength does not occur before the cracks have attained some width and concrete has undergone an expansion exceeding approximately 0.12-0.17%. In the case of accelerated laboratory tests, the high temperature of an accelerated curing condition contributes to the continued hydration of cement [4,5,7]. Since the mechanisms and rates of degradation vary for different mechanical properties and since the degradation is often coupled with the improvement caused by the continued hydration of cement, an understanding of the degradation mechanisms becomes complicated. Moreover, the reaction products of ASR undergo chemical and physical changes over time [8–11]. Thus, it is sometimes difficult to interpret the experimental results aimed at investigating the loss of mechanical properties of concrete due to ASR. One particular aspect is that of partial recovery, a slight improvement in the mechanical properties following a loss due to ASR. The change in different mechanical properties was also observed in the reactive concrete specimens in this study. Table
7.1 shows the evolution of compressive strength, modulus of elasticity and splitting tensile strength of reactive concrete cylinders subjected to an accelerated curing environment of 50±0.5 °C and > 95% relative humidity.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value (mean ± standard deviation based on three cylinders)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Ref. (at 23±3 °C before accelerated curing) 3 months of accelerated curing 8 months of accelerated curing 12 months of accelerated curing</td>
</tr>
<tr>
<td>Compressive strength (MPa)</td>
<td>43.9 ± 1.6 54.0 ± 1.6 53.5 ± 1.9 54.8 ± 2.2</td>
</tr>
<tr>
<td>Modulus of elasticity (GPa)</td>
<td>38.2 ± 0.3 29.6 ± 0.7 31.6 ± 1.8 32.2 ± 2.5</td>
</tr>
<tr>
<td>Splitting tensile strength (MPa)</td>
<td>3.0 ± 0.1 2.8 ± 0.2 1.5 ± 0.05 2.3 ± 0.03</td>
</tr>
</tbody>
</table>

Note: the accelerated curing was performed at 50±0.5 °C and > 95% relative humidity.

Partial recovery has been observed in many studies [1,12,13]. Swamy and Al-Asali [1] observed partial recovery in the tensile strength and the dynamic modulus of elasticity of reactive concrete specimens and attributed it to the effect of the continued hydration of cement paste. Bektas and Wang [12] observed partial recovery in the modulus of rupture and modulus of elasticity in the reactive concrete specimens. The authors attributed the recovery due to the filling of cracks and voids by the reaction product. With time, the soft and fluid alkali-rich gel becomes stable through alkali-calcium exchange and is stronger than its predecessor alkali-rich gel.

Since the mechanism of partial recovery appears as a characteristic phenomenon in the laboratory investigation of ASR-affected concrete specimens, this study is dedicated to examine the mechanism of partial recovery. As the mechanism of recovery follows the mechanism of damage due to ASR, this study microscopically examines the mechanism of damage due to ASR. Optical and digital microscope images captured from a number of polished surfaces of concrete samples are analyzed. The chemical compositions of the reaction products at different stages of reaction and at different locations within concrete are investigated by scanning electron microscope (SEM) backscattered electron (BSE) imaging and SEM elemental mapping. The effect of fluid pressure of the reaction products, the physical and chemical changes of reaction products, and the analysis of cracks and microstructure of concrete are utilized to elucidate the mechanism of partial recovery.
7.2 Materials and Methods

7.2.1 Concrete mixes and specimens

This study considered three reactive concrete mix designs made with reactive coarse aggregate and non-reactive fine aggregate as shown in Table 7.2. Spratt aggregate from a quarry near Ottawa, Ontario, Canada was used as the reactive coarse aggregate. In addition to the three reactive mixes, a non-reactive concrete mix, with mix design similar to that of Mix S but using non-reactive coarse aggregate, was used to cast control specimens. The alkali level of the concrete mixes was 5.25 kg Na₂O equivalent per m³ of concrete.

This experimental study involved tests on prism, cylinder and cube specimens made from reactive concrete. The size of the prism specimens was 285 mm by 75 mm by 75 mm and that of the cylinders was 100 mm (diameter) by 200 mm (height). The size of the cube specimens was 254 mm. The specimens were subjected to an accelerated curing condition of > 95% relative humidity and an elevated temperature of either 38±2 °C or 50±0.5 °C. Some cylinder specimens were cured at room temperature (23±3 °C) and > 95% relative humidity. All the tests on the specimens conditioned at elevated temperature were performed at room temperature by acclimatizing the specimens for 16 to 24 hours before testing.

Table 7.2: Mix designs of reactive concrete

<table>
<thead>
<tr>
<th>Mix</th>
<th>Cement (kg/m³)</th>
<th>Water (kg/m³)</th>
<th>Reactive coarse aggregate (kg/m³)</th>
<th>Non-reactive sand (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>19.0–12.5 mm</td>
<td>12.5–9.5 mm</td>
</tr>
<tr>
<td>S</td>
<td>420</td>
<td>184.8</td>
<td>371.7</td>
<td>371.7</td>
</tr>
<tr>
<td>F</td>
<td>260.2</td>
<td>371.7</td>
<td>483.2</td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>483.2</td>
<td>371.7</td>
<td>260.2</td>
<td></td>
</tr>
</tbody>
</table>

7.2.2 Test details

7.2.2.1 Mechanical properties

Concrete prisms were measured for the longitudinal expansion from day 1 to 2 years. Two prisms were tested by third-point loading as outlined in ASTM C78 [14] at different ages from 14 days to 2 years. The prisms were also measured for fundamental transverse resonant frequency as per ASTM C215 [15], as a non-destructive method of estimating the dynamic
modulus of elasticity. Cylinders were tested for static modulus of elasticity in accordance with ASTM C469 [16] and compressive strength in accordance with ASTM C39 [17].

7.2.2.2 Microscopic examination

Concrete samples from selected prism and cube specimens were taken for microscopic examination. Slices were prepared for examination under stereo-binocular microscope by successively polishing to a surface roughness of approximately 5 µm by using a diamond grinder and water cooling. Thin sections were prepared for examination under an optical microscope and an SEM. Kerosene-based processes were performed to prepare the thin sections so as to avoid any influence of water.

7.3 Mechanical Properties

Figure 7.1a shows the variation in modulus of rupture and dynamic modulus of elasticity with longitudinal expansion of concrete prisms from Mix S conditioned at 38 °C and 50 °C. The mechanical properties are presented on a scale normalized to the corresponding property when the ASR expansion was quite small (~0.01%). ASR caused a loss in modulus of rupture by approximately 60% and a loss of approximately 25% in dynamic modulus of elasticity. As observed in Figure 7.1a, the loss in mechanical property (modulus of rupture (MOR) and dynamic modulus of elasticity (E_{dyn})) was followed by a partial recovery after the longitudinal expansion exceeded approximately 0.2%. Accordingly, the reaction is divided into two stages, namely the early stage and the later stage, by a line corresponding to a longitudinal expansion of 0.2%. Degradation of mechanical properties was prevalent in the early stage and the mechanism of partial recovery was effective in the later stage. While the recovery in dynamic modulus of elasticity was approximately 10%, the recovery in modulus of rupture was 34%, when compared to the lowest value (at maximum loss). The extent of recovery is far greater than the improvement in mechanical properties due to the effect of continued hydration. The improvement, as measured on control prisms, had been observed as 7% in dynamic modulus of elasticity and 0% in modulus of rupture.

Figure 7.1b shows the variation in compressive strength and static modulus of elasticity of concrete cylinders from Mix F and Mix C, both conditioned at 38 °C. It should be noted that since expansion was not measured on cylinders, the longitudinal expansion in Figure 7.1b is of
the concrete prisms from the corresponding mixes and cured identically to the cylinders. Compressive strength increased until an expansion of approximately 0.15% and experienced a loss thereafter for both Mix F and C. Static modulus of elasticity was observed to reduce from the onset of measurable expansion until approximately 0.2% expansion at which time the mean static modulus appears to remain statistically similar despite the increase in longitudinal expansion to 0.35%.

![Figure 7.1](image_url)

Figure 7.1: Loss followed by partial recovery in the mechanical properties of concrete affected by ASR

### 7.4 Microscopic Analysis of the Mechanism of Partial Recovery

#### 7.4.1 Early stage: fluid reaction products and pore pressure

Figures 7.2a and 7.2c show the 2D images and Figures 7.2b and 7.2d show the respective 3D images of concrete surfaces showing exudation of reaction product from cracks. The surfaces were polished to a surface roughness of approximately 5 µm. The exudation took place within 3-4 days after polishing the surfaces that had been prepared for a microscopic analysis (some influence of additional water supply during polishing might be possible). As shown in Figures 7.2b and 7.2d, a grid line drawn for the microscopic analysis was raised due to the swelling product. The exudation was as high as 390 µm as shown in Figure 7.2b. The concrete samples had been taken from a concrete cube specimen at the early stage of the reaction. The images indicate that when ASR is active inside the specimen, the reaction products are considerably fluid and can generate a relatively large fluid pressure. As a reference, Ichikawa and Miura [18]...
showed that the chemical reaction of ASR can possibly produce an expansive pressure of 400 MPa. It is intuitive that the large pressure within concrete causes the reaction product to enter into micro-cracks and capillary pores and to exude from the cracks that are exposed.

Figure 7.2: Exudation of reaction product from cracks on polished surfaces of concrete

The fluid pressure within the concrete microstructure influences the mechanical properties of concrete. During the early stage of the reaction, the pressure built-up inside concrete can provide some resistance to externally applied compressive loads through fluid pressure [19]. Thus, even if concrete has developed micro-cracking, its effect may not be reflected in the compressive strength of concrete. However, owing to its hydrostatic nature, the fluid pressure will not have a positive contribution to the tensile strength or the modulus of elasticity of concrete. The pressure will rather contribute to accelerate the process of cracking when concrete is subjected to tensile stress. Thus, as shown in Table 7.1, a sharp degradation of tensile strength is observed during the early stage of ASR in concrete specimens, but no degradation of compressive strength is noticed.
7.4.2 Later stage: Filling of cracks and interstitial transition zone (ITZ) with reaction products and transformation into solid products

At the later stage of the reaction, the rate of formation of reaction product will be low and there will be low affinity for water even though the accumulated pressure continues to expand concrete [20]. The fluid reaction products gradually transform into solid products.

Figure 7.3 compares two instances of aggregate debonding typically observed during performing a microscopic examination. Empty debonding cracks (Figure 7.3a) were commonly observed during the early stage of the reaction, and the filled debonding cracks (Figure 7.3b) were commonly observed during the later stage of the reaction. The surfaces were prepared by polishing with water as detailed in Section 7.2.2.2. During the early stage, as shown in Figure 7.3a, the fluid reaction product was mostly washed away. In contrast, during the later stage, the reaction product was already hardened into solid product in the debonded crack as shown in Figure 7.3b. This observation supports that during the early stage, many of the micro-cracks and debonding cracks contained fluid reaction product that did not have role on the tensile strength and modulus of elasticity of concrete but imparted to the compressive strength of concrete.

![a) Empty debonding crack](image1)

![b) Filled debonded crack](image2)

Figure 7.3: Two instances of aggregate debonding

Figure 7.4 shows SEM images of ASR gel present within and around aggregate particles from concrete specimens during the later stage of the reaction. Figure 7.4a shows an interstitial transition zone (ITZ) filled with solid reaction products. Figure 7.4b shows a crack which indicates that the reaction product might have been solidified during the later stage of the reaction. With formation of solid products and increased macroscopic expansion of concrete, the fluid pressure is gradually diminished which can no longer contribute to resist compressive
stress. Accordingly, concrete can experience a loss in compressive strength as shown in Figure 7.1b. However, when the fluid gradually transforms to solid phase, the solid products will contribute in bridging the cracks in concrete (Figure 7.4b, d and e) and thus sharing some stress. While both tensile and compressive stresses can be supported, any support to tensile stress can be significant owing to relatively low tensile strength of concrete. But any support to compressive stress may still be too low compared to that previously provided by the fluid pressure. Depending on the extent of crack bridging, the modulus of elasticity can also experience partial recovery. Figure 7.4d shows the ITZ being partially bridged with calcium hydroxide crystals. The bridging provided by calcium hydroxide along the ITZ is particularly attributed to improve the modulus of elasticity of concrete [21].

Figure 7.4e shows an aggregate crack that has been partially bridged by fibrous ASR product. Figure 7.4f shows the energy-dispersive x-ray spectrum of a small region at location “3” (which is identified in Figure 7.4e) within the fibrous product. The spectrum of the fibrous product is typical for ASR product [22]. The fibrous product appears bridging the crack to some extent while the crack was getting wider. This suggests that solidified ASR product may contribute to partially recover the loss of some mechanical properties.

Figure 7.5 shows the microscopic images and SEM elemental mapping for an ASR crack spanning two aggregate particles and traversing through the paste matrix. The elemental maps distinctly show that the ASR product in the crack within the aggregate particles is relatively free of calcium and becomes rich in calcium when traversing the paste matrix. While traversing through the paste matrix, the sodium and potassium veins are getting thinner and the crack becomes richer in calcium. As can be seen clearly on the fluorescent image (Figure 7.5f), the microstructure around the vicinity of crack at the paste matrix has become denser when compared to the microstructure of the paste matrix farther from the crack. Figures 7.5e–g show that calcium from the paste matrix near the crack was attracted towards the crack to get associated with the ASR product. This indicates that ASR products in the paste matrix are gradually associated with calcium to form a denser and a non-swelling lime-alkali-silicate complex that imparts strength to the microstructure of ASR-affected concrete [12]. Leemann and Lura [11] showed that uptake of calcium by ASR product can increase the modulus of elasticity from 10 GPa to 45 GPa. Phair et al. [23] demonstrated that the elastic modulus of ASR product synthesized by dissolving calcium hydroxide to sodium silicate solution increased from a range
of 4–8 GPa to a range of 16-25 GPa when the concentration of Ca(OH)$_2$ was increased from 0.08 M to 0.8 M. The increase in the modulus of elasticity was attributed to the formation of complex silica species and an inter-connected silica network promoted by the increased concentration of calcium. Thaulow et al. [10] have shown that irrespective of the location of the reaction product, they tend to be richer in calcium with age.

Figure 7.4: BSE image of ASR cracks within and around aggregate particles in concrete specimens cured at 50 °C
These studies [10-12,23] suggest that the association of calcium with the ASR product in the paste crack contributes to improve the mechanical properties of concrete. While alkalis are reported to be recycled to some extent after being substituted by calcium (especially in relatively thicker concrete structures) [24,25], they are also likely to be leached in relatively smaller concrete specimens in the accelerated laboratory studies typical to this study [e.g. 26]. Hence, with age, the alkali-silica reaction is expected to subside, and the gradual association of calcium with the ASR products attempts to repair the cracked microstructure of concrete, thus, demonstrating a partial recovery in some of the mechanical properties that were damaged by ASR. Even though partial recovery can be attained more distinctly by the tensile strength and the modulus of elasticity, continuous improvement in the microstructure may ultimately lead to have some partial recovery even in the compressive strength of concrete, as can be noticed in some results on compressive strength by Swamy and Al-Asali [1].

7.5 Conclusion

Samples of ASR-affected concrete were microscopically analyzed to investigate the mechanism of partial recovery that has been observed in many laboratory studies to characteristically follow the loss of some mechanical properties due to ASR. Concrete samples were examined under digital microscope, optical microscope and scanning electron microscope. Concrete samples from different stages of ASR were used to reveal the physical and chemical states of the reaction products and concrete microstructure. The following are the main findings:

**Pore pressure at the early stage of the reaction**: When concrete is subjected to accelerated curing conditions, ASR becomes vigorous and produces fluid reaction products. The fluid products impose pore pressure to the concrete microstructure. Such pressure was observed to cause exudation of the fluid products from exposed cracks. The fluid products inside the micro-cracks and concrete pores can contribute to resisting externally applied compressive loads, thus, generally masking any loss in compressive strength due to ASR. However, the fluid products cannot provide any strength to positively contribute to the tensile strength and modulus of elasticity of concrete. Rather the pore pressure is detrimental to the tensile strength and modulus of elasticity of concrete.
Figure 7.5: Microscopic images and elemental mapping of a crack spanning paste and aggregate
Physical and chemical transformation of the reaction products at the later stage of the reaction: With time, the reaction subsides and the fluid products gradually solidify. Debonding cracks and cracks in the paste matrix that were caused by ASR are either filled with calcium hydroxide or are filled with ASR products transformed by calcium hydroxide into dense and stable lime–alkali-silicate complex. While the ASR products inside the aggregate are not associated with calcium, the ASR products, which were once fluid, solidify and start bridging the cracks. The effect of solidification of fluid products, the effect of crack bridging and the effect of transformation of ASR products into lime-alkali-silicate complex all contribute to improve the microstructure of concrete and to cause partial recovery in the damaged mechanical properties.

7.6 References


Chapter 8
Conclusions and Recommendations

8.1 Summary

This PhD thesis is an outcome of the research work investigating the effect of multiaxial stresses on the expansion, cracking and degradation of mechanical properties of concrete. Performance of ASR-affected concrete was evaluated by measuring expansion, analyzing cracks along the three mutually perpendicular planes and on the concrete surfaces, testing of the mechanical properties of concrete by destructive and non-destructive methods, and performing microscopic analysis of concrete samples. Moreover, the effect of an elevated accelerating temperature from 38 to 50 °C and the effect of coarse aggregate grading were examined on the ASR performance of concrete. The key research outcomes were compiled into six chapters (Chapter 2 to 7). The key findings and contributions from this thesis are summarized hereunder:

This study developed a new method of applying multiaxial (uniaxial, biaxial or triaxial) compressive stresses in concrete and sustaining the stresses for the duration of ASR. The specimens were measured for triaxial expansion. Axial and volumetric expansion of multiaxially loaded ASR-affected concrete were elucidated. By performing damage rating index (DRI) analysis of concrete samples, this study portrayed the ASR cracking along three mutually perpendicular planes of multiaxially loaded concrete. A relationship was formulated between the ASR expansion and DRI for multiaxially loaded ASR-affected concrete. Moreover, by testing cores taken along different directions, this study determined the influence of stress on the degradation of mechanical properties. ASR-affected concrete was shown to behave orthotropically. This study developed an expansion-stress relationship for ASR-affected concrete. The proposed relationship was validated by performing a finite element analysis to predict the measured expansion values on concrete specimens. The predicted expansions are in reasonable agreement with the measured expansions.
8.2 Concluding Remarks

Chapter 2 reported the development of a new method for applying and sustaining multiaxial compressive stresses in concrete specimens undergoing ASR until the reaction is exhausted. The conclusions are:

- Expansive reactions, such as ASR, have been recognized as an urgent problem for many important concrete structures, such as dams, bridges and nuclear power plants. Often, concrete in such structures is subjected to stresses in one or more directions.
- With the advancement of computational technology, it is now possible to make a thorough analysis of a structure numerically. However, the crucial input for such an analysis is missing, owing to the lack of proper experimental studies that elucidate the ASR expansion, cracking and degradation of mechanical properties of concrete subjected to multiaxial stress state.
- To investigate ASR evolution in a multiaxially loaded concrete specimen, the greatest challenge is perhaps the unavailability of a proper experimental setup. Even though triaxial tests (including polyaxial system in geotechnical engineering) have been practiced for years, no suitable method is available for applying and maintaining triaxial stress in concrete specimens for an extended duration sufficient for the reaction to evolve and exhaust. In this study, a new experimental method was developed for applying and sustaining multiaxial compressive stresses in concrete specimens undergoing ASR until the reaction is exhausted.
- The proposed method utilized the post-tensioning method of prestressing to apply stresses in concrete. Reasonably constant stress state was maintained by stressing the bolts close to yielding. Expansion due to ASR eventually yielded the bolts. Once yielded, the stress level in the bolt would remain reasonably constant at the yield plateau for large strains in excess of 5000 microstrain (0.5% strain). The reasonably constant stress was sustained for a long duration (> 1 year) in the concrete specimens that were conditioned in a hot (50±0.5 °C) and humid (> 95% relative humidity) chamber. Multiple cube specimens were loaded for uniaxial, biaxial or triaxial stress states. The specimens were conditioned under the accelerated conditions since the age of 6 months and were periodically measured for the triaxial expansions. The ultimate expansion was achieved between 8 to 12 months of accelerated curing.
• This chapter presents the expansion results which not only demonstrate that ASR expansion is strongly influenced by the stress state but also provide a basis to formulate an expansion-stress relationship for the three dimensional domain.

• The proposed method is customizable, scalable and practical to simulate the practical stress state, and will be useful to perform experimental studies that can advance the understanding about the evolution and consequences of ASR on concrete structures. Moreover, the setup could inspire similar studies to investigate other expansive problems in concrete, such as the sulfate attack.

Chapter 3 developed a relationship between the ASR expansion and the stress state of ASR-affected concrete. The conclusion from the chapter is:

• A relationship between ASR expansion and stress state was proposed based on an experimental study that measured the ASR expansion in the three mutually perpendicular directions in a number of unrestrained and restrained concrete cube specimens subjected to uniaxial, biaxial or triaxial stresses. The experimental results showed that the reaction kinetics is not coupled with the expansion-stress relationship. Therefore, the proposed empirical relationship is not coupled with the reaction kinetics. The relationship is simple to implement in a numerical analysis program. It was implemented in a finite element analysis software, VecTor3. The relationship was validated with the experimental results in cube specimens with uniaxial, biaxial and triaxial stress states. Rooted on an experimental investigation involving the measurement of ASR expansion for the case of multiaxial stress states, this model is expected to be an accurate basis for reliably estimating the ASR expansion of concrete structures in multiple directions.

The chapter discussed several important considerations regarding the proposed expansion-stress relationship as:

• The proposed empirical model was validated against the experimental measurements for the cube specimens having multiaxial stress states. A small discrepancy might have occurred due to the simplifications made in the numerical analysis. However, the experimental results were simulated with reasonable accuracy.

• Based on the proposed model, a hydrostatic stress state lower than 1.5 MPa cannot reduce volumetric ASR expansion.
Based on the experimental observation, the present model decouples the effect of stress on reaction kinetics. This consideration greatly simplifies the simulation of ASR-affected concrete structures.

The proposed model decouples ASR from creep simply based on experiments involving non-reactive concrete. More complicated experimental investigation to decouple the ASR and creep strains in reactive concrete can be a topic of future research interest.

The proposed model indicates that a minimum axial expansion of 11.36% of the free volumetric expansion will occur despite a large compressive stress. This appears unreasonable for a relatively larger compressive stress. However, for the typical stress level in concrete structures, which is mostly below 10 MPa, this level of expansion may not be unreasonable and is conservative. A small amount of ASR expansion in concrete structures might be compensated by elastic shortening, creep and shrinkage strain due to compressive stress.

Any attempts to interpret ASR expansion based on the concept of the first invariant of the stress tensor is not valid.

The arrangement for stress application in the experiment caused non-uniform stress distribution in the cube specimens. However, the stress distribution was obtained by a numerical analysis, and the stress condition helped in formulating and validating the expansion-stress relationship.

Chapter 4 investigated the expansion, cracking, and degradation of the mechanical properties of ASR-affected concrete subjected to seven different compressive stress states, namely no-stress and two variations each of uniaxial, biaxial and triaxial stresses. The following are the key new contributions:

- ASR axial expansion was highly sensitive to the stress state of concrete and behaved as a directional property. ASR axial expansion was reduced due to stress and transferred to a no-stress or relatively lower stress direction. ASR attempts to conserve the volumetric expansion in concrete as long as one unstressed direction is available.

- The expansion transfer mechanism was confirmed by the DRI results. The comparison of DRI values along different planes demonstrated that even the biaxial confinement was not adequate to suppress ASR damage. However, triaxial specimens showed markedly
smaller DRI values compared to the other stress conditions suggesting that the triaxial confinement can reduce ASR damage. Even though the triaxial confinement did not prevent the reaction, the triaxial confinement was effective in largely reducing cracking, particularly the mortar matrix cracking.

- The stress state influenced the opening of cracks. Cracks were opened preferentially along the unstressed direction or the direction with a relatively smaller compressive stress.

- Similar to the ASR axial expansion, the cracking of concrete and the change in mechanical properties showed a directional behavior with the stress state. The loss in static modulus of elasticity was the greatest for the no-stress specimens compared to the stressed specimens. The modulus of elasticity was observed to be anisotropic as a direct result of the directional stress state.

- Triaxial compressive stress in ASR concrete contributed to having reduced volumetric expansion, lower DRI value, less cracking, and less overall degradation of the mechanical properties.

- This study integrates the understanding of the three main effects of ASR, namely expansion, cracking and degradation of mechanical properties of concrete in the context of concrete structures. The findings from this study will be useful in the analysis of reinforced and multiaxially stressed concrete structures, in particular, suggesting the necessity of treating ASR-affected concrete as an orthotropic material. This is anticipated to increase the accuracy of numerical analysis. Similar studies with various types of reactive aggregates and various mix designs will improve the understanding of the consequences of ASR in the concrete structures. The findings from this study can further help in performing field investigation programs, in particular, to ascertain the direction of expansion measurement and coring, and to interpret the orientation of cracks.

While Chapter 2 to 4 formed the central concept of this thesis in developing a new method and in presenting the results and analysis of the experiment based on the new method, this thesis further investigated the effect of some other factors on the ASR performance of concrete. Chapter 5 investigated the effect of elevated temperature on the expansion, damage rating index and mechanical properties of ASR-affected concrete. The main conclusions from this chapter are:
• Two sets of tests were performed, namely (1) concrete prism test (CPT) performed at 38±2 °C and (2) accelerated concrete prism test (ACPT) performed at 50±0.5 °C.

• With an increase of 12 °C temperature from 38±2 to 50±0.5 °C, the expansion in ACPT was 3.22 times faster than in CPT. Except for the rate of expansion, the general trend of expansion and the ultimate expansion were similar for both the ACPT and CPT specimens.

• Accounting for the 3.22 times faster rate of reaction in ACPT than in CPT, the trends of reduction and partial recovery in modulus of rupture and in dynamic modulus of elasticity were identical in both CPT and ACPT. However, the reduction was slightly greater in ACPT than in CPT.

• DRI results elucidated the evolution of damage due to ASR in the reactive concrete. The ability of the DRI method to account for tight cracks captured the onset of damage due to ASR. Despite some limitations, DRI is a comprehensive method for investigating the damage due to ASR. However, rather than the absolute DRI value, the individual petrographic features of the DRI method are equal or more instrumental to understand the evolution of damage due to ASR. The faster rate of reaction in ACPT induced greater number of cracks at the earlier stage of reaction. This resulted in larger DRI value in ACPT than in CPT for identical expansion.

• Identical expansions and similar trends of modulus of rupture and dynamic modulus of elasticity suggest that an increase in conditioning temperature from 38±2 °C to 50±0.5 °C speeds up the test with little effect on the ASR mechanism based on specimens in this study. Nevertheless, a slight increase in the extent of damage and microstructural cracking with a 12 °C rise in temperature indicates that while accelerated tests are necessary for ASR studies, an increased temperature is likely to involve an increased deviation in the response of concrete from that in the field concrete structures, which was, however, not investigated in this study.

Chapter 6 reported the effect of coarse aggregate grading on the ASR expansion and damage of concrete. The following are the main conclusions:
This study investigated the effects of variation in the grading of reactive coarse aggregate, Spratt, on the ASR expansion and damage of concrete. Three mixes of reactive concrete were studied by considering three coarse aggregate gradings.

A deviation in specified grading of coarse aggregate by 10% towards the finer size resulted in 50% deviation in the expansion measurement compared to the standard mix. This was attributed partly to the weaker mechanical properties and relatively less aggregate interlocking effect in the fine-dominant mix compared to the standard mix. DRI results demonstrated that any deviation in aggregate grading from the standard grading significantly increases the damage due to ASR.

Over a 1-year duration, modulus of rupture of the reactive concrete prisms was reduced due to ASR by 53–60%, irrespective of the type of coarse aggregate grading used. The fastest reduction occurred for the fine-dominant mix. The coarse-dominant mix showed a slightly smaller reduction in the modulus of rupture indicating increased interlocking provided by the coarse aggregates.

The static modulus of elasticity was affected by ASR since its initiation. The maximum reduction in the modulus of elasticity of the fine-dominant and the coarse-dominant mixes was approximately 35%.

The compressive strength for the coarse-dominant mix was the largest among the three mixes indicating that the larger aggregates helped in enhancing the strength of concrete. Compared to the UPV of the coarse-dominant mix, the UPV of the fine-dominant mix was always smaller and showed larger reduction due to ASR. Bulk resistivity of concrete indicated that the fine-dominant grading resulted in a mix that was more permeable than the coarse-dominant mix, and this appeared the case irrespective of ASR.

The findings of this study highlight the importance of grading in performing CPT or other studies investigating the ASR performance of concrete mixtures, and also in minimizing the ASR expansion of concrete structures. The quality control in maintaining a specified grading of coarse aggregate should be stringent in a project associated with assessing the ASR performance of concrete.

This study indicates that an aggregate that is concluded as innocuous based on the concrete prism test, such as ASTM C1293, may still show marked expansion and damage in concrete members in which the aggregate grading is widely different from the ASTM
C1293 aggregate grading. Specifications related to concrete aggregates for field concrete should consider the influence of aggregate grading on the ASR performance of concrete. Similar studies with a variety of reactive aggregates is expected to increase the understanding of the effect of coarse aggregate grading on the ASR performance of concrete.

Chapter 7 focused on the microscopic analysis of concrete samples in an attempt to investigate the mechanisms behind the partial recovery in the degraded mechanical properties of ASR-affected concrete that has been observed in many laboratory studies to characteristically follow the loss of some mechanical properties due to ASR. Concrete samples were examined under digital microscope, optical microscope and scanning electron microscope. Concrete samples from different stages of ASR were used to reveal the physical and chemical states of the reaction products and concrete microstructure. The following are the main findings:

- **Pore pressure at the early stage of the reaction**: When concrete is subjected to accelerated curing conditions, ASR becomes vigorous and produces fluid reaction products. The fluid products impose pore pressure to the concrete microstructure. Such pressure was observed to cause exudation of the fluid products from exposed cracks. The fluid products inside the micro-cracks and concrete pores can contribute to resisting externally applied compressive loads, thus, generally masking any loss in compressive strength due to ASR. However, the fluid products cannot provide any strength to positively contribute to the tensile strength and modulus of elasticity of concrete. Rather the pore pressure is detrimental to the tensile strength and modulus of elasticity of concrete.

- **Physical and chemical transformation of the reaction products at the later stage of the reaction**: With time, the reaction subsides and the fluid products gradually solidify. Debonding cracks and cracks in the paste matrix that were caused by ASR are either filled with calcium hydroxide or are filled with ASR products transformed by calcium hydroxide into dense and stable lime–alkali-silicate complex. While the ASR products inside the aggregate are not associated with calcium, the ASR products, which were once fluid, solidify and start bridging the cracks. The effect of solidification of fluid products, the effect of crack bridging and the effect of transformation of ASR products into lime-alkali-silicate complex all contribute to improve the microstructure of concrete and to cause partial recovery in the damaged mechanical properties.
8.3 Suggestions for Future Work

This study developed a new method of applying long-term multiaxial stresses in concrete specimens undergoing ASR. The stresses involved were compressive. The method of stress application involves a non-uniform stress distribution on the concrete specimens which was obtained through a numerical analysis in this study. Specific to the stress application method, the following are the suggestions for future work:

- Use of a mechanical torque wrench may be beneficial for tightening the bolts.
- Galvanization of the fasteners was necessary considering the hot and humid environment that encourages corrosion. However, galvanization increased the friction of the bolts. Non-corrosive grades of steel could be explored to avoid the galvanization process.
- Protection of strain gauges and their leadwires could be improved. Post-yield type strain gauges could be used. Additional protective measures could be applied for longevity of the strain gauges subjected to a harsh environment. Also, the strain gauges could be monitored continuously, if needed.
- In this study, the grooves were cut in the concrete specimens for passing the leadwires underneath the bearing plates. In the future, an extrusion could be designed in the formwork so that the groove is formed in the concrete surface.
- Methods should be explored to improve the stress distribution in concrete. Numerical analysis should be performed in order to analyze the stress distribution. One possible way of reducing the effect of stress concentration on expansion measurements could be to embed expansion studs at a relatively larger depth and to minimize bond with concrete along the stud surface.
- Investigation of a narrow range of 0.5 to 2 MPa compressive stress is recommended for better understanding the mechanism of reduction and transfer of ASR expansion.
- A new setup involving tensile stresses could be regarded as a future development in the field.

This study indicated the effect of specimen size and geometry on the ASR expansion of concrete and adopted cube specimens in order to rule out the effect of specimen geometry and aspect ratio. However, considering the complicated nature of ASR, the effect of size on ASR expansion
deserves further investigation. Future studies should investigate the effect of specimen size and geometry on the ASR expansion and damage of concrete.

This study focused on Spratt aggregate as the reactive aggregate. A similar study in the future for a broad range of reactive aggregates will elaborate the understanding of the effect of stress state on the ASR performance of concrete.

Apart from expansion measurement, measurement of cracking through the DRI method and the testing of mechanical properties were effective to evaluate the ASR performance of concrete. This study recommends the widespread use of these techniques in future studies. Moreover, future studies should consider utilizing the stiffness damage test in evaluating the degradation of mechanical properties of multiaxially loaded ASR-affected concrete.

This study indicated that the study of ASR damage of concrete from the early stage of reaction can reveal important mechanisms related to ASR evolution. Microscopic examination of ASR products should aim to focus on the evolution of ASR from an early stage of the reaction.

This study revealed that the effect of stress state transforms the ASR-affected concrete into an orthotropic material. Future studies, both experimental and numerical, should focus on this aspect.
Appendices
Appendix A:
Accelerated mortar bar test (AMBT) of Orillia sand

Natural sand from Orillia, Ontario, Canada (referred to as Orillia sand in this study) was used for the concrete mixes used in this study. The sand was required to be non-reactive to alkali-silica reaction. The sand was tested for confirming the non-reactivity using the accelerated mortar bar test based on ASTM C1260 [1]. The test results are shown in Figure A.1. As shown in the figure, the expansion was 0.096% at 14 days, which is lower than 0.1% thus confirming the sand as non-reactive.

Figure A.1: Expansion of Orillia sand in the accelerated mortar bar test as per ASTM C1260 [1]
Appendix B:  
Construction of the acceleration chamber

This study required a chamber for accelerating the ASR by storing concrete specimens in 50±0.5 °C and >95% relative humidity condition. In coordination with research colleagues working for the research project funded through a contract by the Canadian Nuclear Safety Commission [2], an acceleration chamber was constructed inside a basement room of the Department of Civil Engineering, University of Toronto. The chamber is 5.6 m by 3.4 m in plan area and has a clear height of 2.2 m. As shown in Figure B.1, the cross section of the wall was designed to ensure reasonable heat insulation and suitability for high humidity application. The wall and roof sections were made up of Exceliner brand FRP boards at the interior, followed by “mold tough” dry walls, polyethylene vapour barrier, Roxul® batt insulation (R-value = 14) and plywood at the outer face. The frame for the room was made up of pressure treated lumber. Some photographs during the construction of the chamber are shown in Figures B.2 to B.4.

Figure B.1: Cross-section of the wall of the acceleration chamber
Figure B.2: Wooden frame of the acceleration chamber

Figure B.3: Roxul® batt insulation being covered by a vapour barrier (left), and being overlaid with drywall (right)

Figure B.4: FRP panel being attached on top of drywall
Door and window

A 2.3 m wide and 2 m high door was constructed to access the chamber. The heaters and humidifier in the chamber were turned off and the doors were left open for few hours before accessing the chamber. The cross-section of a door shutter was made similar to the section of wall as shown in Figure B.1 except for the drywall was substituted by pressure-treated plywood. Joints and seams around the door shutters were sealed with suitable sealants. A triple pane glazed window was installed in a 0.5 m by 0.5 m slot left in a door shutter.

Heating and humidifying systems

Three radiant type heaters were installed in three walls of the chamber. A fogging fan was mounted inside the chamber to create high humidity condition. Figure B.5 shows the fogging fan and two of the three heaters in the chamber. The figure also shows the flowmeter that was used to control the supply of water to the fogging fan. Relative humidity inside the chamber was continuously monitored. For this purpose, a portable humidity sensor was used for which the display unit was kept outside the chamber but the sensor connected by a cord was inserted inside the chamber through a small hole pierced across the wall of the chamber.

The heaters were equipped with a control panel to control the heaters, and to display and log the temperature. A photograph of the control panel is shown in Figure B.6. Three heaters had three independent control systems. Six temperature sensors were spatially distributed in the chamber for monitoring and controlling the temperature. Because of this arrangement and also because of the constantly circulating air by the fogging fan used to maintain the relative humidity, a fairly constant temperature was maintained in the chamber. The temperature data logged at a location of the chamber showed that the temperature during constant operation stayed within ±0.5 °C of the target temperature of 50 °C. Because of various logistic challenges during the construction of the chamber, the cube specimens were subjected to the accelerated curing environment only at the age of 180 days.
Figure B.5: Fogging fan and heaters in the acceleration chamber (left) and the flowmeter (right)

Figure B.6: Control panel for the acceleration chamber
Appendix C:
Strain in the high-strength bolts in the cube specimens

Each high-strength bolt in the loaded cube specimens consisted of one strain gauge. The strain gauges were monitored for the strain in the bolts during three stages, namely, during loading at the age of approximately 2 months, before relocating the specimens to the acceleration chamber at the age of 6 months, and during unloading after 3, 8 or 12 months of accelerated curing. Table C.1 presents the strain data in the strain gauges. The data indicate that the bolts yielded after having ASR.

Table C.1: Strain in the high-strength bolts in the cube specimens

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Notes:
• Stage 1 represents the stage after finishing the loading at an age of approximately 2 months; Stage 2 represents the stage before relocating the specimens to the acceleration chamber at the age of 6 months; and Stage 3 represents the stage just before unloading the specimens after 3, 8 or 12 months of accelerated curing.
• Cells with a dash indicate missing data.
• The data logger was calibrated to read a maximum value of 20,000 microstrain. While a value of 20,000 was recorded for many strain gauges, the value, rather than being an actual strain value, might also have occurred due to the rupture of strain gauges.
• *Wedge failure occurred during loading. Thus, the stress level (and strain) was adjusted to the actual concrete area remaining.
Table C.1 (continued): Strain in the high-strength bolts in the cube specimens

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Appendix D: Preliminary analysis of the cube specimens by using ANSYS

Selected cube specimens in this study were analyzed by using the finite element analysis software ANSYS. Considering the symmetry of the specimen, only one-eighth of the specimen was modeled as shown in Figure D.1a. The bearing plates and sheaths were modeled as shown in Figure D.1b. A contact analysis was performed in order to simulate the no-bond condition between concrete and the sheath. A snapshot of the model with stress analysis results is illustrated in Figure D.1c.

ANSYS has a provision for modeling material swelling. A swelling effect can be implemented by writing a swelling rule in the subroutine "usersw". However, the swelling is assumed as isotropic, and only one variable can be output by the subroutine. Such an approach has been used to simulate ASR expansion in the literature [3]. However, it appears not possible to use ANSYS for simulating the ASR expansion of concrete in which ASR expansion needs to be defined differently on three directions. Therefore, further analysis using ANSYS was not conducted; rather a finite element analysis program, “VecTor3”, was used for validating the expansion-stress relationship proposed in this study.
a) 3-D model of the one-eighth of a specimen
b) Bearing plates and sheaths along three directions

c) Distribution of the normal stress along the Z-direction

Figure D.1: Stress analysis of a triaxially stressed (3.9, 3.9, 3.9) specimen using ANSYS
Appendix E:
Crack visualization using ultraviolet fluorescent imaging

In an attempt to visualize the ASR cracks, selected surfaces of concrete prepared for DRI analysis were vacuum impregnated with epoxy containing a fluorescent dye after performing DRI analysis. The surface after epoxy impregnation was polished again just to remove the epoxy veneer. A number of images were taken in a defined sequence with 40% overlap under a stereobinocular microscope at 10X magnification using ultraviolet (UV) light as the source of illumination. The images were stitched together by using a plugin [4] available in “Fiji” software. After stitching, the green channel of the image was extracted and converted into greyscale image. Brightness and contrast were adjusted. Figures E.1 and E.2 show the UV images of the polished surfaces of concrete in the YZ plane from the no-stress n (0, 0, 0) specimens, respectively, at 3 and 12 months after accelerated curing. The images illustrate the cracks in the aggregate and in the paste matrix too. However, as the polished surfaces had contained a number of filled cracks as evidenced in the damage rating index analysis, epoxy was not able to penetrate all the cracks. Considering this drawback and the difficulty associated with the large size of samples (up to 254 mm long), the impregnation was not performed for other samples.

Figure E.1: UV image showing cracks in the polished surface in the YZ plane from the no-stress n (0, 0, 0) specimen after 3 months of accelerated curing (the length of the sample is 254 mm)
Figure E.2: UV image showing cracks in the polished surface in the YZ plane from the no-stress n (0, 0, 0) specimen after 12 months of accelerated curing (the length of the sample is 130 mm)
Appendix F:
Mechanical properties of cores of reactive cube specimens

Cores that were taken from the reactive cube specimens were tested for the compressive strength, static modulus of elasticity and Poisson’s ratio. While the compressive strength and static modulus of elasticity results were presented graphically in Chapter 4, the results along with the Poisson’s ratio results are presented in Table F.1. Some results from this test program were also reported in Gautam et al. [5].

The lateral strain of the core specimens for calculating the Poisson’s ratio was measured by using strain gauges. Figure F.1 shows three core specimens being affixed with strain gauges before being tested for mechanical properties. Two strain gauges of length 60 mm each (PL-60 type strain gauge from Tokyo Sokki Kenkyujo Co. Ltd., Japan) were affixed at diametrically opposite surfaces at approximately the mid-height of the core specimens. Since an extensometer was also used to measure the lateral strain in some cores, the strain gauges were offset by approximately 6 mm on either side from the mid-height such that the extensometer could be aligned at the mid-height.

Figure F.1: Core specimens (dia. 75 mm and height 150 mm) being affixed with strain gauges before being tested for mechanical properties
Table F.1: Mechanical properties of the diameter 75 mm cores from the reactive cube specimens and the 28-day cylinder compressive strength of concrete that was used to cast the reactive specimens

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<th>Compressive strength of core (MPa)</th>
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<td>41.9 ± 1.0</td>
<td>60.3</td>
<td>38.9</td>
<td>0.24</td>
</tr>
<tr>
<td></td>
<td>B1</td>
<td>Z</td>
<td>48.9 ± 2.1</td>
<td>56.6</td>
<td>34.2</td>
<td>0.23</td>
</tr>
<tr>
<td></td>
<td>B2</td>
<td>X</td>
<td>48.9 ± 2.1</td>
<td>56.2</td>
<td>42.8</td>
<td>0.26</td>
</tr>
<tr>
<td></td>
<td>t3</td>
<td>X</td>
<td>43.9 ± 1.6</td>
<td>53.7</td>
<td>38.8</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td>T3</td>
<td>X</td>
<td>39.0 ± 0.6</td>
<td>48.8</td>
<td>34.6</td>
<td>0.24</td>
</tr>
<tr>
<td>12 months of accelerated curing</td>
<td>n3</td>
<td>X</td>
<td>44.2 ± 1.6</td>
<td>54.5</td>
<td>35.9</td>
<td>0.23</td>
</tr>
<tr>
<td></td>
<td>u1</td>
<td>X</td>
<td>45.6 ± 2.2</td>
<td>52.9</td>
<td>38.6</td>
<td>0.22</td>
</tr>
<tr>
<td></td>
<td>U1</td>
<td>X</td>
<td>42.2 ± 1.0</td>
<td>59.9</td>
<td>36.7</td>
<td>0.24</td>
</tr>
<tr>
<td></td>
<td>b3</td>
<td>X</td>
<td>41.9 ± 1.0</td>
<td>60.4</td>
<td>41.4</td>
<td>0.22</td>
</tr>
<tr>
<td></td>
<td>B3</td>
<td>X</td>
<td>41.9 ± 1.0</td>
<td>63</td>
<td>41.1</td>
<td>0.22</td>
</tr>
<tr>
<td></td>
<td>t2</td>
<td>X</td>
<td>43.9 ± 1.6</td>
<td>56.6</td>
<td>42.2</td>
<td>0.23</td>
</tr>
<tr>
<td></td>
<td>T2</td>
<td>X</td>
<td>39.0 ± 0.6</td>
<td>53.9</td>
<td>39.2</td>
<td>0.19</td>
</tr>
</tbody>
</table>

Notes:
- Specimens were subjected to the accelerated curing at the age of 6 months.
- The designation for the specimens, such as “n1”, is such that the letter represents one of the seven stress states as n (0, 0, 0), u (3.9, 0, 0), U (9.6, 0, 0), b (3.9, 3.9, 0), B (9.6, 3.9, 0), t (3.9, 3.9, 3.9) or T (9.6, 3.9, 3.9), and the number represents the specific number among multiple specimens of a given stress state.
- Each value for the core represents the test result obtained from testing a single core.
- The 28-day compressive strength of the cylinder is presented as the mean ± one standard deviation based on the testing of 3 cylinders.
Appendix G: Ultrasonic pulse velocity (UPV)

Ultrasonic pulse velocity (UPV) method was used as a non-destructive test method to monitor the performance of the reactive and non-reactive concrete used in this study. Cylinder and cube specimens were tested by UPV method at different ages. The UPV results along the three axes are presented in Table G.1 for the non-reactive and in Table G.2 for the reactive cube specimens. The results for the reactive cube specimens are the average of three specimens until 3 months of accelerated curing, the average of 2 specimens from 4.5 to 8 months of accelerated curing, and for a single specimen at 12 months of accelerated curing.

UPV was not sensitive to the ASR damage based on this study. The maximum reduction in UPV with respect to the reference measurement at 2 months of casting was 5% for the reactive cube specimens. Similarly, the maximum increase in UPV with respect to the reference measurement was 5% for the non-reactive specimens. As also reported previously [6], the UPV method appears not a suitable method for estimating the ASR degradation of laboratory specimens.

However, attempts were made to correlate the UPV with the mechanical properties of concrete obtained from destructive tests. For this purpose, UPV was measured in the cylinder specimens that were tested for the static modulus of elasticity and compressive strength as reported in Chapter 4. At least three cylinders were measured at each test age. For compressive strength, the correlation of UPV was made with the fourth root of the compressive strength as suggested by Panesar and Chidiac [7]. The correlation curves are shown in Figure G.1 and Figure G.2. A good correlation with an $R^2$ of 0.86 is observed for the static modulus of elasticity while not a good correlation is observed for the compressive strength.
Table G.1: Ultrasonic pulse velocity (UPV) of non-reactive (NR) cube specimens along three axes (m/s)

<table>
<thead>
<tr>
<th>Stress state</th>
<th>Axis</th>
<th>Reference (2 month of casting)</th>
<th>Age after beginning accelerated curing at 50 °C and &gt; 95% RH after 6 months of casting (month)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>3</td>
</tr>
<tr>
<td>NR-n (0, 0, 0)</td>
<td>X</td>
<td>5090</td>
<td>5142</td>
</tr>
<tr>
<td></td>
<td>Y</td>
<td>5142</td>
<td>5194</td>
</tr>
<tr>
<td></td>
<td>Z</td>
<td>4962</td>
<td>5058</td>
</tr>
<tr>
<td>NR-u (3.9, 0, 0)</td>
<td>X</td>
<td>5194</td>
<td>5248</td>
</tr>
<tr>
<td></td>
<td>Y</td>
<td>5194</td>
<td>5303</td>
</tr>
<tr>
<td></td>
<td>Z</td>
<td>5108</td>
<td>5263</td>
</tr>
<tr>
<td>NR-U (9.6, 0, 0)</td>
<td>X</td>
<td>5090</td>
<td>5142</td>
</tr>
<tr>
<td></td>
<td>Y</td>
<td>5248</td>
<td>5194</td>
</tr>
<tr>
<td></td>
<td>Z</td>
<td>5078</td>
<td>5108</td>
</tr>
<tr>
<td>NR-b (3.9, 3.9, 0)</td>
<td>X</td>
<td>5142</td>
<td>5194</td>
</tr>
<tr>
<td></td>
<td>Y</td>
<td>5194</td>
<td>5194</td>
</tr>
<tr>
<td></td>
<td>Z</td>
<td>5159</td>
<td>5210</td>
</tr>
<tr>
<td>NR-B (9.6, 3.9, 0)</td>
<td>X</td>
<td>5194</td>
<td>5194</td>
</tr>
<tr>
<td></td>
<td>Y</td>
<td>5142</td>
<td>5194</td>
</tr>
<tr>
<td></td>
<td>Z</td>
<td>5263</td>
<td>5263</td>
</tr>
<tr>
<td>NR-t (3.9, 3.9, 3.9)</td>
<td>X</td>
<td>5194</td>
<td>5359</td>
</tr>
<tr>
<td></td>
<td>Y</td>
<td>5292</td>
<td>5359</td>
</tr>
<tr>
<td></td>
<td>Z</td>
<td>5111</td>
<td>5248</td>
</tr>
<tr>
<td>NR-T (9.6, 3.9, 3.9)</td>
<td>X</td>
<td>5090</td>
<td>5248</td>
</tr>
<tr>
<td></td>
<td>Y</td>
<td>5142</td>
<td>5194</td>
</tr>
<tr>
<td></td>
<td>Z</td>
<td>4990</td>
<td>5142</td>
</tr>
<tr>
<td>Stress state</td>
<td>Axis</td>
<td>Reference (2 month of casting)</td>
<td>Age after beginning accelerated curing at 50 °C and &gt; 95% RH after 6 months of casting (month)</td>
</tr>
<tr>
<td>--------------</td>
<td>------</td>
<td>--------------------------------</td>
<td>-------------------------------------------------</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>3</td>
</tr>
<tr>
<td>R-n (0, 0, 0)</td>
<td>X</td>
<td>4962</td>
<td>4787</td>
</tr>
<tr>
<td></td>
<td>Y</td>
<td>4942</td>
<td>4757</td>
</tr>
<tr>
<td></td>
<td>Z</td>
<td>4962</td>
<td>4839</td>
</tr>
<tr>
<td>R-u (3.9, 0, 0)</td>
<td>X</td>
<td>4926</td>
<td>4787</td>
</tr>
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<td></td>
<td>Y</td>
<td>4917</td>
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</tr>
<tr>
<td></td>
<td>Z</td>
<td>4968</td>
<td>4869</td>
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<tr>
<td>R-U (9.6, 0, 0)</td>
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<td>4887</td>
</tr>
<tr>
<td></td>
<td>Y</td>
<td>4990</td>
<td>4855</td>
</tr>
<tr>
<td></td>
<td>Z</td>
<td>5026</td>
<td>4931</td>
</tr>
<tr>
<td>R-b (3.9, 3.9, 0)</td>
<td>X</td>
<td>4963</td>
<td>4863</td>
</tr>
<tr>
<td></td>
<td>Y</td>
<td>4990</td>
<td>4910</td>
</tr>
<tr>
<td></td>
<td>Z</td>
<td>5010</td>
<td>4916</td>
</tr>
<tr>
<td>R-B (9.6, 3.9, 0)</td>
<td>X</td>
<td>5007</td>
<td>4958</td>
</tr>
<tr>
<td></td>
<td>Y</td>
<td>5007</td>
<td>4990</td>
</tr>
<tr>
<td></td>
<td>Z</td>
<td>4978</td>
<td>4847</td>
</tr>
<tr>
<td>R-t (3.9, 3.9, 3.9)</td>
<td>X</td>
<td>4943</td>
<td>4879</td>
</tr>
<tr>
<td></td>
<td>Y</td>
<td>4958</td>
<td>4926</td>
</tr>
<tr>
<td></td>
<td>Z</td>
<td>4894</td>
<td>4927</td>
</tr>
<tr>
<td>R-T (9.6, 3.9, 3.9)</td>
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<td>4942</td>
</tr>
<tr>
<td></td>
<td>Y</td>
<td>4942</td>
<td>4982</td>
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<tr>
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<td>Z</td>
<td>4841</td>
<td>4911</td>
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</table>
Figure G.1: Correlation between static modulus of elasticity and UPV of reactive concrete cylinders

Figure G.2: Correlation between compressive strength and UPV of reactive concrete cylinders
Appendix H:
Transverse expansion measurement of concrete prisms

The expansion behavior has been reported to be influenced by the direction of casting of concrete [8,9]. Thus, this study investigated the effect of casting direction on the variability of expansion along the longitudinal and transverse directions of reactive concrete prism specimens. Four prism specimens from the concrete prism test program reported in Chapter 5 were used for measuring the transverse expansion along the two directions of the cross-section of the prisms in addition to the longitudinal expansion measurement [10]. The expansions measured in the longitudinal direction was perpendicular to the casting direction of the prisms. The two transverse direction measurements comprised of one parallel to casting direction and the other perpendicular to the casting direction. The transverse expansion measurements perpendicular to the casting direction are measured on two concrete surfaces that have been cast against the formwork and so are also referred to as the ‘form-form’ transverse measurements. The transverse expansion measurements parallel to the casting direction are measured on one concrete surface that has been hand finished and the other surface cast against the formwork and so are also referred to as the ‘finish-form’ transverse measurements.

Measurement of expansion

The longitudinal expansion was measured as per ASTM C1293 [11] using a comparator against the steel studs which were embedded in the ends of the prisms with a gauge length of 250 mm. Transverse expansion was measured by a 75-100 mm range digital micrometer. Figure H.1a illustrates a digital micrometer and the orientation of a prism during transverse expansion measurement. The digital micrometer with a resolution of 1 µm was calibrated against a micrometer standard before taking the prism measurements. Transverse expansion measurements were taken against the small brass targets, which were epoxy-glued on the sides of the prisms as shown in Figure H.1a. Two targets were glued in each of the four faces of a prism; thus, each prism had two measurements in each transverse direction. Steel guides were prepared as shown in Figure H.1b to assist in accurately fixing the targets.
Results

The mean expansion measurements ± one standard deviation in the longitudinal, transverse (finish-form), and transverse (form-form), of the prisms are plotted in Figure H.2. Each data point for the longitudinal expansion is the mean of four measurements whereas each data point for the transverse expansions is the mean of eight measurements, including two measurements in each prism.

![Figure H.1: a) Micrometer; b) steel guides for affixing the brass targets](image)

![Figure H.2: Average longitudinal and transverse expansions of the concrete prisms](image)

Note: The error bar is ± one standard deviation.
Figure H.2 shows that the transverse expansion measurements were greater than the longitudinal expansion. As shown in Table H.1, the ratio of expansion in the transverse directions to the longitudinal direction was 3.72 at 28 days, and decreased to 1.31 by 365 days.

The transverse expansions in the two directions (parallel and perpendicular to the casting direction) were close to each other. However, as shown in Table H.1, the transverse expansion along the direction parallel to the casting direction was always slightly greater (even though mostly statistically insignificant) than the transverse expansion along the direction perpendicular to the casting direction.

Table H.1: Expansion ratios and coefficient of variation (COV) in the transverse expansions

<table>
<thead>
<tr>
<th>Age (days)</th>
<th>Ratio of expansions</th>
<th>COV (of eight measurements)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Average transverse to</td>
</tr>
<tr>
<td></td>
<td></td>
<td>longitudinal</td>
</tr>
<tr>
<td>28</td>
<td>3.72</td>
<td>1.33</td>
</tr>
<tr>
<td>56</td>
<td>2.09</td>
<td>1.09</td>
</tr>
<tr>
<td>91</td>
<td>1.66</td>
<td>1.04</td>
</tr>
<tr>
<td>182</td>
<td>1.45</td>
<td>1.04</td>
</tr>
<tr>
<td>273</td>
<td>1.23</td>
<td>1.05</td>
</tr>
<tr>
<td>365</td>
<td>1.31</td>
<td>1.03</td>
</tr>
</tbody>
</table>

In comparison to the COV for the longitudinal expansion measurements, which was less than 6% after 56 days, the COV corresponding to the transverse measurements was markedly larger (14 - 55 %) as shown in Table H.1. Smaoui et al. [8] also reported relatively larger COV for transverse expansions compared to the longitudinal expansion measurements. This may be partly attributed to the fact that transverse measurements correspond to a relatively smaller section of concrete (75 mm in this study) being measured compared to 285 mm for the longitudinal expansion.

The difference in the longitudinal and transverse expansions was attributed to the aspect ratio of the specimen. Based on the outcome of this study, cube specimens were considered for the study of multiaxially stressed ASR-affected concrete so that any differential effect of the aspect ratio of the specimens could be avoided.
Appendix I:
DRI along the longitudinal direction of concrete prisms

The damage rating index (DRI) analysis on the prism specimens of the concrete prism test reported in Chapter 5 was performed on the transverse (cross-sectional) slices from the prisms. In order to understand the DRI along the longitudinal direction of the prisms, three slices from two prisms were also analyzed for the DRI along the longitudinal direction. The prisms were conditioned at 38 °C and > 95% relative humidity for 2 years (730 days) before performing the DRI analysis. Figure I.1 shows a flatbed scan image of a polished section analyzed for DRI along the longitudinal direction of the prism. The average DRI results from the three slices are presented in Table I.1 in the form of contribution of the seven petrographic features of DRI. The average DRI value was 928 with the standard deviation of 20. Table I.1 also compares the DRI results with the results obtained from the analysis of four slices taken along the transverse direction from the prisms conditioned identically and as reported in Chapter 5. The average DRI for the transverse direction slices was 892 with the standard deviation of 102. The DRI results obtained from the transverse direction slices and the longitudinal direction slices are statistically similar. However, the transverse direction slices appear to have an increased level of paste cracking. On the other hand, the contributions of the “debonded coarse aggregate” and the “disaggregated/corroded aggregate particles” to the DRI value appear slightly larger for the longitudinal direction slices compared to the transverse direction slices.

Figure I.1: Polished section taken along the longitudinal direction of a reactive concrete prism conditioned at 38 °C and > 95% relative humidity for 730 days (size 285 mm by 75 mm)
Table I.1: Comparison of DRI values along the transverse and the longitudinal sections of concrete prisms conditioned at 38 °C and > 95% relative humidity for 730 days

<table>
<thead>
<tr>
<th>Petrographic features</th>
<th>Plane of the prism for DRI analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Transverse</td>
</tr>
<tr>
<td>Closed/ tight cracks in coarse aggregate particle</td>
<td>27</td>
</tr>
<tr>
<td>Opened cracks or network cracks in coarse aggregate particle</td>
<td>171</td>
</tr>
<tr>
<td>Cracks or network cracks with reaction product in coarse aggregate particle</td>
<td>379</td>
</tr>
<tr>
<td>Debonded coarse aggregate</td>
<td>54</td>
</tr>
<tr>
<td>Disaggregated / corroded aggregate particle</td>
<td>60</td>
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<tr>
<td>Cracks in cement paste</td>
<td>84</td>
</tr>
<tr>
<td>Cracks with reaction product in cement paste</td>
<td>117</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>892</strong></td>
</tr>
<tr>
<td>Standard deviation of slices (no. of slices)</td>
<td>102 (4)</td>
</tr>
</tbody>
</table>
References for the Appendices


