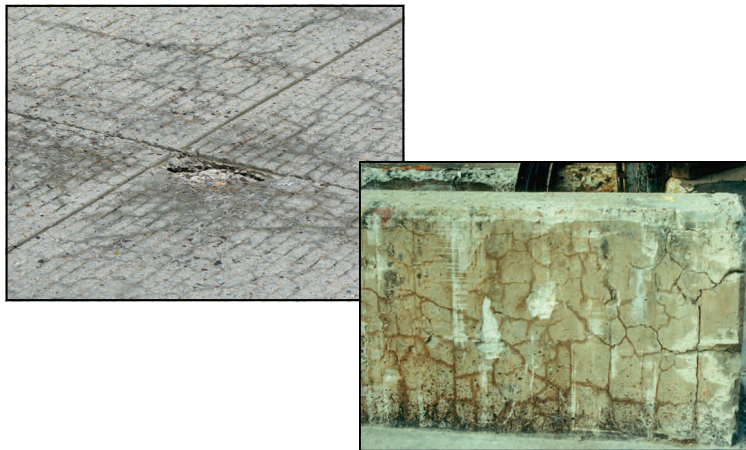U.S. Department of Transportation
Federal Highway Administration

ALKALI-AGGREGATE REACTIVITY (AAR) WORKSHOPS FOR ENGINEERS AND PRACTITIONERS

Reference Manual

January 2013

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1 – Introduction

1.1 BACKGROUND

Funding for the Federal Highway Administration (FHWA) Alkali-Silica Reactivity (ASR) Development and Deployment Program was provided under SAFETEA-LU. A related Conference Report provides additional guidance stating that "project and programs related to ASR should…assist states in inventorying existing structures for ASR." Throughout the program, the FHWA has been leading a national effort to further the development and deployment of techniques which can prevent and mitigate ASR.

In closing of the ASR Development and Deployment Program, workshops are being delivered to effectively transfer the deliverables of the Program and provide State Department of Transportation (DOT) engineers and practitioners with the information and tools necessary to address ASR in their states.

This document serves as the Reference Guide for the FHWA workshops on Alkali-Aggregate Reactivity (AAR).

1.2 WORKSHOP OBJECTIVES

Upon completion of the workshop, the participants should be able to:

- Understand the fundamentals of AAR, which includes alkali-silica reaction (ASR) and alkali-carbonate reaction (ACR).
- Describe available methods of preventing or mitigating ASR-induced damage in newly-constructed and existing concrete structures.
- Identify test methods and specifications related to ASR.

Case studies of ASR-affected concrete structures and rehabilitation strategies are provided throughout the workshop.

1.3 WORKSHOP OUTLINE

The workshop is divided into eight sessions or major topics. This Reference Guide also follows the same outline, and a brief description of each topic follows:

Introduction and Workshop Objectives

This first session provides an introduction of the speakers, workshop objectives and outline, and includes the opportunity for participants to present themselves.

Session 2 - Fundamentals of Alkali-Silica Reaction

This session provides a brief history of alkali-silica reaction (ASR) and alkali-carbonate reaction (ACR), and discusses the mechanisms of reaction and expansion. The main methods of mitigating ASR are presented as well.

Session 3 - Symptoms of ASR

This session describes the symptoms of ASR and provides case studies of ASR-induced damage in concrete structures. Methods of monitoring and evaluating existing structures for ASR are described.

Session 4 - ASR Test Methods

This session reviews available ASR test methods, including tests for aggregates, supplementary cementing materials (SCMs), etc. The instructors make recommendations for test methods based on the state-of-the-practice.

Session 5 - Prevention of ASR

This session discusses various strategies for preventing alkali-silica reaction (ASR) in concrete, including avoiding reactive aggregates, controlling the alkali content of the concrete, using supplementary cementing materials, and the use of lithium-based compounds.

Session 6 - ASR Specifications

This session discusses performance-based and prescriptive specifications for alkali-aggregate reaction. The instructors focus on AASHTO PP65-11 "Standard Practice for Determining the Reactivity of Concrete Aggregates and Selecting Appropriate Measures for Preventing

Deleterious Expansion in New Concrete Construction." Requirements within ASTM and ACI are also discussed.

Session 7 - Diagnosis and Prognosis of ASR

This session describes field and laboratory evaluations to confirm that AAR is the main cause or a significant contributor to the deterioration observed. In addition, this session covers management of ASR-affected structures, which includes in-situ monitoring of the deterioration.

Session 8 - Repair Methods

This session presents repair methods to mitigate ASR-induced damage in newly-constructed or existing concrete structures, including: topical application of silanes and similar products, application of lithium compounds, and confinement.

1.4 RELATED RESOURCES

Some relevant publications and guidance developed under the FHWA ASR Development and Deployment Program include:

- *Report on the Diagnosis, Prognosis, and Mitigation of Alkali-Silica Reaction in Transportation Structures*. The report is available online at:
  http://www.fhwa.dot.gov/pavement/concrete/pubs/hif09004/asr00.cfm
- *Alkali-Silica Reactivity Field Identification Handbook*, which can be downloaded online at: http://www.fhwa.dot.gov/pavement/pub_details.cfm?id=800
- *Alkali-Silica Reactivity Surveying and Tracking Guidelines*, which can be downloaded online at: http://www.fhwa.dot.gov/pavement/pub_details.cfm?id=820
- The online *FHWA ASR Reference Center*, which includes research reports, guidance documents, specifications related to ASR, field trials reports, and other helpful links. It can be accessed at: http://www.fhwa.dot.gov/pavement/concrete/asr/reference.cfm

REFERENCES

AASHTO. 2011. PP65-11, "Standard Practice for Determining the Reactivity of Concrete Aggregates and Selecting Appropriate Measures for Preventing Deleterious Expansion in New Concrete Construction," Washington, D.C.

109th Congress. 2005. House Report 109-203, "Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users," Section 5203.

2 – Fundamentals of Alkali-Aggregate Reaction

2.1 INTRODUCTION

This section provides a brief history of alkali-silica reaction (ASR) and alkali-carbonate reaction (ACR), and discusses the mechanisms of reaction and expansion. Requirements for both ASR and ACR include a source of reactive aggregate, a sufficient concentration of alkalis and moisture availability. Sources of reactive silica and the role of alkalis in ASR are discussed in detail.

2.2 TYPES OF ALKALI-AGGREGATE REACTION

Alkali-aggregate reaction (AAR) is a reaction in concrete between the alkali hydroxides, which originate mainly from the portland cement, and certain types of aggregate. Two types of AAR are currently recognized; these are alkali-silica reaction (ASR) and alkali-carbonate reaction (ACR). As the names imply, these types of reaction differ in that they involve reactions with either siliceous or carbonate phases in the aggregates. The reactions are briefly defined as follows:

AAR   Chemical reaction in either concrete or mortar between hydroxyl ions (OH^-) of the alkalies (sodium and potassium) from hydraulic cement (or other sources), and certain constituents of some aggregates; under certain conditions deleterious expansion of concrete or mortar may result.

ACR   Chemical reaction in either concrete or mortar between hydroxyl ions (OH^-) of the alkalies (sodium and potassium) from hydraulic cement (or other sources) and certain carbonate rocks, particularly calcitic dolostone and dolomitic limestones, present in some aggregates. The reaction is usually accompanied by dedolomitization and expansion of the affected aggregate particles, leading to abnormal expansion and cracking of concrete in service.

ASR   Chemical reaction in either concrete or mortar between hydroxyl ions (OH^-) of the alkalies (sodium and potassium) from hydraulic cement (or other sources), and certain siliceous rocks and minerals, such as opal, chert, microcrystalline quartz, and acidic volcanic glass, present in some aggregates. This reaction and the development of the alkali-silica gel reaction product can, under certain circumstances, lead to abnormal expansion and cracking of the concrete.

ASR is far more widespread than ACR and is the main focus of this workshop. However, it should be noted that some test methods used to detect alkali-silica reactive aggregates may fail to detect alkali-carbonate reactive aggregates. In addition, measures used to prevent damaging ASR are generally ineffective in preventing ACR expansion and, consequently, alkali-carbonate reactive rocks should not be used in the production of concrete.

2.3 HISTORY OF AAR

Problems due to ASR were first identified in the State of California in the 1930s and reported by Thomas Stanton of the California State Division of Highways in 1940 (Stanton 1940). Stanton's studies demonstrated that the expansion of mortar bars was influenced by the alkali content of the cement, the type and amount of the reactive silica in the aggregate, the availability of moisture and temperature. He further showed that expansion was negligible when the alkali content of the cement was below 0.60% Na_2Oe and that expansion could be reduced by pozzolans, thus setting the groundwork for preventive measures. Subsequent to Stanton's discovery, ASR was diagnosed as the cause of abnormal cracking in a number of dams operated by the U.S. Bureau of Reclamation and in the 1940s a number of agencies initiated studies on ASR in the U.S.A. (Army Corps of Engineers, Bureau of Public Roads, Portland Cement Association) and other countries (Denmark and Australia). ASR is now recognized as a major cause of concrete deterioration in the U.S.A., incidences having occurred in most, if not all, of the contiguous states, and worldwide.

Alkali carbonate reaction (ACR) was first discovered by Swenson (1957) as the cause of concrete deterioration in Canada and was subsequently implicated in cases of degradation of concrete structures in the USA (Hadley 1961). Unlike ASR, problems with ACR are restricted to a few isolated locations worldwide. Consequently, there has been comparatively little research conducted on this topic.

2.4 FUNDAMENTALS OF ASR

2.4.1 ASR Mechanisms of Reaction and Expansion

Figure 2.1 shows the sequence of events in the alkali-silica reaction. The pore solution of concrete is composed primarily of sodium (Na^+) and potassium (K^+) ions, balanced by hydroxyl ions (OH^-), giving rise to a high pH (typically 13.2 to 14.0). At high pH, the OH^- ions "attack" the silanol (Si-OH) groups and, subsequently, the siloxane (Si-O-Si) groups of reactive silica present in the aggregate, causing dissolution of the silica.

Concrete pore solution is dominated by Na, K & OH (with minor amounts of Ca).

If the silica in the aggregate is reactive – the OH and then the Na & K will react with the SiO_2

The product of the reaction is in alkali-silica gel composed of Na, K, Ca & Si

The gel forms around and within the aggregate

The gel imbibes water from the surrounding cement paste

The gel expands

Eventually the swelling pressures may exceed the tensile strength of the surrounding paste and cause expansion & cracking of the concrete
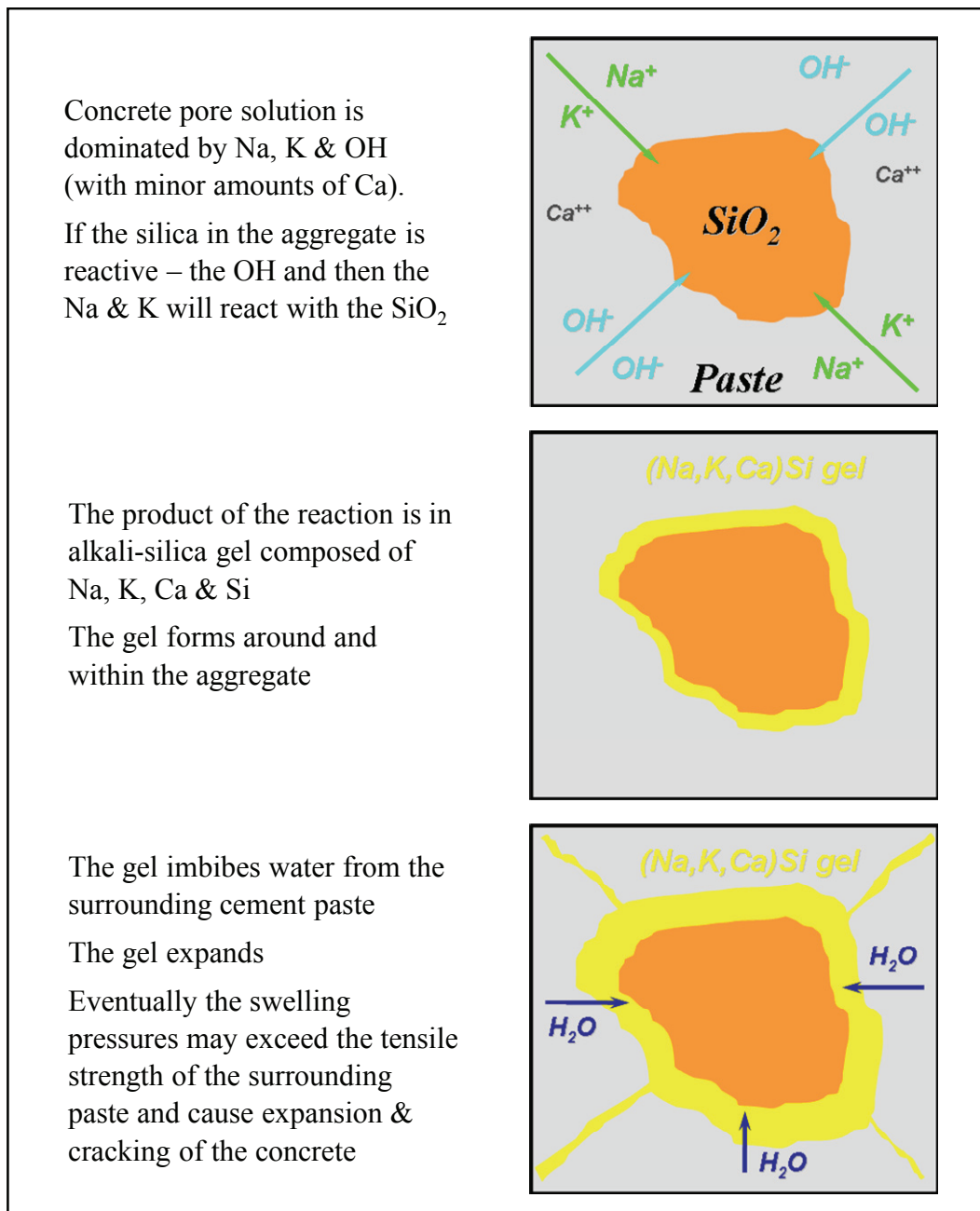
Figure 2.1. Alkali-Silica Reaction Sequence

These reactions can be represented by Eqns. 2.1 and 2.2 as follows (modified from Dent Glasser and Kataoka 1981)[1]:

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[1] Note that Na is used in these equations to represent both sodium (Na) and potassium (K); similar equations can be written replacing Na for K.

$$\text{Si-OH} + \text{OH}^- + \text{Na}^+ \rightarrow \text{Si-O-Na} + \text{H}_2\text{O} \qquad \text{Eqn. 2.1}$$

$$\text{Si-O-Si} + 2\text{OH}^- + 2\text{Na}^+ \rightarrow 2(\text{Si-O-Na}) + \text{H}_2\text{O} \qquad \text{Eqn. 2.2}$$

In the presence of calcium, the silica precipitates from solution as an alkali-silica gel, primarily composed of sodium, potassium and silica, with small amounts of calcium. The gel, which forms within and/or around the aggregate, is hygroscopic and has the capacity to absorb water and swell. Under certain conditions, the formation and subsequent swelling of the gel may cause expansion and, eventually, cracking of the concrete.

Figure 2.2 shows a petrographic thin section of ASR-affected concrete viewed under a microscope with transmitted light. The image shows a flint sand particle (at left) embedded in cement paste. The flint particle has reacted, expanded and cracked, and the crack extends out into the surrounding cement paste. The crack within the aggregate and in the paste is partially filled with a reaction product, alkali-silica gel.
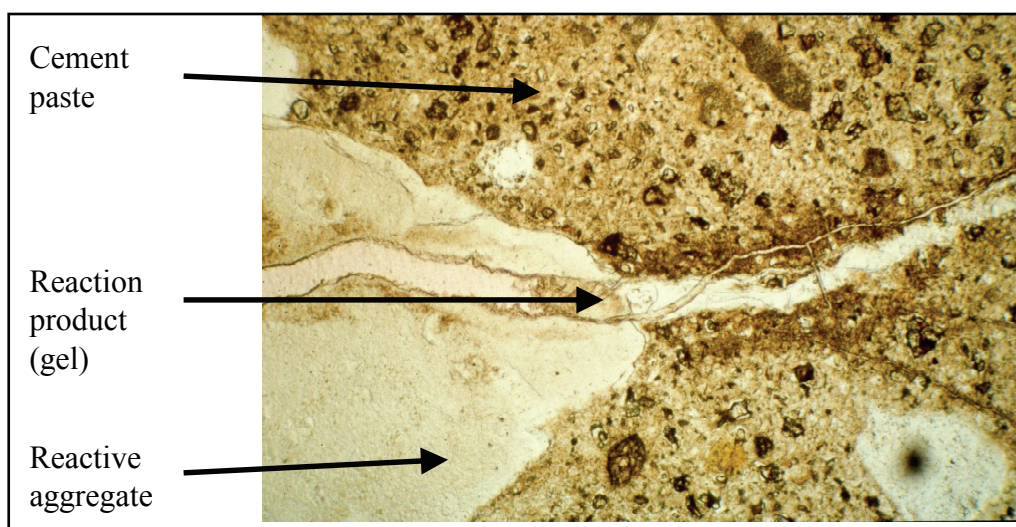
Figure 2.2. Photograph of a Thin Section of Concrete Undergoing ASR (field of view ~ 5 mm)

Despite general acceptance of the chemical reactions involved, a number of different mechanisms of expansion have been proposed as follows:

- Hansen (1944) suggested that the cement paste surrounding reactive grains acts as a semi-permeable membrane through which water (or pore solution) may pass but not the larger complex silicate ions. The water is drawn into the reacting grain where its chemical potential is lowest. An osmotic pressure cell is formed and increasing hydrostatic pressure is exerted on the cement paste, inevitably leading to cracking of the surrounding mortar.

- McGowan and Vivian (1952) disputed the classical osmotic theory on the basis that cracking of the surrounding cement paste "membrane" due to ASR would relieve hydraulic pressure and prevent further expansion. They proposed an alternative mechanism based on the physical absorption of water by the alkali silica-gel and subsequent swelling of the gel.

- Powers and Steinour (1955a, 1955b) proposed a compromise, suggesting that both osmotic and imbibition pressures may be generated depending on whether the alkali-silicate complex is fluid or solid. In their hypothesis, the reaction product itself may act as a semi-permeable membrane depending on its composition.

Regardless of the mechanism, the fundamental cause of swelling is thermodynamically the same, i.e. the entry of water into a region where the effect of a solute or of adsorption reduces its free energy.

A number of experimental studies have concluded that significant expansion only occurs when an adequate supply of calcium is available as calcium hydroxide [$Ca(OH)_2$]. In systems with abundant alkali hydroxides and reactive silica, but no calcium hydroxide, silica dissolves and remains in solution. Although the precise role calcium plays in gel expansion remains unclear, a series of mechanisms have been proposed, as listed below (ACI 201):

- Calcium promotes "alkali recycling" by replacing alkalies in the reaction product thereby regenerating alkalies for further reaction (Thomas 2001; Hansen 1944).

- $Ca(OH)_2$ provides a reservoir of OH^- ions to maintain a high level of OH^- in solution (Wang and Gillott 1991).

- High calcium concentrations in the pore solution prevent the diffusion of silica away from reacting aggregate particles (Chatterji 1979; Chatterji and Clausson-Kass 1984).

- If calcium is not available, reactive silica dissolves in alkali hydroxide solution without causing damage (Thomas 1998; Diamond 1989).

- The formation of calcium-rich gels is necessary to cause expansion either directly or indirectly through the formation of a semi-permeable membrane around reactive aggregate particles (Thomas 1998; Thomas et al. 1991; Bleszynski and Thomas 1998).

Although the precise role(s) of $Ca(OH)_2$ is equivocal, it is evident that calcium must be available for damaging reaction to occur. Thus, reducing the availability of calcium – for example, by consuming $Ca(OH)_2$ through pozzolanic reactions – should result in a reduction in the expansion due to ASR.

2.4.2 Requirements for ASR

On the basis of the discussion on reaction and expansion mechanisms above, it is clear that there are three requirements for damaging ASR to occur; these are:

- A sufficient quantity of reactive silica (within aggregates)

- A sufficient concentration of alkali (primarily from portland cement)

- Sufficient moisture

Elimination of any one of these requirements will prevent the occurrence of damaging alkali-silica reaction; this is discussed further in the sections dealing with prevention and mitigation of ASR. Each of these requirements is discussed below.

2.4.2.1 Sources of Reactive Silica

Silica, SiO_2, is a component of many rocks; however, not all forms of silica react significantly with the pore solution of concrete and, thus, not all siliceous aggregates produce damaging ASR. For example, the mineral quartz is stable whereas the mineral opal is highly reactive, although both are silica minerals with similar chemical composition, being primarily composed of SiO_2. Figure 2.3 shows the differences in the solubility of the silica and the crystalline structure of these two minerals. Opal has a highly disordered (amorphous) structure which renders it unstable at high pH and, as such, aggregates containing significant quantities of the mineral opal may be expected to react and result in expansion when used in concrete, provided there is sufficient alkali present. On the other hand, quartz will not react deleteriously regardless of the alkali content of the concrete.

(a) Soluble Silica in Aggregates

(b) Schematic Showing Crystalline Structure of Opal (top) and Quartz (bottom)
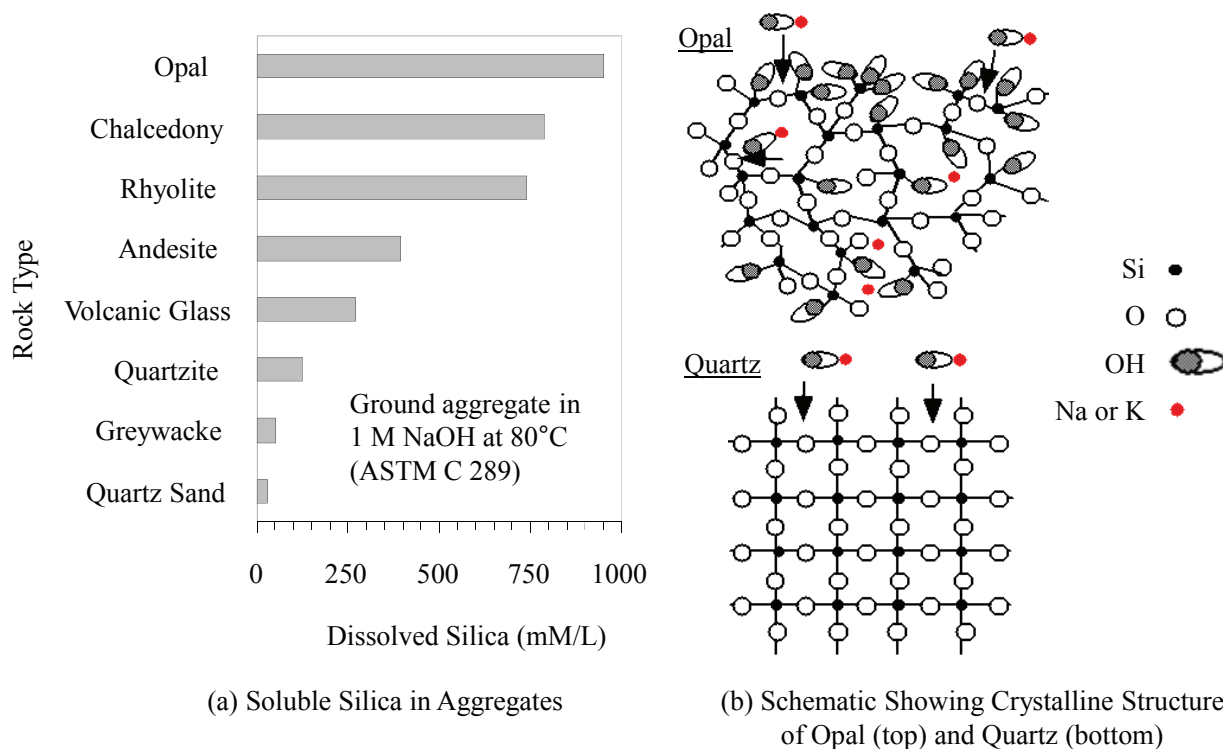
Figure 2.3. Solubility and Structure of Opal and Quartz

The following silica minerals are considered to be alkali-silica reactive: opal, tridymite, cristobalite, volcanic glass, chert, cryptocrystalline (or microcrystalline) quartz and strained quartz. These minerals may be found in the following rock types: shale, sandstone, silicified carbonate rocks, chert, flint, quartzite, quartz-arenite, gneiss, argillite, granite, greywacke, siltstone, arenite, arkose and hornfels. However, this does not mean that all sources of such rocks will produce deleterious reaction when used in concrete. For example, granitic aggregate is widely used in concrete and only certain sources produce damaging ASR. The reactivity of a rock depends on the type and quantity of reactive minerals present, if any.

Although the presence of reactive minerals can usually be detected by an experienced petrographer, appropriate performance testing of specific aggregate sources is recommended to confirm alkali-silica reactivity. Test methods are discussed in Section 4.

2.4.2.2 Sources of Alkali

While portland cement is considered the main contributor of alkalis, under certain conditions, other materials may provide additional alkalis that are available to the reaction. The source of alkalis can be from any of the following:

- Portland cement

- Supplementary cementing materials (e.g., fly ash, slag, silica fume, natural pozzolans)

- Aggregates

- Chemical admixtures

- External sources (e.g., seawater and deicing salts)

- Wash water (if used)

The quantity of alkalis in portland cement is typically expressed in terms of equivalent sodium (written either Na_2Oe or Na_2Oeq) which may be calculated using the following formula:

$$Na_2Oe = Na_2O + 0.658 \times K_2O \hspace{4cm} \text{Eqn. 2.3}$$

where: Na_2O and K_2O are the mass percentages of sodium oxide and potassium oxide in the portland cement as reported on the cement mill test report.

Although the percentage of alkalis in portland cement is relatively low (in the range of 0.2 to 1.3% Na_2Oe for most North American sources) in comparison to other oxides, the alkalis are highly soluble and dominate the pore solution of the concrete. Figure 2.4 shows the composition of the pore solution extracted from portland cement pastes and mortars at various ages. During the first few hours the pore solution is mainly composed of alkali sulfates (Na^+, K^+ and SO_4^{2-}), but beyond this time the formation of solid sulfate phases (calcium mono-sulfo-aluminate and ettringite) results in a reduction of the SO_4^{2-} concentration in solution, and hydroxyl ions (OH^-) come into solution to maintain balance with the positively charged alkali cations. Beyond the first day, the pore solution is composed almost entirely of Na^+, K^+ and OH^-.

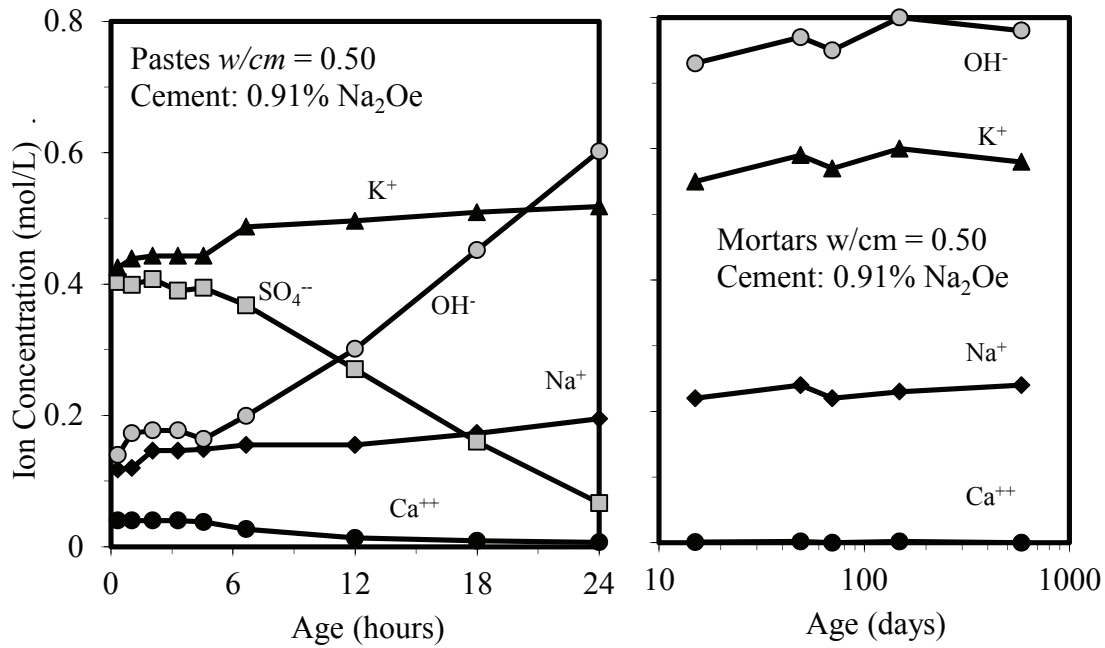Figure 2.4. Pore Solution Composition in Portland Cement Pastes and Mortars (Diamond 1983)

Figure 2.5 shows that the alkali concentration of the pore solution is strongly influenced by the alkali content of the portland cement. Note in Figure 2.5 that the OH⁻ concentration ranges from about 0.15 M for the lowest alkali cement to about 1.00 M for the highest; this represents a pH range from 13.2 to 14.0.
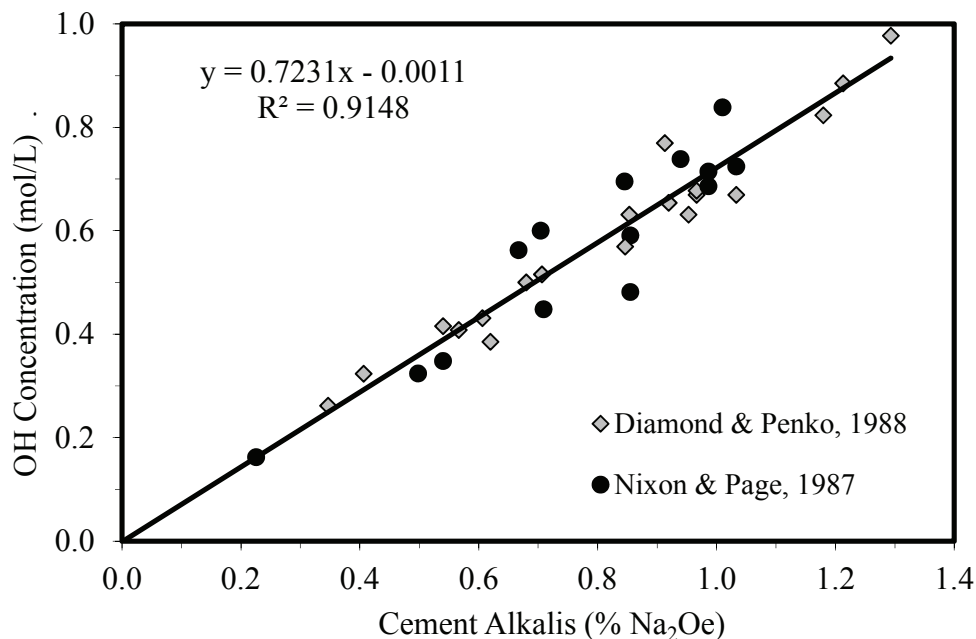
Figure 2.5. Relationship between the Alkalinity of the Pore Solution and the Alkali Content of the Portland Cement (Diamond and Penko 1992; Nixon and Page 1987)

Based on Stanton's early work (1940), it was proposed for many years that expansion resulting from the alkali-silica reaction is unlikely to occur when the alkali content of the cement is below 0.6% Na$_2$Oe. Many specifications allow the use of potentially-reactive aggregates provided that the cement alkali content does not exceed 0.6% Na$_2$Oe. However, it is now recognized that limiting the alkali content of portland cement is not, by itself, an effective way of preventing ASR-induced damage, because this approach does not control the total alkali content of the concrete mixture.

Limiting the maximum alkali content of concrete is the preferred approach when specifying alkali levels. The alkali content of portland cement concrete is calculated by multiplying the cement content by the alkali content of the cement. Figure 2.6 illustrates the effects of the concrete alkali content on the expansion of concrete prisms stored over water at 38°C (100°F) for one year. Concrete mixtures were produced with a reactive siliceous limestone and varying cement contents (275 to 450 kg/m^3, 458 to 750 pcy) using cements with a wide range of alkali contents (0.67 to 1.40% Na$_2$Oe). From the data it is evident that the expansion of concrete with a given reactive aggregate is dependent on the alkali content of the concrete. Figure 2.6 indicates that deleterious expansion of the concrete prisms containing this reactive aggregate may be prevented if the alkali content of the concrete is kept below 3.0 kg/m^3 (5.0 lb/yd^3) Na$_2$Oe. It should be noted that expansion may occur in the field at lower alkali contents than that found necessary to cause expansion in the laboratory. The reason for this is that a portion of the alkalis may be lost through leaching under the conditions of the concrete prism test (Thomas et al,

2006). For example, the aggregate for which expansion data are presented in Figure 2.6 caused expansion and cracking of field-exposed concrete blocks (approximately 0.6 x 0.6 x 2.0 m, 2 x 2 x 6 ft.) with an alkali content of just 1.9 kg/m^3 (3.2 lb/yd^3) Na$_2$Oe (MacDonald et al. 2012).
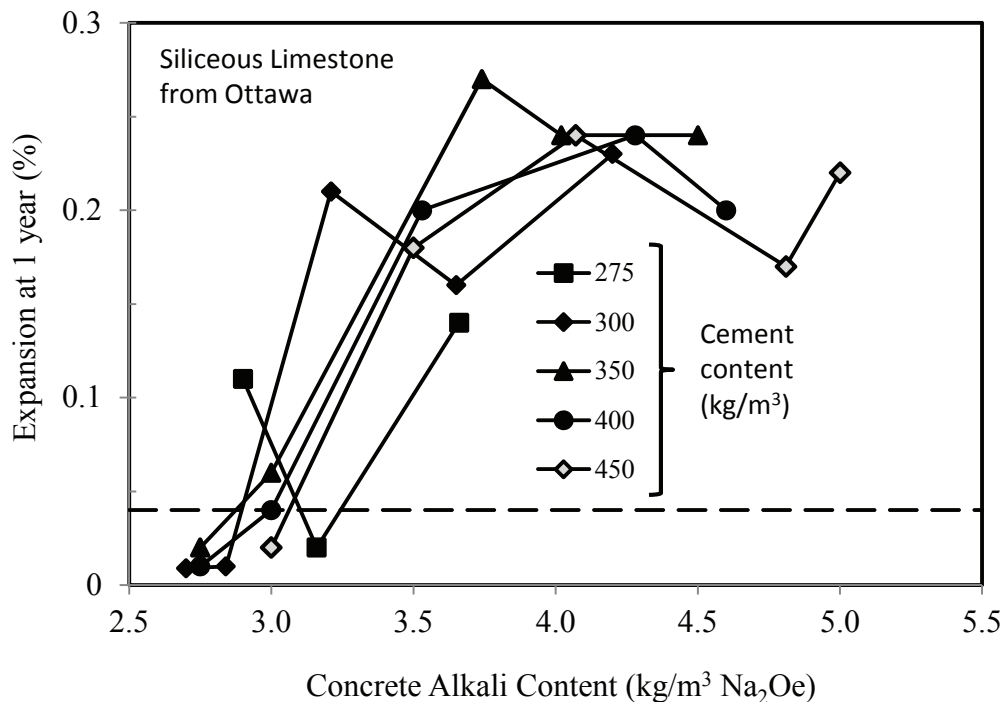
Figure 2.6. Effect of Concrete Alkali Content on Expansion

The alkali content of concrete may increase during service due to (i) alkali migration caused by moisture movements or electrical currents, (ii) penetration of alkalis from external sources (e.g. deicing salts), or (iii) long-term release of alkali from aggregates. These factors should be taken into consideration when setting alkali limits for concrete containing reactive aggregates.

2.4.2.3 Role of Moisture

Sufficient moisture is required to both sustain the chemical reaction and to provide for the expansion of the gel. It is generally considered that the chemical reaction will cease if the internal relative humidity inside the concrete falls below 80% (Figure 2.7). Local differences in moisture availability within the same structure can result in very different levels of ASR damage occurring within that structure. Specifically, portions of the structure exposed to a constant or steady source of moisture (e.g., as a result of poor drainage or poor detailing) can exhibit significant ASR-induced damage, while other portions of the structure that remain essentially dry may show little or no damage.
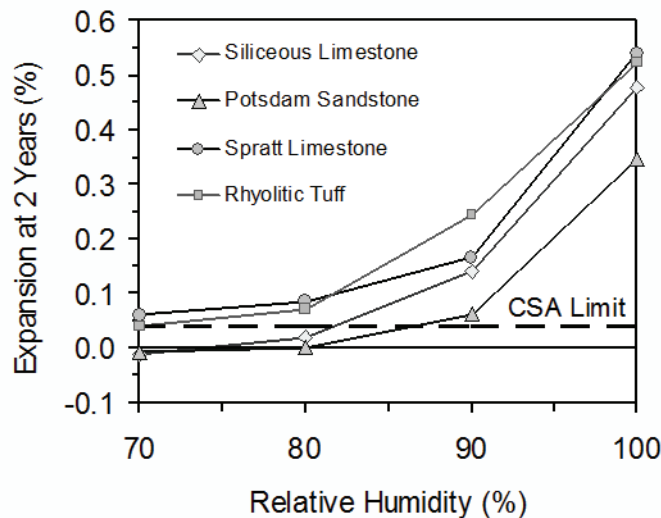
Figure 2.7. Effect of Relative Humidity on Expansion of Concrete Prisms (Pedneault 1996)

2.5 FUNDAMENTALS OF ACR

Alkali-carbonate reaction occurs between alkali hydroxides and certain argillaceous dolomitic limestones; these dolomites are characterized by a matrix of fine calcite and clay minerals with scattered dolomite rhombohedra. The reaction is manifested in the rapid expansion and extensive cracking of concrete, and structures affected by ACR usually show cracking within five years or less. Although there is a lack of consensus regarding the precise mechanisms involved, it is generally agreed that the reaction is accompanied by the dedolomitization process, as follows:

$$Ca.Mg(CO_3)_2 + 2ROH \rightarrow CaCO_3 + Mg(OH)_2 + R_2CO_3 \qquad \text{Eqn 2.4}$$
dolomite + alkali hydroxide → calcite + brucite + alkali carbonate
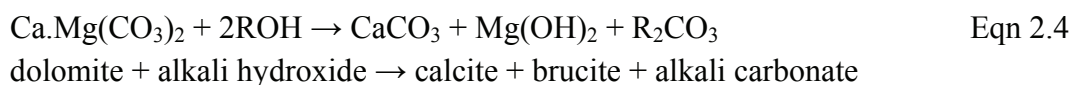
where R represents K or Na. However, since this reaction results in a reduction in solid volume, the expansion must be attributed to an alternative mechanism. Several theories have been proposed to explain the expansion mechanism (Swenson and Gillott 1964; Tang et al. 1987; Fournier and Berube 2000), and these include:

- Hydraulic pressures caused by the migration of water molecules and alkali ions into the restricted spaces of the calcite/clay matrix around the dolomite rhombs;

- Adsorption of alkali ions and water molecules on the surfaces of the "active" clay minerals scattered around the dolomite grains; and

- Growth and rearrangement of the products of dedolomitization (i.e., brucite and calcite).

The alkali carbonate produced in the dedolomitization reaction may react with calcium hydroxide in the cement paste as follows:

$$R_2CO_3 + Ca(OH)_2 \rightarrow CaCO_3 + 2ROH \hspace{3cm} \text{Eqn. 2.5}$$

thereby "regenerating" alkalis for further reaction. Thus, provided there is sufficient alkali available to initiate the reaction, the process may continue independently of the amount of alkalis available in the concrete. This could explain why low-alkali cements are not effective in controlling damaging reaction in some instances.

As with ASR, a source of reactive aggregate, sufficient alkali and a supply of moisture is required to sustain alkali-carbonate reaction. Although it has been reported (Newlon et al. 1964) that ACR may be prevented by using low-alkali cement, damaging expansion has been observed in structures with very low alkali contents.

REFERENCES

Bleszynski, R.F. and Thomas, M.D.A. 1998 "Microstructural Studies of Alkali-Silica Reaction in Fly Ash Concrete Immersed in Alkaline Solutions." Advanced Cement Based Materials, 7, 66-78.

Chatterji, S. 1979, "The Role of $Ca(OH)_2$ in the Breakdown of Portland Cement Concrete due to Alkali-Silica Reaction." Cement and Concrete Research, 9(2), 185-188.

Chatterji, S. and Clausson-Kass, N.F. 1984 "Prevention of Alkali-Silica Expansion by Using Slag-Portland Cement." Cement and Concrete Research, 14(6), 816-818.

Dent Glasser, L.S. and Kataoka, N. 1981. "The chemistry of alkali-aggregate reactions." Proceedings of the 5th International Conference on Alkali-Aggregate Reaction, Cape Town, S252/23.

Diamond, S. 1983. "Alkali reactions in concrete - Pore solution effects." Proceedings of the 6th International Conference on Alkalis in Concrete, (Ed. G.M. Idorn and Steen Rostam), Danish Concrete Association, Copenhagen, 155-166.

Diamond, S. 1989. "ASR - Another Look at Mechanisms." Proceedings of the 8th International Conference on Alkali-Aggregate Reaction, (Eds. K. Okada, S. Nishibayashi, and M. Kawamura), Kyoto, Japan, 83–94.

Diamond, S. and Penko, M. 1992. "Alkali silica reaction processes: the conversion of cement alkalis to alkali hydroxide." Durability of Concrete – G.M. Idorn International Symposium (Ed. Jens Holm), ACI SP-131, American Concrete Institute, Detroit, 153-168.

Fournier, B. and Berube, M-A. 2000. "Alkali-aggregate reaction in concrete: a review of basic concepts and engineering implications." Canadian Journal of Civil Engineering, 27, 167-191.

Hadley, D.W. 1961. "Alkali reactivity of carbonate rocks: expansion and dedolomitization." Proceedings of the Highway Research Board, 40, 462-474.

Hansen, W.C. 1944. "Studies relating to the mechanism by which the alkali-aggregate reaction proceeds in concrete." Journal of the American Concrete Institute, 15, 3, 213-227.

MacDonald, C.A., Rogers, C. and Hooton, R.D. 2012. "The relationship between laboratory and field expansion – observations at the Kingston outdoor exposure site for ASR after twenty years." Proceedings of the 14th International Conference on Alkali-Aggregate Reaction in Concrete, Austin, TX, USA, May.

McGowan, J.K. and Vivian, H.E. 1952. "Studies in cement-aggregate reaction: correlation between crack development and expansion of mortars." Australian Journal of Applied Science, 3, 228-232.

Newlon, H.H., Jr. and Sherwood, W.C. (1964). "Methods for Reducing Expansion of Concrete Caused by Alkali-Carbonate Rock Reactions." Highway Research Record No. 45, Highway Research Board, Washington, D.C., 134-150.

Nixon, P.J. and Page, C.L. 1987. "Pore solution chemistry and alkali aggregate reaction." In Concrete Durability, Katherine and Bryant Mather International Conference, (Ed. J.M. Scanlon), ACI SP-100, Vol. 2, American Concrete Institute, Detroit, 1833-1862.

Pedneault, A. 1996. "Development of testing and analytical procedures for the evaluation of the residual potential of reaction, expansion, and deterioration of concrete affected by ASR." M.Sc. Memoir, Laval University, Québec City, Canada, 133p.

Powers, T.C. and Steinour, H.H. 1955a. "An investigation of some published researches on alkali-aggregate reaction. I. The chemical reactions and mechanism of expansion." Journal of the American Concrete Institute, 26(6), 497-516.

Powers, T.C. and Steinour, H.H. 1955b. "An interpretation of some published researches on the alkali-aggregate reaction. Part 2: A hypothesis concerning safe and unsafe reactions with reactive silica in concrete." Journal of the American Concrete Institute, 26(8), 785-811.

Stanton, T.E. 1940. "Expansion of concrete through reaction between cement and aggregate," Proceedings of the American Society of Civil Engineers, 66(10), 1781-1811.

Swenson, E.G. 1957. "A reactive aggregate undetected by ASTM tests." Proceedings of American Society for Testing and Materials, 57, 48-51.

Swenson, E.G. and Gillott, J.E. 1964. "Alkali-carbonate rock reaction." Highway Research Record, 45, 21-40.

Tang, M., Liu, Z. and Han, S. 1987. "Mechanism of alkali-carbonate reaction." Proceedings of the 7th International Conference on Concrete Alkali-Aggregate Reactions, (Ed. P.E. Grattan-Bellew), Noyes Publications, New Jersey, 275-279.

Thomas, M.D.A. 1998 "The Role of Calcium in Alkali-Silica Reaction." Materials Science of Concrete - The Sidney Diamond Symposium, (Eds. M. Cohen, S. Mindess, and J.P. Skalny), American Ceramic Society, Westerville, OH, 325-331.

Thomas, M.D.A. 2001. "The role of calcium hydroxide in alkali recycling in concrete." Materials Science of Concrete Special Volume on Calcium Hydroxide in Concrete, (Eds. J. Skalny, J. Gebauer, and I. Odler), American Ceramic Society, Westerville, OH, 269-280.

Thomas, M.D.A., Nixon, P.J. and Pettifer, K. 1991. "The effect of pfa on alkali-silica reaction." In Second CANMET/ACI Conference on the Durability of Concrete (Ed. V.M. Malhotra), ACI SP-126, II, American Concrete Institute, Detroit, 919-940.

Thomas, M.D.A., Fournier, B., Folliard, K., Ideker, J. and Shehata, M. 2006. "Test methods for evaluating preventive measures for controlling expansion due to alkali-silica reaction in concrete." Cement and Concrete Research, 36(10), 1842-1856.

Wang, H. and Gillott, J.E. 1991. "Mechanism of Alkali-Silica Reaction and the Significance of Calcium   Hydroxide."   Cement   and   Concrete   Research,   21(4),   647-654.

3 – Symptoms of Alkali-Silica Reaction (ASR)

3.1 INTRODUCTORY REMARKS

As mentioned before, three conditions are necessary to initiate and sustain AAR in concrete: (1) reactive mineral forms must be present in the aggregate materials; (2) the concentration of alkali hydroxides ($[Na^+, K^+\text{-}OH^-]$) in the concrete pore fluid must be high, and (3) sufficient moisture must be present. Concrete elements affected by AAR respond quite differently from one another, reflecting wide variations in the above conditions.

Common visual symptoms of ASR consist of:

- Cracking

- Expansion causing deformation, relative movement, and displacement

- Localized crushing of concrete

- Extrusion of joint (sealant) material

- Surface pop-outs

- Surface discoloration and gel exudations

A brief description and photographs of these symptoms are given hereafter. Additional photographs could be found in Thomas et al. (2011).

3.2 CRACKING

The pattern of cracking due to ASR is influenced by factors such as the shape or geometry of the concrete member, the environmental conditions, the presence and arrangement of reinforcement, and the load or stress fields (restraint) applied to the concrete. The classic symptom of ASR is *map cracking* (also called *pattern cracking*), which takes the form of randomly-oriented cracks on the surface of concrete elements that are relatively free (unrestrained) to move in all directions (Figure 3.1A&B). However, drying shrinkage, freezing/thawing cycles, and sulfate attack can also result in a pattern of cracks showing a random orientation.

A

B

C

D

E

Figure 3.1. (A). Map-cracking in the wing wall of a 30-year-old bridge structure affected by ASR (CSA 2000). (B). Severe map-cracking and associated gel staining around cracks in a median highway barrier affected by ASR. (C). Well-defined crack pattern associated with the development of ASR in highway pavement; the orientation of predominant cracks is longitudinal, while map- or pattern-cracking is also identified. (D). Preferred alignment of cracks in an ASR-affected concrete column. (E). Longitudinal cracking in a precast, reinforced concrete beam affected by ASR.

Steel reinforcement or other restraint arising from applied compressive stress may reduce ASR expansion in concrete. However, surface cracking due to AAR is often not significantly reduced by the use of internal or external restraint. When expansion is restrained in one or more directions, more expansion occurs in the direction of least confinement, and the cracks become oriented in the same direction as the confining stresses. For example, with concrete pavements, the expansion being restrained in the longitudinal direction, a greater amount of expansion occurs in the transverse direction and cracks develop preferentially in the longitudinal direction (Figure 3.1C); in the case of reinforced concrete columns, cracks tend to be aligned vertically due to the restraint imposed by the primary reinforcement and the dead load (Figure 3.1D); for prestressed bridge girders, the cracks will usually be aligned horizontally due to the confinement imposed by the prestressing tendons parallel to the beam axis (Figure 3.1E).

In many cases, discoloration occurs around the cracks, often due to gel exudation in the vicinity of the cracks (Figure 3.1B).

Cracking is usually most severe in areas of structures where the concrete has a renewable supply of moisture, such as close to the waterline in piers, from the ground behind retaining walls, beneath pavements slabs, elements/parts of structures exposed to rain or by wick action in piers or columns (Figure 3.2).

A

B

C

D

E

Figure 3.2. (A&B). Map-cracking in wing walls (exposed elements) of bridge structures affected by ASR. (C). Cracking in the column and the exposed portion of a beam affected by ASR. (D&E). Cracking in the exposed portion (above ground) of the wing wall of a bridge structure affected by ASR; the wing wall has been excavated before being demolished. The lower ground portion shows limited cracking; discoloration occurs around the cracks above ground (E).

Concrete members undergoing ASR and experiencing cyclic exposure to sun, rain and wind, or portions of concrete piles in tidal zones often show more severe surface cracking resulting from induced tension cracking in the "less expansive" (due to alkali leaching/dilution processes, variable humidity conditions, etc.) surface layer under the expansive thrust of the inner concrete core (Stark and Depuy 1987, ACI 1998) (Figure 3.3).

A
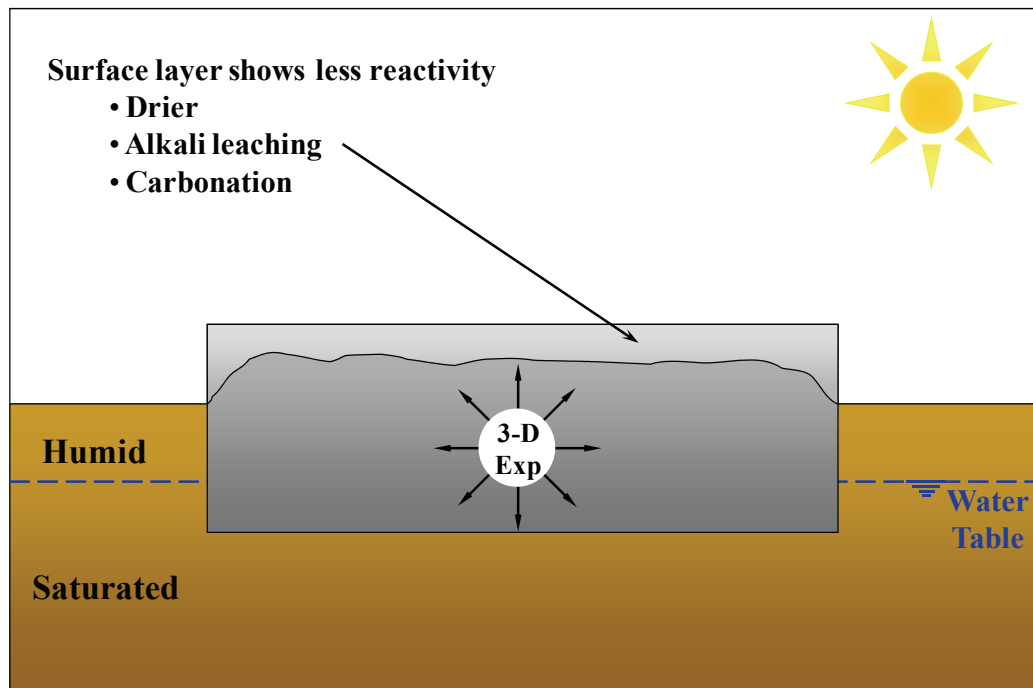
B
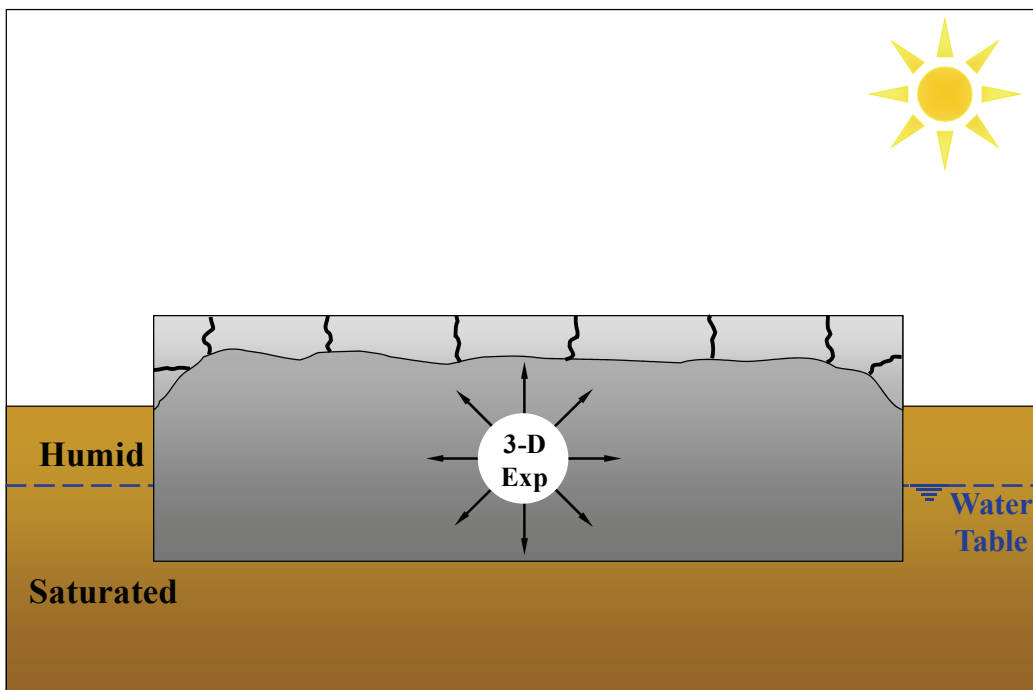
Figure 3.3. Concrete member undergoing ASR and experiencing cyclic exposure to sun, rain and wind shows more severe surface cracking resulting from induced tension cracking in the "less expansive" (due to alkali leaching/dilution processes, variable humidity conditions, etc.) surface layer under the expansive thrust of the inner concrete core.

3.3 EXPANSION CAUSING MOVEMENTS AND DEFORMATIONS

The extent of ASR often varies between or within the various members/parts of an affected concrete structure, thus causing distresses such as:

- relative movement of adjacent concrete members or structural units (Figure 3.4A);
- deflection, closure of joints with associated squeezing/extrusion of sealing materials (Figures 3.4B&C); and
- ultimately, spalling of concrete at joints (Figures 3.4C-E).

Figure 3.4. (A). Relative movement of abutting sections of parapet wall in a bridge structure affected by ASR (Stark 1991). (B). Expansion of bridge girder leading to loss of clearance between the girder and embankment and eventually crushing of the girder end with localized spalling. (C). Expansion causing spalling at joints in a concrete pavement incorporating a highly-reactive aggregate. (D). Expansion causing spalling at joints and extrusion of sealant material in a concrete pavement incorporating a highly reactive aggregate. (E). Expansion with associated severe spalling in abutting jersey barrier sections affected by ASR.

It is important to remember that deformations in concrete structures may be caused by a range of different mechanisms, such as loading, thermal or moisture movements, differential shrinkage, gravity and foundation effects, hydraulic pressure, creep, impact, and vibrations (BCA 1992).

3.4 POP-OUTS

The expansion of individual unsound or frost-susceptible aggregate particles [such as laminated, schistose and argillaceous, clayey or porous particles or certain varieties (porous) of chert, ironstones] at or near the concrete surface due to frost-action is likely to be the main factor for the development of pop-outs in northern regions (Figure 3.5A). Pop-outs can also be caused by a poor bond between the cement paste and dusty coarse aggregate particles. Alkali-silica reactive aggregates undergoing expansion near the concrete surface may also induce the detachment of a portion of the skin of concrete leaving the reactive aggregate in the bottom (Figure 3.5B).

A
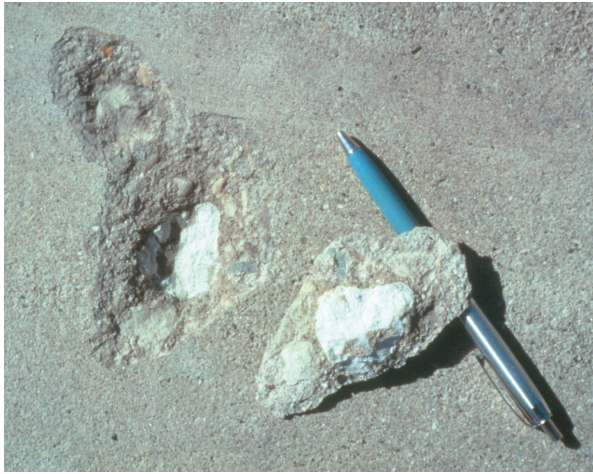
B
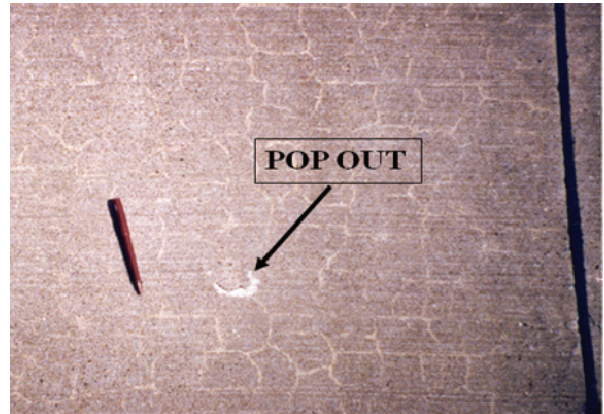
C

D

E

Figure 3.5. (A). Pop-out created by the expansion of a frost-susceptible porous coarse aggregate particle (leached chert). (B). ASR-induced pop-out in a concrete pavement incorporating highly-reactive aggregates; also noted pattern cracking. (C). Efflorescence and exudations of alkali-silica gel at the surface of the concrete foundation of a 25-year-old highway bridge affected by ASR. (D). Efflorescence and exudations of alkali-silica gel at the surface of a small concrete monument affected by ASR. (E). Surface discoloration and exudation associated with cracks.

3.5 SURFACE DEPOSITS (GEL EXUDATION VS. EFFLORESCENCE)

Although surface gel exudation is a common and characteristic feature of ASR, the presence of surface deposits is not necessarily indicative of ASR as other mechanisms (such as frost action or the movement of water through cracked concrete members) can also cause surface deposits called efflorescence (without the presence of ASR gel) (Figures 3.5C-E).

REFERENCES

American Concrete Institute (ACI). "State-of-the-Art Report on Alkali-Aggregate Reactivity." ACI 221.1R-98, 1998.

British Cement Association (BCA). "The Diagnosis of Alkali-Silica Reaction – Report of a Working Party." Wexham Springs, Slough (UK), SL3 6PL, 44p., 1992.

Canadian Standards Association (CSA). "Guide to the Evaluation and Management of Concrete Structures Affected by Alkali-Aggregate Reaction." CSA A864-00, Canadian Standards Association, Mississauga, Ontario, Canada, 2000.

Institution of Structural Engineers (ISE). "Structural Effects of Alkali-Silica Reaction – Technical Guidance Appraisal of Existing Structures." Institution of Structural Engineers, 11 Upper Belgrave Street, London SW1X 8BH, 45p., 1992.

Stark, D. "Handbook for the Identification of Alkali-Silica Reactivity in Highway Structures." SHRP-C/FR-91-101, TRB National Research Council, 49p., 1991.

Stark, D. and Depuy, G. "Alkali-Silica Reaction in Five Dams in Southwestern United States." Proceedings of the Katherine and Bryant Mather International Conference on Concrete Durability, Atlanta, Georgia (USA), ACI SP-100, 1759-1786, April 1987.

Thomas, M.D.A., Fournier, B., Folliard, K.J., Resendez, Y.A. 2011. "Alkali-Silica Reactivity Field Identification Handbook." Office of Pavement Technology, Federal Highway Administration, March 2011, 72p.

4 – Test Methods

4.1 INTRODUCTION

This section describes the various test methods currently being used to assess the reactivity of aggregates (with regard to alkali-silica reaction (ASR) and alkali-carbonate reaction (ACR)). In addition to describing and recommending appropriate test methods for evaluating aggregate reactivity, this chapter also provides recommendations for tests aimed at evaluating measures for preventing ASR, such as supplementary cementing materials (SCMs) and lithium nitrate. Only limited information is provided on ACR, with the primary focus on test methods aimed at identifying aggregates susceptible to ACR so that they can be avoided for use in concrete as there are no available preventive measures for such aggregates.

4.2 ASR TEST METHODS FOR EVALUATING AGGREGATE REACTIVTY

Ever since Stanton discovered ASR in the late 1930s, a wide variety of test methods have been developed and implemented. Some have been successful, some have proven to be complete failures, and others fall somewhere in the middle. Through research and development, as well as trial and error, test methods have evolved over the years, and there has been a general convergence in terms of the tests that are generally used (for good or bad). This section describes the various test methods that are currently being used, discusses the positive and negative attributes of each test, and provides recommendations on what current standard tests are most appropriate for testing aggregate reactivity and preventive measures.

Table 4.1 lists the most common standard test methods that have been or are currently being used to assess aggregate reactivity, including relevant comments related to the positive and/or negative attributes of each test. A more detailed discussion and critique on the Accelerated Mortar Bar Test (AMBT) and Concrete Prism Test (CPT) are provided next as these are the two most commonly used ASR test methods and they are integrated into AASHTO PP65-11, *Standard Practice for Determining the Reactivity of Concrete Aggregates and Selecting Appropriate Measures for Preventing Deleterious Expansion in New Concrete Construction,* which is a newly-developed standard practice based on Thomas et al. (2008). Although past field performance and a petrographic evaluation are considered under AASHTO PP65-11, the AMBT and CPT are the primary approaches for evaluating the potential reactivity of a given aggregate, and modified versions of these methods may subsequently be used to select appropriate preventive measures, including supplementary cementing materials and lithium nitrate (used as a chemical admixture).

Table 4.1. Test Methods for Evaluating Aggregate Reactivity

Test Method	Comments
ASTM C 295: Standard Guide for Petrographic Examination of Aggregates for Concrete	• Useful evaluation to identify many (but not all) potentially reactive components in aggregates. • Reliability of examination depends on experience and skill of individual petrographer. • Results should not be used exclusively to accept or reject aggregate source – findings best used in conjunction with other laboratory tests (e.g., ASTM C 1260 and/or ASTM C 1293).
ASTM C 289: Standard Test Method for Potential Alkali-Silica Reactivity of Aggregates (Chemical Method)	• Aggregate test in which crushed aggregate is immersed in 1M NaOH solution for 24 hours – solution is then analyzed for amount of dissolved silica and alkalinity. • Poor reliability. • Problems with test include: – Other phases present in aggregate may affect dissolution of silica (Bérubé and Fournier 1992). – Test is overly severe, leading aggregates with good field performance to fail the test. – Some reactive phases may be lost during pretest processing.
ASTM C 227: Standard Test Method for Potential Alkali Reactivity of Cement-Aggregate Combinations (Mortar-Bar Method)	• Mortar bar test (aggregate/cement = 2.25), intended to study cement-aggregate combinations. • Specimens stored in high-humidity containers at 38°C. • Several reported problems with test, including excessive leaching of alkalis from specimens.
AASHTO T 303 (ASTM C 1260): Standard Test Method for Potential Alkali Reactivity of Aggregates (Mortar-Bar Method)	• Mortar bar test, originally designed to assess aggregate reactivity. • Bars are soaked in 1N NaOH solution for 14 days. • Accelerated test suitable as screening test, but because of severity of test, it should not be used, by itself, to reject a given aggregate. If aggregate is tested using both AASHTO T 303 and ASTM C 1293, the results of ASTM C 1293 should govern.
ASTM C 1293: Standard Test Method for Concrete Aggregates by Determination of Length Change of Concrete Due to Alkali-Silica Reaction	• Concrete prism test, generally regarded as best indicator of field performance, is conducted at high humidity (close to 100%) at 38°C. • Uses high-alkali cement (raised to 1.25% Na_2O_e), with a cement content of 420 kg/m^3. • Developed as aggregate test (using non-reactive fine aggregate to test reactivity of coarse aggregate, and vice-versa). • Test requires one year for completion. • Widely accepted test method, but long duration of test is major drawback.

AASHTO T 303 (ASTM C 1260) is based on the method developed by Oberholster and Davies (1986) at the National Building Research Institute in South Africa. The test, often referred to as the accelerated mortar bar test (AMBT), has been adopted by various countries and agencies, including the United States (ASTM and AASHTO) and Canada. The test entails casting mortar bars containing the subject aggregate (either coarse or fine), which is processed to a standard gradation. The mortar bars are then removed from their molds after 24 hours and placed in water

at room temperature. The temperature of the water is then raised to 80°C in an oven, and the mortar bars are stored in this condition for the next 24 hours. Upon removing the bars from the water, they are measured for initial length and then submersed in a 1N NaOH solution at 80°C, where they are then stored for 14 days. Length change measurements are made periodically during this storage period. The total expansion at the end of the 14-day soaking period typically is used in specifications, although the expansion limits specified by different agencies vary. For example, the expansion criteria established by ASTM and CSA are as follows:

ASTM C 1260 expansion criteria:	*CSA A23.2-25A expansion criteria:*
< 0.10% is considered innocuous	> 0.15% (0.10% for limestones) is considered reactive
0.10 to 0.20% is considered potentially reactive	< 0.15% (0.10% for limestones) is considered non-reactive
> 0.20% is considered reactive	

In ASTM C 1260, these expansion limits are proposed in an appendix to the standard test method and are not a mandatory part of the standard, whereas in CSA, the limits form part of a standard practice for assessing reactivity and evaluating the effectiveness of preventive measures (CSA A23.2-27A). AASHTO PP65-11 specifies a 0.10 percent expansion limit at 14 days; aggregates with expansions below this are considered non-reactive. However, AASHTO PP65-11 cautions that AASHTO T 303 is not as accurate as the concrete prism test (ASTM C 1293) for evaluating aggregate reactivity and that users who rely upon AASHTO T 303 are assuming some degree of risk in doing so.

AASHTO T 303 is recognized as a very severe test method because of the extreme test conditions, specifically the use of a highly alkaline storage solution and high temperature. Because of this inherent severity, the test has been shown to identify some aggregates as being reactive, even though they have performed well in concrete prism testing (ASTM C 1293) and in field applications (Bérubé and Fournier 1992). More recently, a number of coarse aggregates have been found to pass AASHTO T 303 but fail ASTM C 1293 and expand and crack in outdoor exposure blocks (Folliard et al. 2006). Thus, when both AASHTO T 303 and ASTM C 1293 data are available for a given aggregate, the ASTM C 1293 data should be relied upon for assessing aggregate reactivity.

ASTM C 1293, commonly referred to as the concrete prism test, is generally considered the most accurate and effective test in predicting the field performance of aggregates. In this test, concrete is cast with a cement content of 420 kg/m^3. The cement is required to have an equivalent alkali content between 0.8% and 1.0%, and additional alkalis (NaOH) then are added to the mixing water to obtain a total alkali content of 1.25% (by mass of cement), which equates to a total alkali content in the concrete mixture of 5.25 kg/m^3. Concrete prisms are cast, cured for 24 hours at 23°C, and then stored over water at 38°C. Expansion measurements are taken at regular intervals, and when testing plain concrete (without SCMs or chemical admixtures), the

test typically is run for one year (or as described later in this section, the test is conducted for two years when evaluating supplementary cementing materials or lithium-based admixtures).

In recent years, more countries and agencies have adopted the concrete prism test as a standard method. An expansion limit of 0.04% (at the end of the one- or two-year test) typically is specified, as this value has been reported to correlate well with cracking of test prisms. This expansion limit (0.04%) is referenced in the appendix to ASTM C 1293 and in AASHTO PP-65.

Although ASTM C 1293 is generally regarded as the most accurate test for assessing aggregate reactivity, it is not without its limitations or drawbacks. Thomas et al. (2006) reported that approximately 35 percent of the internal alkalis within concrete prisms may leach out during the course of a one-year test, with as much as 20 percent of the alkalis leaching out in the first 90 days. As such, higher alkali contents are needed to cause expansion in the CPT than what is actually needed in field concrete or exposure blocks. This was clearly illustrated in a study by Folliard et al. (2006), in which a series of three concrete mixtures with varying alkali loadings were cast using a highly-reactive sand from El Paso, Texas. Prisms from these mixtures were then tested using ASTM C 1293, and exposure blocks were cast and stored outdoors in Austin, Texas. Table 4.2 summarizes the CPT data at one year and the exposure block data after approximately 3.5 years of outdoor exposure. Of most significance is the mixture containing the low-alkali cement (Na_2O_e=0.52 percent), whose prisms easily passed the 0.04 percent expansion criteria after the one-year ASTM C 1293 test but whose exposure blocks expanded and cracked significantly. The reason for this discrepancy is the leaching from the concrete prisms which dropped the concrete below its alkali threshold, thereby suppressing expansion. The larger exposure blocks were less susceptible to leaching and hence exhibited significant expansion and cracking. Based on these results, and results from other researchers, it is not recommended that one use ASTM C 1293 to determine the alkali threshold for a given aggregate or aggregate-binder combination.

Table 4.2. Comparison of expansions in ASTM C 1293 compared to outdoor exposure blocks, highlighting importance of leaching in concrete prism test (Folliard et al. 2006)

Cement Alkalinity (% Na_2O_e)	ASTM C 1293 – Expansion at one year (%)	Outdoor Exposure Blocks – Expansion after approximately 3.5 years (%)
0.52	0.01	0.88
0.95	0.33	1.10
1.25	0.59	1.07

Another drawback to ASTM C 1293 is that the test duration (one year for aggregates, two years for preventive measures) is often deemed to be too long, especially in the United States. As such, very few agencies specify this test and very few practitioners run the test. The excessive duration is due to the fact that the storage conditions (storing real concrete prisms above water at

38°C) are nowhere near extreme as the AMBT, and the source of alkalis is finite and internal. Researchers have attempted to accelerate the test by increasing the storage temperature to 60°C, in hopes of generating relevant data in a few months instead of 1-2 years. However, as reported recently by Ideker et al. (2010), this accelerated version of the CPT yields significantly less expansion than the standard CPT for the following reasons:

- Additional leaching of alkalis at 60°C, compared to 38°C
- Additional prism drying observed at 60°C, compared to 38°C
- Changes in pore solution at 60°C (ettringite begins to dissolve, contributing sulfates to the pore solution at the expense of hydroxyl ions, thus lowering pore solution pH).
- The "non-reactive" sand used in conjunction with the subject coarse aggregate has major effect on expansions at 60°C.

4.3 ASR TEST METHODS FOR EVALUATING PREVENTIVE MEASURES

This section describes test methods for evaluating preventive measures, such as supplementary cementing materials and lithium compounds. Table 4.3 details the most common test methods used to evaluate preventive measures. Specific details are listed in the table for each test, but the primary focus of this section is on ASTM C 1567, which is a modified version of AASHTO T 303, and ASTM C 1293 as these two methods are the most commonly used test methods and the ones that are integrated into AASHTO PP65-11.

ASTM C 1567 is essentially identical to AASHTO T 303, with the one exception that it is aimed at evaluating the use of SCMs. As such, the test utilizes the same test specimens and storage conditions as AASHTO T 303 and is subject to the same issues and pitfalls as the standard version of the AMBT. First and foremost, if the aggregate to be evaluated does not produce accurate results when tested under AASHTO T 303, due to false negatives or false positives, it should be quite evident that the AMBT is not a suitable test for evaluating the specific aggregate in combination with SCMs to determine the dosage needed to control expansion. This is recognized in AASHTO PP65-11, as shown in Figure 4.1, where it is recommended that a reasonable correlation between AASHTO T 303 and ASTM C 1293 must first be developed for a given aggregate of interest; only upon obtaining such agreement between test methods can ASTM C 1567 then be used to evaluate preventive measures, such as SCMs, blended cements, and lithium admixtures.

Table 4.3. Test Methods for Evaluating Preventive Measures

| Test Method	Comments
ASTM C 441: Standard Test Method for Effectiveness of Mineral Admixtures or Ground Blast-Furnace Slag in Preventing Excessive Expansion of Concrete Due to the Alkali-Silica Reaction	• Mortar bar test, intended to assess effectiveness of SCMs in reducing ASR expansion. • Test uses high-alkali cement and Pyrex™ glass. • Test not very reliable because of the use of Pyrex glass, which is sensitive to test conditions and contains alkalis that may be released during the test. Test does not correlate well with data from concrete mixtures containing natural aggregates (Bérubé and Duchesne 1992).
ASTM C 1567: Standard Test Method for Determining the Potential Alkali-Silica Reactivity of Combinations of Cementitious Materials and Aggregate (Accelerated Mortar-Bar Method)	• Mortar bar test, originally designed to assess aggregate reactivity. • Bars are soaked in 1N NaOH solution for 14 days. • Should only be used for aggregates for which a reasonable correlation between AASHTO T 303 (ASTM C 1260) and ASTM C 1293 has been established (see AASHTO PP65-11 for details). • ASTM C 1567 can be modified as per AASHTO PP65-11 to evaluate lithium nitrate-based admixtures, although ASTM C 1293 is deemed to be a more effective test method for determining requisite lithium nitrate dosages for concrete.
ASTM C 1293: Standard Test Method for Concrete Aggregates by Determination of Length Change of Concrete Due to Alkali-Silica Reaction	• Modified version of concrete prism test, allows for testing SCMs, blended cements, and lithium admixtures. • Test duration is two years, with expansion limit of 0.04 percent.
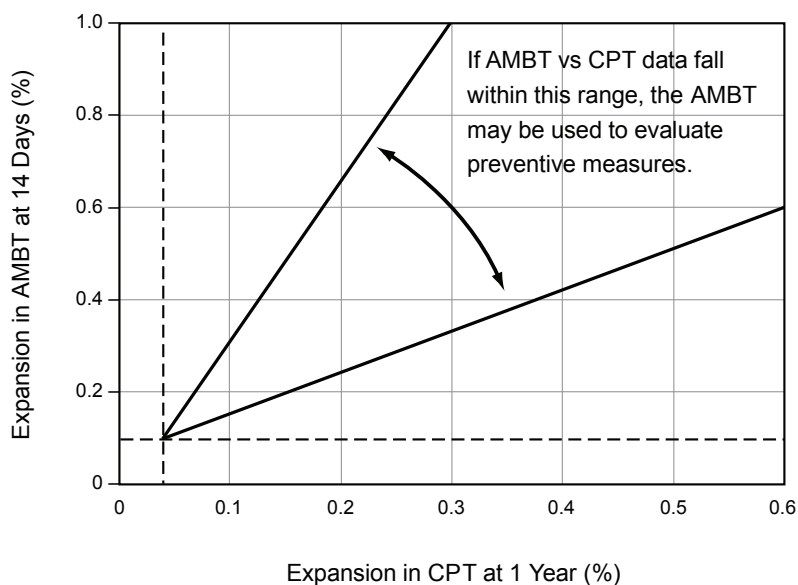
Figure 4.1. Comparison of AMBT and CPT Data for the Purpose of Determining Whether the AMBT is Suitable for Evaluating Preventive Measures with a Specific Aggregate (after AASHTO PP65-11)

Once it has been determined that ASTM C 1567 is an appropriate test method for evaluating a given aggregate, past work has shown that there is generally a good agreement between ASTM C

1567 (using a 0.10 percent expansion limit at 14 days) and ASTM C 1293 (using a 0.04 percent expansion limit at two years) when testing various SCMs (Thomas and Innis 1998), as shown in Figure 4.2. This figure illustrates that similar SCM dosages are needed in each of these tests to reduce the expansions below the aforementioned limits.
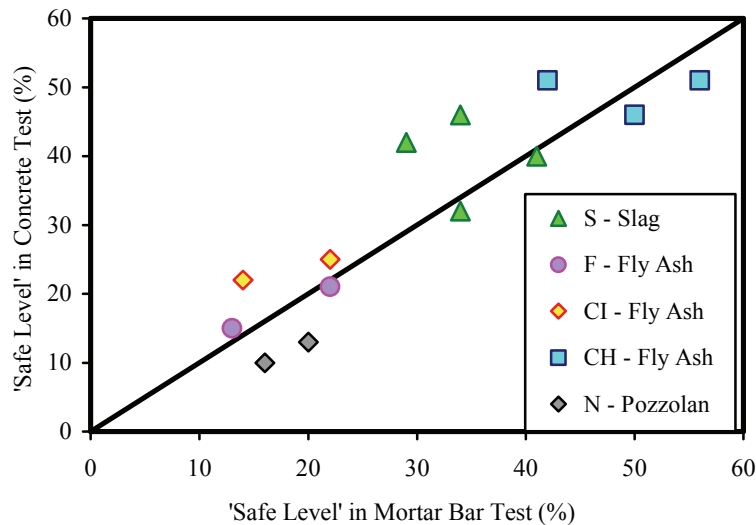
Figure 4.2. Comparison of SCM dosage needed to reduce expansions below test limits for ASTM C 1567 and ASTM C 1293 (after Thomas and Innis 1998)

When considering using lithium nitrate as an admixture in concrete, AASHTO PP65-11 recommends ASTM C 1293 as the most suitable and accurate test for determining the requisite dosage to control expansion below an expansion limit of 0.04 percent at two years. The AMBT, in its current form, is not suited for testing lithium compounds as any lithium present in the mortar bars would be overwhelmed by the sodium hydroxide in the soak solution. AASHTO PP65-11 provides guidance, based on Tremblay et al. (2008), for how to test lithium compounds in a modified version of the AMBT. This approach recognizes that certain aggregates cannot be tested using the AMBT in conjunction with lithium nitrate as erroneous results are generated that do not correlate with actual performance in concrete prisms or exposure blocks. As such, the procedure recommended in AASHTO PP65-11 identifies such aggregates and requires that these aggregates only be tested using ASTM C 1293. Aggregates that are suitable for being tested in combination with lithium nitrate in the modified AMBT are also identified and a testing regime is then specified that will arrive at an estimated dosage of lithium to control expansion, based on an extensive database that is correlated with CPT results. A broad overview of the approach specified in AASHTO PP65-11 is shown in Figure 4.3; readers are directed to Tremblay et al. (2008) for a more detailed description of this methodology for evaluating lithium nitrate in the modified AMBT (or in the CPT if it is deemed not possible to use the modified AMBT for a given aggregate).
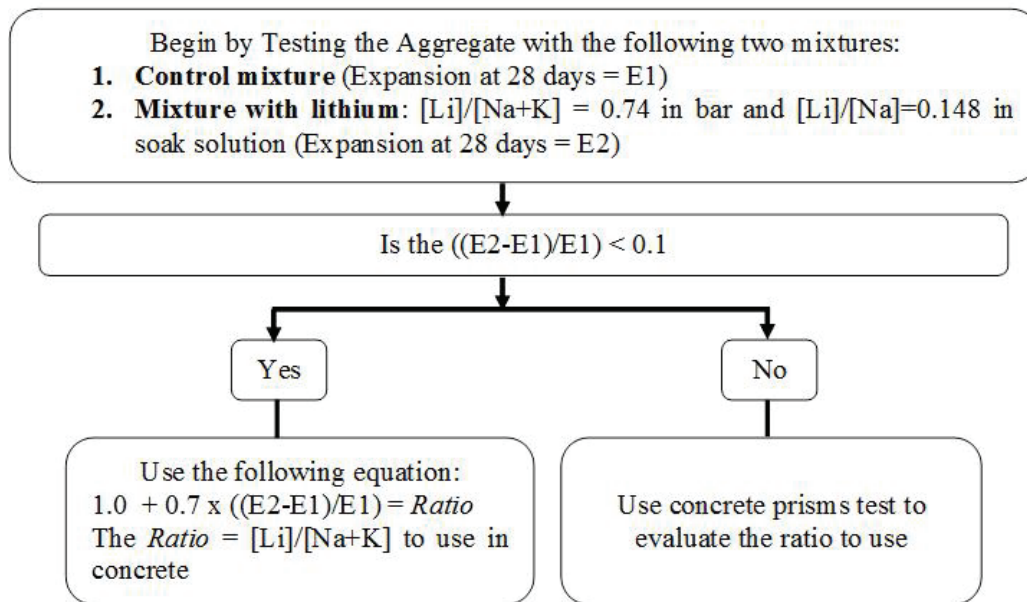
Figure 4.3. Approach to using the modified accelerated mortar bar test to determine the dosage of lithium nitrate (expressed as [Li]/ [Na+K]) to adequately suppress expansion in concrete (Tremblay et al. 2008)

4.4 ACR TEST METHODS FOR EVALUATING AGGREGATE REACTIVITY

ACR is a unique form of AAR that is highly expansive and damaging to concrete structures. It is also a reaction for which there are no known preventive measures. Measures that are effective in controlling ASR, such as the use of low-alkali cement, SCMs, and lithium compounds, are ineffective in suppressing ACR-induced expansion. As such, the only safe strategy for dealing with ACR is to identify susceptible aggregates and disallow their use in concrete. This section briefly describes how practitioners and researchers can identify ACR-susceptible aggregates.

AASHTO PP65-11 provides guidance, based primarily on Canadian experience, on how to evaluate aggregates that are potentially susceptible to ACR-induced expansion and cracking. If the aggregate being assessed is a quarried carbonate rock, the potential for alkali-carbonate reaction may be assessed on the basis of its chemical composition using the test method CSA A23.2-26A. This test involves the determination of the lime (CaO), magnesia (MgO), and alumina (Al_2O_3) content of the rock, and determining where the composition of the rock falls on a plot of CaO/MgO ratio versus the Al_2O_3 content, as shown in Figure 4.4.
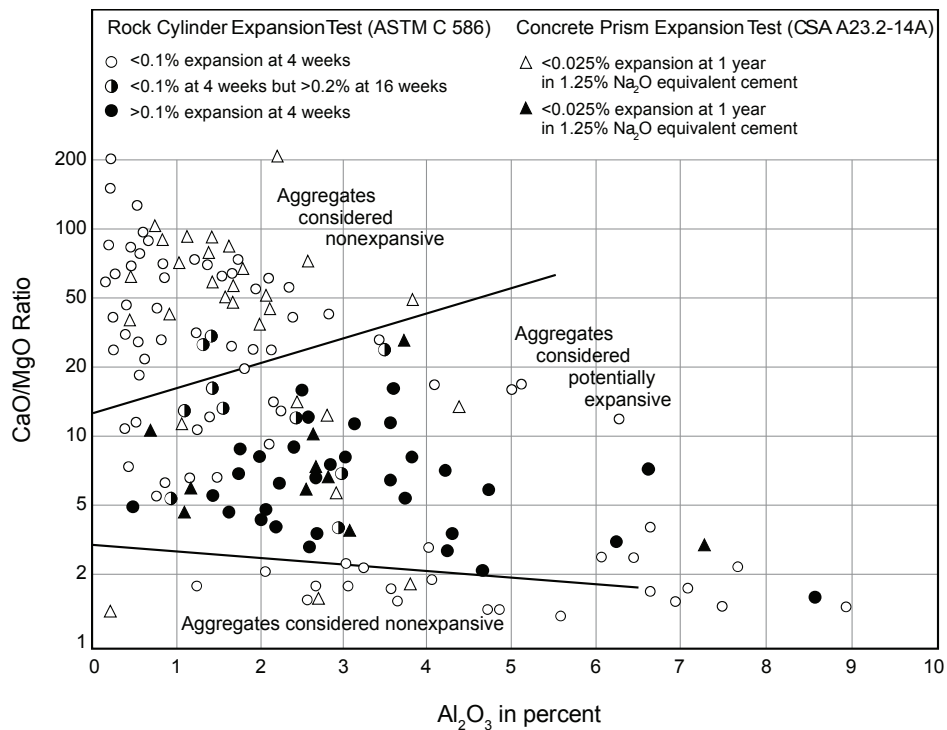
Figure 4.4. Using Chemical Composition as a Basis for Determining Potential Alkali-Carbonate Reactivity of Quarried Carbonates (from CSA A23.2-26A)

If the composition falls in one of the two ranges identified as "Aggregates considered non-expansive" in Figure 4.4, the aggregate is not potentially alkali-carbonate reactive, and it should be tested to determine the potential for alkali-silica reaction using the methods described in Section 4.2.

If the composition falls in the range of "aggregates considered to be potentially expansive" in Figure 4.4, the aggregate is potentially alkali-carbonate reactive and must be evaluated further. There are two options for further testing. One option is to test the aggregate in the concrete prism test, ASTM C 1293, to simultaneously determine the potential for alkali-carbonate and alkali-silica reactivity. After the test, the prisms are examined by petrography to determine the role played by the alkali-carbonate reaction. The second option is to test using the specific concrete prism test developed for ACR-susceptible aggregates, ASTM C 1105, which employs a reduced alkali loading to determine the potential for alkali-carbonate reaction only. If expansion of the specific aggregate-cement combination is equal to or greater than 0.025 percent at 6 months or 0.030 percent at 1 year, the aggregate shall be considered to be alkali-carbonate reactive and shall not be used in concrete. If the specific aggregate-cement combination passes the above expansion criteria, it is considered not to be alkali-carbonate reactive and can be tested in the same manner as aggregates with a composition that falls in the two ranges identified as "Aggregate considered non-expansive" in Figure 4.4.

4.5 SUMMARY

This section provided a brief summary of the available, standard test methods for evaluating aggregates susceptible to ASR or ACR. Recommendations were made on what tests are most appropriate for evaluating aggregate reactivity, as well as tests suitable for assessing preventive measures for ASR-susceptible aggregates.

REFERENCES

AASHTO PP65-11. 2011, "Standard Practice for Determining the Reactivity of Concrete Aggregates and Selecting Appropriate Measures for Preventing Deleterious Expansion in New Concrete Construction, American Association of State and Highway Transportation Officials." 24p.

Bérubé, M.A. and Duchesne, J. 1992. "Evaluation of Testing Methods Used for Assessing the Effectiveness of Mineral Admixtures in Suppressing Expansion Due to Alkali-Aggregate Reaction." Proceedings of the Fourth International Conference on Fly Ash, Silica Fume, Slag, and Natural Pozzolans in Concrete, (Ed. V.M. Malhotra), ACI SP132, Vol. 1, American Concrete Institute, Detroit, MI, 549–575.

Bérubé, M.A. and Fournier, B. 1992. "Accelerated Test Methods for Alkali-Aggregate Reactivity." Advances in Concrete Technology, (Ed. V.M. Malhotra), CANMET/EMR, Ottawa, Canada, 583–627.

Canadian Standards Association (CSA). 2000. "Standard Practice to Identify Degree of Alkali-Aggregate Reactivity of Aggregates and to Identify Measures to Avoid Deleterious Expansion in Concrete." CSA A23.2-27A, 2000a, Ontario, Canada.

Folliard, K.J., Barborak, R., Drimalas, T., Du, L., Garber, S., Ideker, J., Ley, T., Williams, S., Juenger, M., Thomas, M.D.A. and Fournier, B. 2006. "Preventing ASR/DEF in New Concrete: Final Report." The University of Texas at Austin, Center for Transportation Research (CTR), CTR 4085-5.

Oberholster, R.E. and Davies, G. 1986. "An Accelerated Method for Testing the Potential Reactivity of Siliceous Aggregates." Cement and Concrete Research, Vol. 16, 181–189.

Thomas, M.D.A. and Innis, F.A. 1998. "Effect of Slag on Expansion Due to Alkali-Aggregate Reaction in Concrete." ACI Materials Journal 95 (Dec. 1998), 716–24.

Thomas, M.D.A., Fournier, B., Folliard, K., Ideker, J. and Shehata, M. 2006. "Test methods for evaluating preventive measures for controlling expansion due to alkali-silica reaction in concrete." Cement and Concrete Research, 36(10), 1842-1856.

Thomas, M.D.A., Fournier, B. and Folliard, K.J. 2008. "Report on determining the reactivity of concrete aggregates and selecting appropriate measures for preventing deleterious expansion in new concrete construction." Federal Highways Administration, FHWA-HIF-09-001, National Research Council, Washington, D.C. A34.

Tremblay, C., Bérubé, M.A., Fournier, B., Thomas, M.D.A., Folliard, K.J. and Nkinamubanzi, P.C. 2008. "Use of the Accelerated Mortar Bar Test to Evaluate the Effectiveness of $LiNO_3$ Against Alkali-Silica Reaction -- Part 2: Comparison with Results from the Concrete Prism Test." Journal of ASTM International (JAI), Online ISSN: 1546-962X, Published Online: 17 September 2008, 21p.

5 – Prevention of Alkali-Silica Reaction

5.1 INTRODUCTION

This section discusses various strategies for preventing alkali-silica reaction (ASR) in concrete, including avoiding reactive aggregates, controlling the alkali content of the concrete, using supplementary cementing materials, and the use of lithium-based compounds. The section does not discuss methods for preventing alkali-carbonate reaction (ACR). Strategies for controlling ASR are not generally effective for preventing ACR and, consequently, alkali-carbonate reactive rocks must not be used in concrete.

5.2 PREVENTIVE MEASURES – OPTIONS

In Section 2 it was shown that there are three basic requirements for damaging ASR to occur in concrete; these are:

- A sufficient quantity of reactive silica (within aggregates)

- A sufficient concentration of alkali (primarily from portland cement)

- Sufficient moisture

Elimination of any one of these requirements will prevent the occurrence of damaging alkali-silica reaction. Exclusion of water from civil engineering structures is not practical in most cases and so the most obvious options for preventing expansion due to ASR are the following:

1. Avoid the use of reactive aggregates

2. Minimize the amount of alkalis from the portland cement

Two other options are as follows:

3. Use of supplementary cementing materials (SCM)

4. Use of lithium-based compounds

In fact, the use of SCM is a form of Option 2 as these materials consume alkalis and reduce their availability for reaction with the aggregate. Lithium compounds work in a different manner by changing the nature of the reaction product. Both of these mechanisms are discussed later in the chapter.

5.3 USE OF NON-REACTIVE AGGREGATES

Using non-reactive, or more correctly, non-deleteriously-reactive, aggregates is certainly a viable method of preventing ASR-induced damage if such aggregates are available. Competent and thorough testing is required to ensure that aggregate sources are non-reactive and this can be achieved through a combination of petrographic examination, expansion testing of mortar (ASTM C 1260) or concrete (ASTM C 1293), and field performance. Such testing should be performed on a regular basis to ensure that the composition (and reactivity) does not change within a pit or a quarry. If the aggregate sources can be confirmed to be truly non-reactive, no further precaution is required to prevent ASR.

There are a number of reasons why the option of using non-reactive aggregates is not always feasible; these include:

- Non-reactive aggregates are not available locally and the cost of shipping non-reactive materials from other locations is prohibitive

- Reactive aggregates that are otherwise wholly suitable for concrete are readily and abundantly available at lower cost and reduced environmental impact compared with non-reactive materials

- Lack of confidence in test results (or testing laboratories) or test results are ambiguous (e.g. different test methods do not agree)

- All of the locally-available materials fail the very aggressive accelerated test (regardless of true reactivity)

In such cases, it is necessary to adopt one of the other options for preventing damaging ASR.

Furthermore, some instances warrant extra caution even when using aggregates believed to be non-reactive; examples include the design of critical structures (e.g. prestigious structures or those with an extended design life) and the construction of structural elements exposed to a very aggressive environment (e.g. structures exposed to seawater or deicing salts, which may provide an external source of alkalis).

5.4 LIMITING THE ALKALI CONTENT OF THE CONCRETE

The role of alkalis and the relationship between the alkali content of the concrete and expansion is discussed in Section 2.4.2.2. If the alkali content of the concrete is below some threshold value deleterious expansion is prevented. This threshold value is likely to vary between aggregates of different reactivity but, unfortunately, there is currently no suitable test method for determining the threshold. A number of specifications have employed a maximum concrete alkali content as

an option to control expansion in concrete containing reactive aggregates. Nixon and Sims (1992) reported that maximum permissible alkali contents between 2.5 and 4.5 kg/m^3 (4.2 to 7.5 lb/yd^3) Na$_2$Oe have been specified by various countries and agencies, with the allowable alkali content sometimes varying depending on aggregate reactivity. In 2000, the alkali limit used as a preventive measure in the Canadian Standard (CSA A23.2-27A) was modified from the single value of 3.0 kg/m^3 (5 lb/yd^3) Na$_2$Oe specified in the 1994 edition to a sliding scale from 1.8 to 3.0 kg/m^3 (3 to 5 lb/yd^3) Na$_2$Oe; the limits are as follows:

Table 5.1. Alkali Limits Specified in CSA A23.2-27A

Level of prevention required	Alkali limit (Na$_2$Oe)	
	kg/m^3	lb/yd^3
Mild	3.0	5.0
Moderate	2.4	4.0
Strong	1.8	3.0
Exceptional	1.8 + SCM	3.0 + SCM

These same alkali limits were adopted in AASHTO PP65; this is discussed further in Section 6. The selection of the appropriate level depends on the risk and consequences of ASR as determined by the reactivity of the aggregate and the nature, exposure conditions, and design life of the structure. An example where a "strong" prevention level is required would be when a moderately reactive aggregate (expansion in concrete prism tests is greater than 0.04% but less than 0.12%) is used in an exposed structure with a design life of greater than 75 years (e.g. a bridge or a dam). If a highly reactive rock is used in the same type of structure an "exceptional" level of prevention would be required. If a moderately reactive aggregate was used in an exposed structure with a design life of less than 75 years (e.g. sidewalk or pavement) only a moderate level of prevention is required. An example of "mild" prevention is that required for unexposed massive elements incorporating a moderately-reactive aggregate in a dry environment (e.g. interior elements of buildings).

Aggregates that are used in concrete with an alkali content below the threshold for expansion may cause damaging expansion if the alkali content of the concrete increases at some locations during the service life of the structure. This may occur through alkali concentration caused by drying gradients, alkali release from aggregates, or the ingress of alkalis from external sources, such as deicing salts or seawater (Nixon et al. 1987). Stark (1978) reported increases in soluble alkali from 1.1 to 3.6 kg/m^3 Na$_2$Oe close to the surface of some highway structures. Migration of alkalis due to moisture, temperature, and electrical gradients has also been demonstrated by laboratory studies.

A number of studies have demonstrated that many aggregates contain alkalis that may be leached out into the concrete pore solution, thereby increasing the risk of alkali-aggregate reaction. Stark and Bhatty (1986) reported that, in extreme circumstances, some aggregates release alkalis equivalent to 10% of the portland cement content.

Supplementary cementing materials (SCM), such as fly ash, silica fume, slag, and natural pozzolans may also contain significant quantities of alkali. However, these alkalis generally do not need to be included in the calculation of the concrete alkali content as SCM tend to reduce the alkalis that are available for reaction with the aggregate; this is discussed in the next section.

5.5 USE OF SUPPLEMENTARY CEMENTING MATERIALS

5.5.1 Effect of SCM on Expansion of Concrete

One of the most efficient means of controlling ASR in concrete containing reactive aggregates is the appropriate use of supplementary cementing materials (SCM). Such materials include pozzolans (e.g. fly ash, silica fume, calcined clay, or shale) and ground-granulated blast furnace slag. The potential use of pozzolans to control ASR dates back as far as the discovery of ASR, having been reported in the first major publication on the phenomenon (Stanton 1940).

Almost any SCM can be used to control ASR provided it is used at a sufficient level of replacement. The amount required varies widely depending on, among other things, the following:

- The nature of the SCM (especially mineralogical and chemical composition); more SCM is required as its silica content decreases or as its alkali and calcium content increase,

- The nature of the reactive aggregate; generally, the more reactive the aggregate, the higher the level of SCM required,

- The availability of alkali within the concrete (i.e., from the portland cement and other sources); the amount of SCM required increases with the amount of available alkali,

- The exposure conditions of the concrete; concrete exposed to external sources of alkali may require higher levels of SCM.

Figure 5.1 shows the (conceptual) relationship between the (long-term) expansion of concrete and the level of replacement for different SCMs. Generally, as the level of replacement increases with a particular SCM, the expansion decreases and eventually reaches an acceptable level at which no damage occurs. SCMs that are very high in reactive silica, such as silica fume and metakaolin, tend to be very efficient in controlling expansion and are only required at relatively low levels of replacement (e.g. 10 to 15%). On the other hand, SCMs with lesser amounts of

silica, such as Class C fly ash and slag, need to be used at higher levels of replacement (e.g. ≥ 40%). Some SCMs (e.g. some Class C fly ashes) may produce a pessimum effect by increasing the amount of expansion (compared to concrete without SCM) if they are used at low levels of replacement, but decreasing expansion at higher levels of replacement.
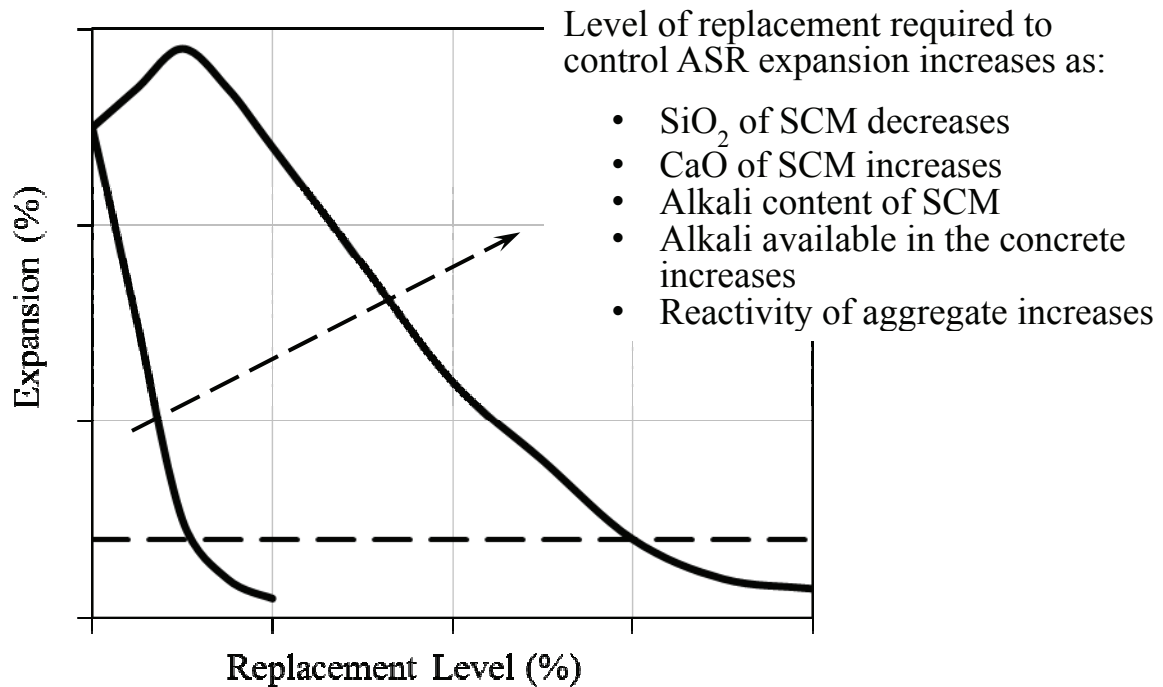
Figure 5.1. Expansion versus SCM Content - Conceptual Relationship (Thomas 2011)

Figure 5.2 shows experimental data to support this concept. In this figure, the two-year expansion of concrete prisms (tested in accordance with ASTM C 1293) produced with high-alkali cement and a highly-reactive siliceous limestone (Spratt) is plotted against the level of replacement for a range of SCMs.
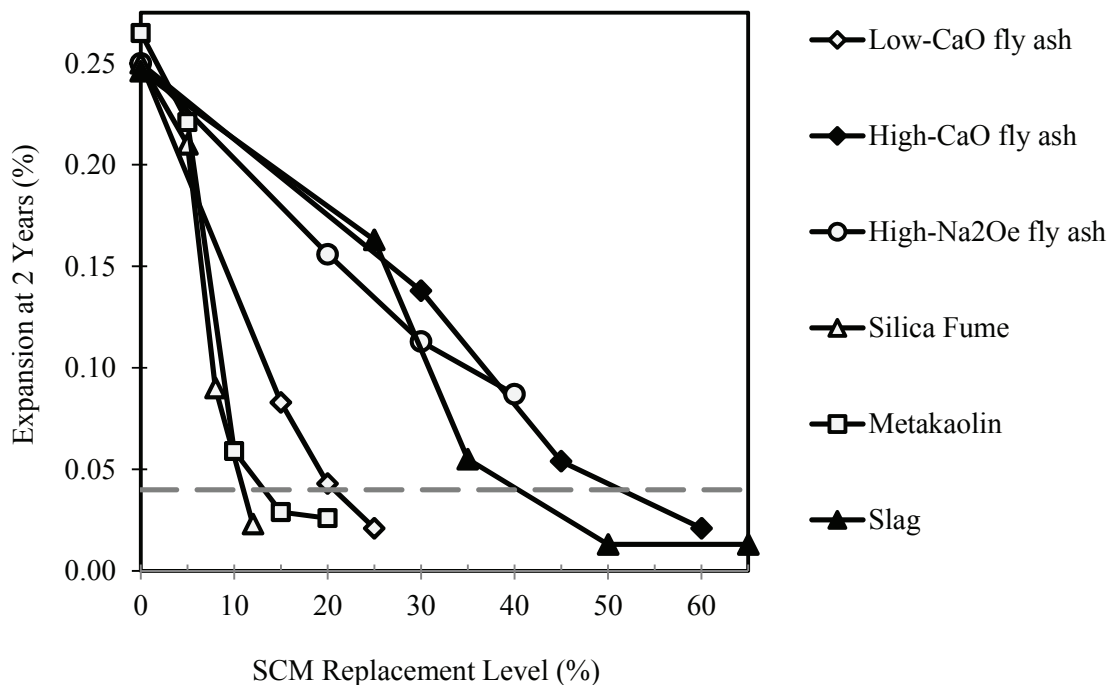
Figure 5.2. Expansion of Concrete with Various SCMs (Thomas 2011)

The amount of SCM required to prevent damaging ASR expansion generally falls in the ranges below (modified from Thomas and Folliard 2007):

Table 5.2. Required Levels of SCM

Type of SCM	Level required (%)
Low-calcium fly ash (< 8% CaO)	20 to 30
Moderate-calcium fly ash (8 - 20% CaO)	25 to 35
High-calcium fly ash (> 20% CaO)	40 to 60
Silica fume	8 to 15
Slag	35 to 65
Metakaolin (calcined kaolin clay)	10 to 20

However, the level of SCM required may exceed these values under exceptional conditions (e.g. extremely reactive aggregate, high alkali availability in concrete – including alkali contribution from aggregates, concrete exposed to high concentrations of alkali in service, or critical structure with extended service life).

The efficacy of fly ash with regards to controlling ASR varies widely with the different types of fly ash commercially available in North America. Class F fly ashes with low calcium contents are generally much more efficient than Class C fly ashes with high calcium contents. The role of calcium in fly ash is shown in Figure 5.3 which shows the 2-year expansion of concrete containing high-alkali cement, highly-reactive siliceous limestone (Spratt) aggregate, and 25% fly ash for fly ashes of varying compositions. Most fly ashes with calcium contents below 20% CaO are effective in controlling expansion, and the level of expansion increases as the calcium content of the fly ash increases above 20% CaO. Note that fly ashes with high alkali contents ($\geq$ 5% Na_2Oe) are not effective at a 25% level of replacement regardless of calcium content. Figure 5.4 shows that high-calcium fly ashes are effective at controlling ASR expansion provided that they are used at higher levels of replacement (typically more than 40%).
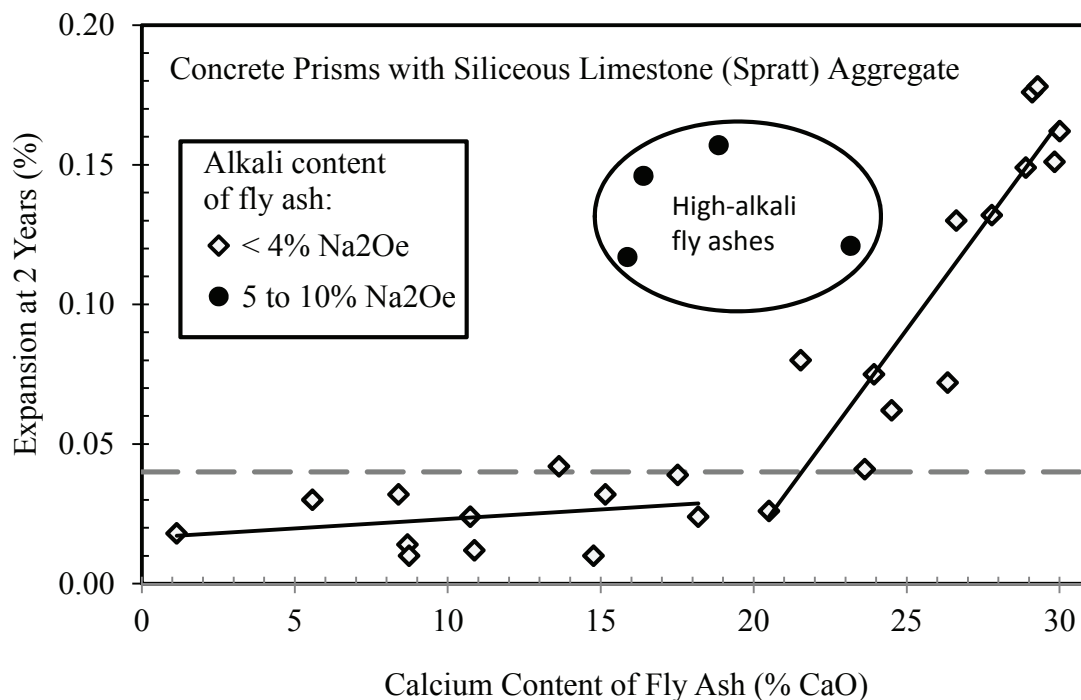
Figure 5.3. Expansion of Concrete with Fly Ashes of Different Calcium Content (Thomas 2011)
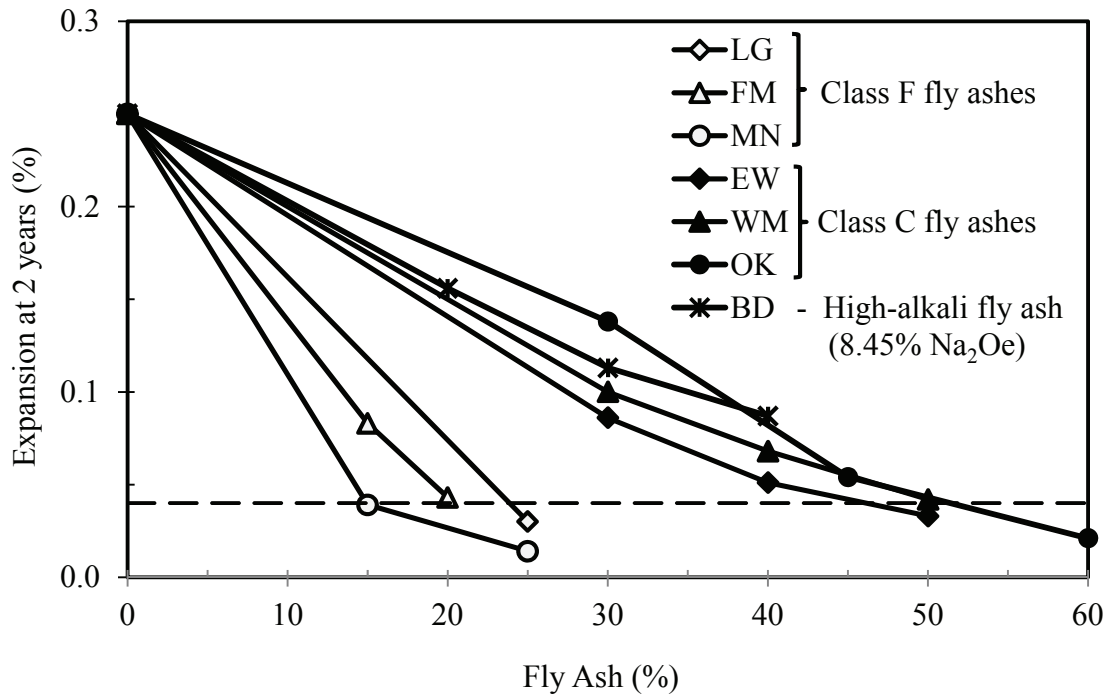
Figure 5.4. Effect of Fly Ash Replacement Level on Expansion of Concrete (Thomas 2011)

Ternary blends containing two SCMs can be very effective in controlling ASR. Figures 5.5 and 5.6 show the impact of combinations of, respectively, silica fume and fly ash, and silica fume and slag.
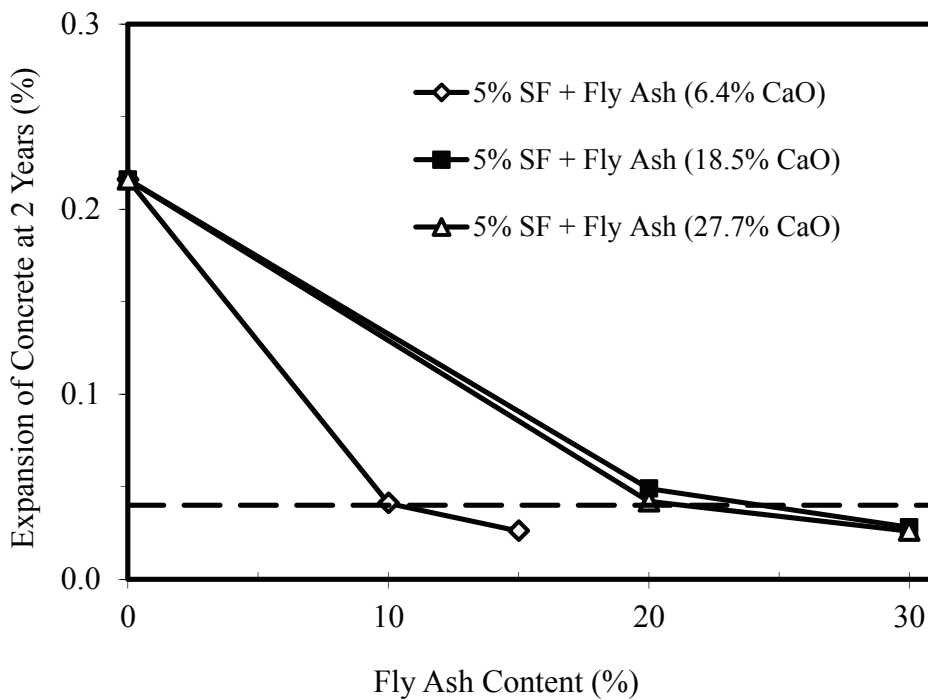
Figure 5.5. Expansion of Concrete with Blends of Silica Fume and Fly Ash
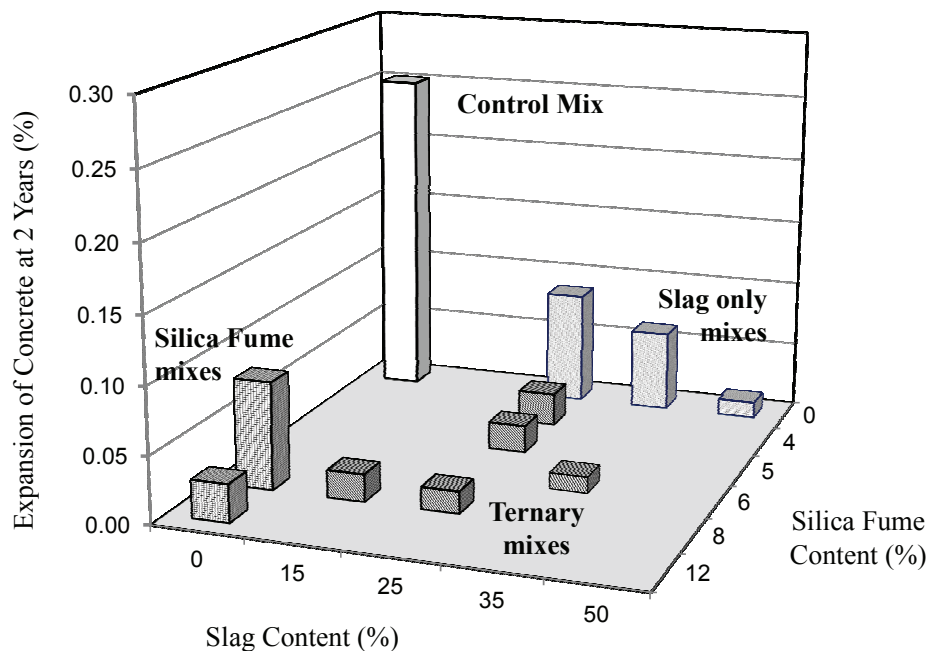
Figure 5.6. Expansion of Concrete with Blends of Silica Fume and Slag

5.5.2 How SCMs Work to Control ASR

Supplementary cementing materials control expansion due to ASR by reducing the availability of alkalis in the concrete. The alkalis, sodium and potassium, in concrete are partitioned between the solid phases (i.e., bound by the hydrates) and the liquid phases (i.e., pore solution) of the concrete. Only the alkali hydroxides in the pore solution can attack the reactive components of the concrete. The binding capacity of the hydrates is, to a large extent, a function of the calcium-silica ratio of the C-S-H that forms. C-S-H with a lower Ca/Si ratio has an increased alkali-binding capacity (Bhatty and Greening 1978) since the surface charge becomes less positive as the calcium content decreases and this attracts more positive cations (Na^+ and K^+) from the surrounding pore solution (Glasser and Marr 1985).

Figure 5.7 shows the evolution of the pore solution alkalinity (OH^- ion concentration) extracted from cement pastes (w/cm = 0.50) produced with high-alkali cement and various SCMs. The OH^- concentration of the control sample is very high approaching 1 Mole/L which equates to a pH of 14. Most SCMs are effective in reducing the OH^- concentration although the extent of the reduction is a function of the SCM type and replacement level, and age. The effect of SCMs on the pore solution composition depends on the amount of alkali (Na_2Oe) they contribute, and the degree to which the presence of the material changes the Ca/Si ratio of the hydrates that form.
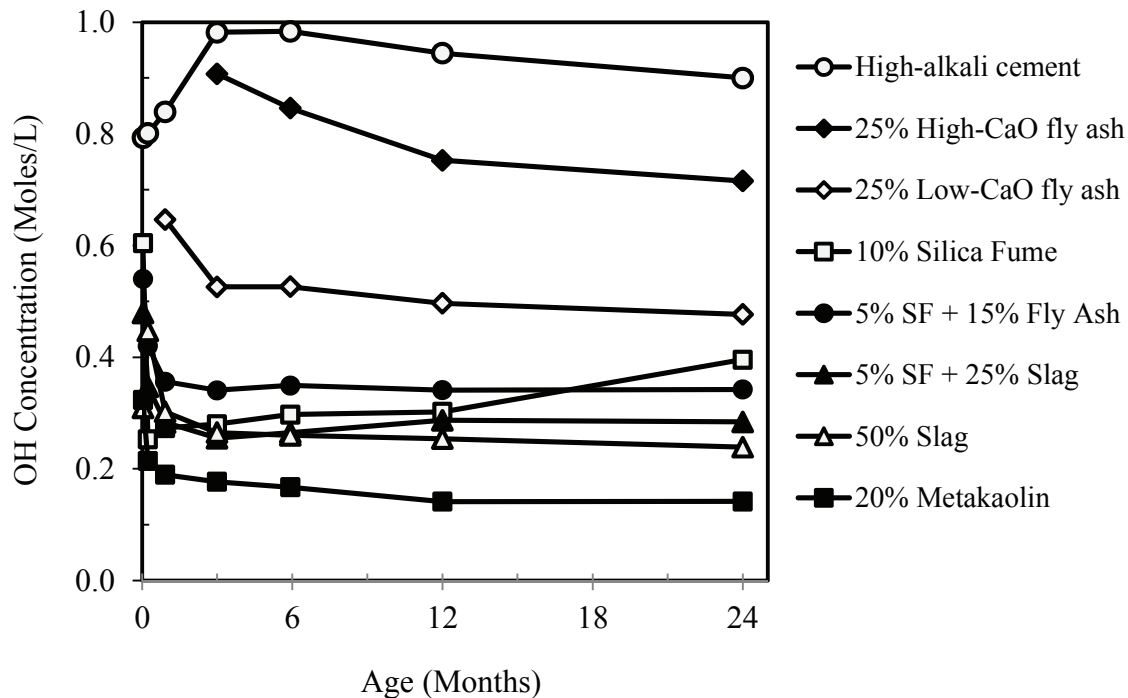
Figure 5.7. Effect of SCM on the Evolution of the Pore Solution of Pastes (Thomas 2011)

Thus SCMs with low levels of Na_2Oe and CaO, and high levels of reactive SiO_2, will contribute little alkali to the system, but significantly increase the alkali-binding capacity of the hydrates (by reducing the Ca/Si ratio). The effect of binder composition on the OH^- concentration of the pore solution was shown in Figure 2.5 and discussed in Section 2.4.2.2. It is evident from this figure that the hydroxyl (and alkali) ion concentration is a function of the alkali, calcium and silica content of the binder including the SCM component.

The reactive silica in the SCM reacts with the alkali hydroxides in the pore solution in the same way as the reactive silica in the aggregate. The initial reaction product is an alkali-silica gel containing small amounts of calcium and, over time, calcium exchanges for alkali in the gel and C–S–H forms with a relatively low Ca/Si ratio compared to that formed in portland cement paste. The only substantial differences between this pozzolanic reaction (between the silica in SCM and the alkalis in solution) and the alkali-silica reaction is the timescale over which the reactions occur and the absence of any detectable expansion due to the pozzolanic reaction. The lack of expansion can perhaps be explained by the fact that pozzolans are very-finely divided materials and the alkali-silica gel that forms and is subsequently converted to C–S–H is distributed throughout the cement paste, whereas the presence of reactive aggregate particles leads to the accumulation of larger deposits of alkali-silica gel in discrete locations that can become sites of expansion.

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The importance of the size and distribution of the reactive silica can be demonstrated in two ways. Firstly, it has been known since the formative work of Stanton (1940) that if a reactive aggregate is ground to sufficient fineness expansion is eliminated. Figure 5.8 shows unpublished data from the author's laboratory relating to the effect of ground Vycor glass (sub-100 μm) on the expansion of mortar bars containing sand-sized Vycor glass as a reactive aggregate. The sand-sized Vycor glass behaves as a reactive aggregate causing expansion of the mortar with portland cement as the only binder. However, the same material, when ground, behaves like a pozzolan when it is used to replace 20% of the Portland cement and prevents expansion. Secondly, if finely-divided pozzolans agglomerate and form sand-sized particles, these particles will behave like reactive aggregates and may result in expansion and cracking. This effect has been observed with agglomerated silica fume both in the field and in the laboratory. Figure 5.9 (photo courtesy of Maria Juenger) shows a back-scattered electron image of a mortar bar containing agglomerated silica fume after storage in 1 M NaOH solution at 80°C for 14 days (Maas et al. 2007). The mortar, which contained non-reactive sand, expanded during test and the expansion was attributed to the reaction of the agglomerated silica fume.
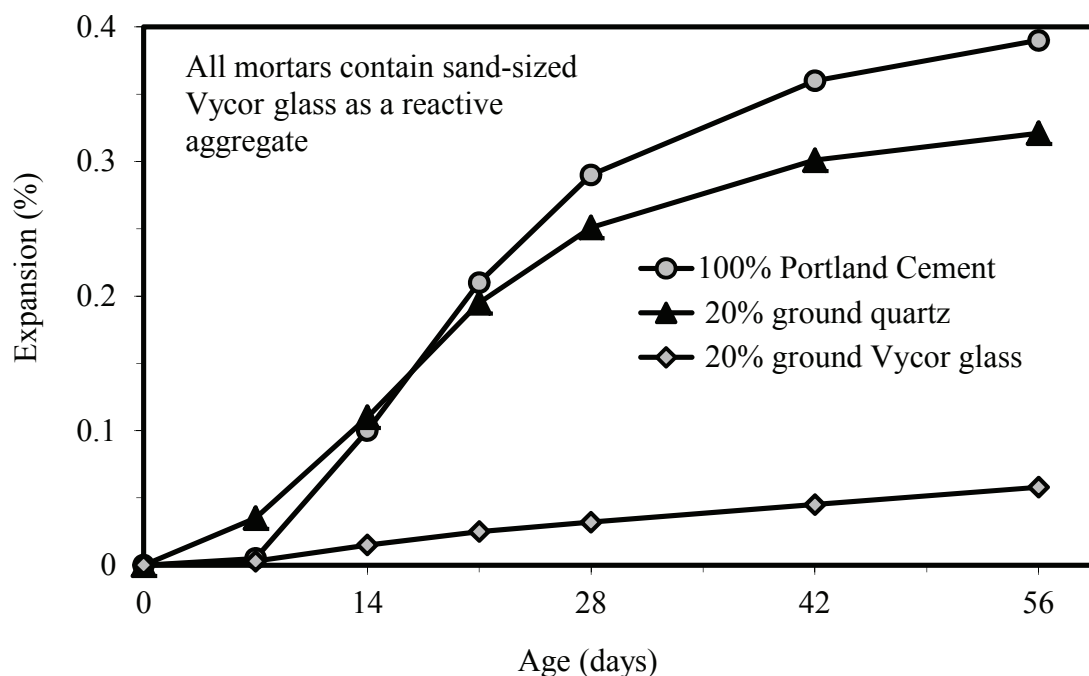
Figure 5.8. Role of Particle Size on the Behaviour of Vycor Glass (Thomas 2011)
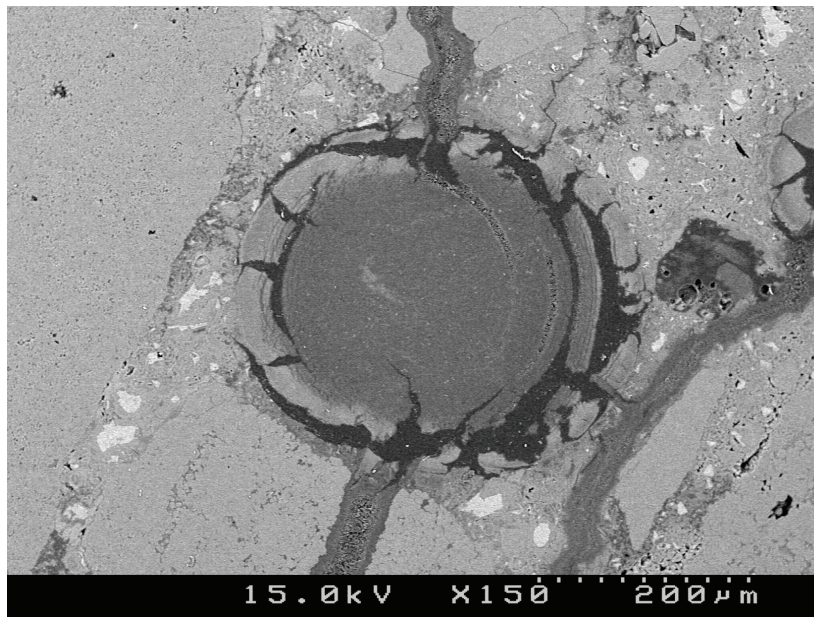
Figure 5.9. Agglomerated silica fume particle behaving as a reactive aggregate and source of ASR expansion (Maas et al. 2007)

5.5.3 Tests Methods for Evaluating the Efficacy of SCM

Either the concrete prism test (ASTM C 1293) or the accelerated mortar bar test (ASTM C 1567) is recommended for evaluating the effect of SCM on ASR or for determining the minimum level of a particular SCM (or combination of SCMs) required to control expansion with a specific reactive aggregate. A detailed discussion of AAR test methods is provided in Section 4 of this reference manual.

It is recommended that the following criteria are used when evaluating SCM-reactive aggregate combinations (Thomas et al. 2006; 2007):

Table 5.3. Criteria to Evaluate SCM-Reactive Aggregate Combinations

Test Method	Expansion Limit
Concrete prism test (ASTM C 1293)	$\leq 0.040\%$ at 2 years
Accelerated mortar bar test (ASTM C 1567)	$\leq 0.10\%$ at 14 days

5.6 USE OF LITHIUM

The ability of lithium to control deleterious expansion due to alkali-silica reaction (ASR) in mortar and concrete was first demonstrated by McCoy and Caldwell (1951). They showed that, out of more than 100 chemical compounds tested, various salts of lithium (e.g. LiCl, Li_2CO_3, LiF, Li_2SiO_3, $LiNO_3$, and Li_2SO_4) were the most promising and could virtually eliminate the expansion of mortar containing Pyrex glass provided they were used at sufficient levels of replacement. Since then, there have been numerous studies which corroborate this earlier discovery (Feng et al. 2005).

It is somewhat paradoxical that lithium compounds are effective suppressants of ASR as lithium is an alkali metal like sodium and potassium. The precise mechanism by which lithium controls ASR is not known, although many theories have been put forward (Feng et al. 2005). The simplest and most commonly used explanation is that lithium salts will react with reactive silica in a similar way to sodium and potassium salts, but the reaction product is an insoluble lithium-silicate with little propensity to imbibe water and swell. The lithium silicate forms around reactive aggregate particles and protects the underlying reactive silica from "attack" by alkali hydroxides.

The initial work of McCoy and Caldwell (1951) showed that the amount of lithium required to control expansion was a function of the availability of other alkalis (Na + K) in the system and they concluded that the expansion of mortar bars containing reactive Pyrex glass could be effectively suppressed provided that the lithium-to-sodium-plus-potassium molar ratio was greater than 0.74, i.e. [Li]/[Na+K] > 0.74. Since then numerous workers have demonstrated a similar relationship between the amount of lithium required and the amount of alkali available, but the minimum value of [Li]/[Na+K] has been shown to vary depending on a number of issues such as the form of lithium, nature of reactive aggregate and, perhaps, the method of test used (Feng et al. 2005).

Although most lithium compounds have a beneficial effect, lithium nitrate ($LiNO_3$) is considered to be the most efficient form for suppressing ASR (Stokes et al. 1997). Lithium nitrate solution is commercially available from a number of companies in North America being marketed as an "ASR-suppressing admixture". Currently the product is sold as a 30% solution of $LiNO_3$. To achieve a lithium-to-sodium-plus-potassium molar ratio of [Li]/[Na+K] = 0.74 using a 30% solution of $LiNO_3$ requires a dose of 4.6 gallons of $LiNO_3$ solution per 1 kg of Na_2Oe (0.55 gallons of solution per 1 lb Na_2Oe). This has been referred to as the "standard dose" of lithium nitrate solution.

Recent research (Tremblay et al. 2007) has highlighted the influence of aggregate type on the amount of lithium required to suppress expansion due to ASR. Figure 5.10 shows the 2-year

expansion of concrete prisms with 12 different reactive aggregates and 1 non-reactive aggregate (NF), and various levels of lithium (standard dose is [Li]/[Na+K] = 0.74). For 6 of the 12 aggregates 75% to 100% of the standard dose was sufficient to control expansion ($\leq$ 0.040% at 2 years). For 3 of the aggregates 125% to 150% of the standard dose was required; however, for the remaining 3 aggregates expansion could not be controlled even at 150% of the standard dose.
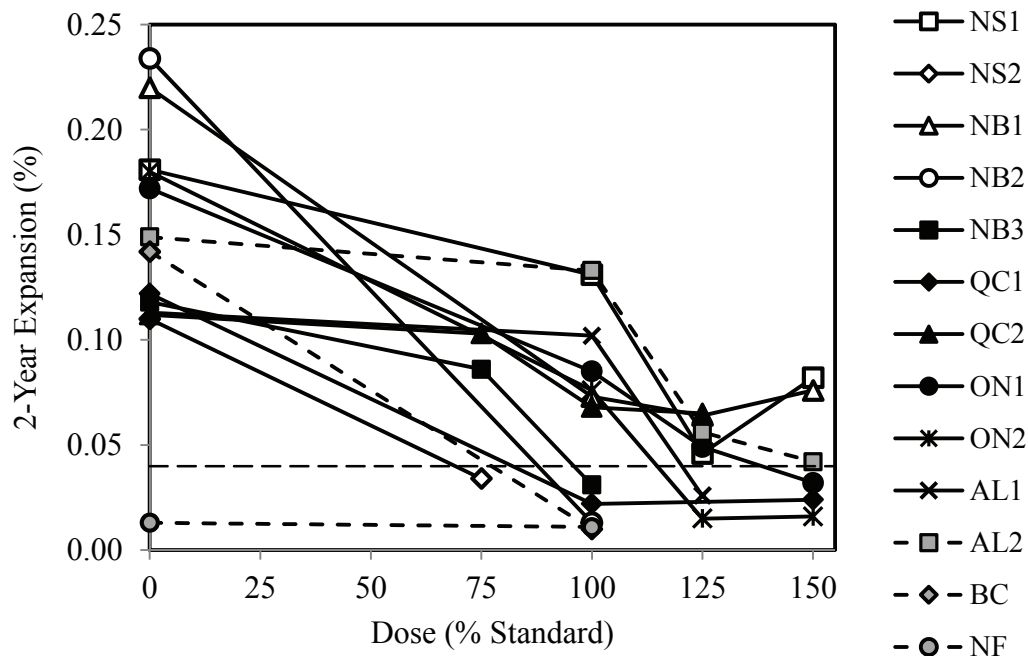
Figure 5.10. Effect of Lithium Dose on the Expansion of Concrete with Different Reactive Aggregates (Tremblay et al. 2007)

As the effectiveness of lithium appears to be extremely aggregate dependent, it is not possible to prescribe a single dose for controlling ASR, and the minimum dose must be determined by testing lithium with the specific reactive aggregate being considered for use. At this point in time there is no consensus regarding the appropriateness of accelerated tests for determining the correct lithium dose and it is recommended that the concrete prism test is used for this purpose.

REFERENCES

Bhatty, M.S.Y. and Greening, N.R. 1978. "Interaction of alkalis with hydrating and hydrated calcium silicates." Proceedings of the Fourth International Conference on the Effects of Alkalis in Cement and Concrete, Purdue, 87-112.

Canadian Standards Association (CSA). 2000. "Standard Practice to Identify Degree of Alkali-Reactivity of Aggregates and to Identify Measures to Avoid Deleterious Expansion in Concrete." CSA A23.2-27A, 2000a, Ontario, Canada.

Feng, X., Thomas, M.D.A., Bremner, T.W., Balcom, B.J. and Folliard, K.J. 2005. "Studies on lithium salts to mitigate ASR-induced expansion in new concrete: a critical review." Cement and Concrete Research, Vol. 35, 1789-1796.

Glasser, F.P. and Marr, J. 1985 "The alkali binding potential of OPC and blended cements." Il Cemento, 82, 85-94.

Maas, A.J., Ideker, J.H. and Juenger, M.C.G. 2007. "Alkali silica reactivity of agglomerated silica fume." Cement and Concrete Research, Vol. 37. 166–174.

McCoy, W.J. and Caldwell, A.G. 1951. "New approach to inhibiting alkali-aggregate expansion." Journal of the American Concrete Institute, 22 (9), 693-706.

Nixon, P.J. and Sims, I. 1992. "Alkali Aggregate Reaction ± Accelerated Tests Interim Report and Summary of National Specifications." Proceedings of the 9th International Conference on Alkali-Aggregate Reaction in Concrete, Vol. 2. The Concrete Society, Slough, 731-738.

Nixon, P.J., Canham, I., and Page, C.L. 1987. "Aspects of the Pore Solution Chemistry of Blended Cements Related to the Control of Alkali-Silica Reaction." Cement and Concrete Research, 17(5), 839-844.

Stanton, T.E. 1940. "Expansion of concrete through reaction between cement and aggregate." Proceedings of the American Society of Civil Engineers, 66(10), 1781-1811.

Stark, D. 1978. "Alkali-silica Reactivity in the Rocky Mountain Region." Proceedings of the 4th International Conference on Effects of Alkalis in Cement and Concrete, CE-MAT-1-78, Purdue University, W. Lafayette, Indiana, 235-243.

Stark, D. and Bhatty, M.S.Y. 1986. "Alkali-silica Reactivity: Effect of Alkali in Aggregate on Expansion." Alkalis in Concrete, ASTM STP 930, American Society for Testing and Materials, Philadelphia, 16-30.

Stokes, D.B., Wang, H.H. and Diamond, S. 1997. "A lithium-based admixture for ASR control that does not increase the pore solution pH." Proceedings of the 5th CANMET/ACI International Conference on Superplasticizers and Other Chemical Admixtures in Concrete, (Ed. V.M. Malhotra), ACI SP-173, American Concrete Institute, Detroit, 855-867.

Thomas, M.D.A. 2011. "The effect of supplementary cementing materials on alkali-silica reaction: A review." Cement and Concrete Research, Vol. 41, 1224-1231.

Thomas, M.D.A., Fournier, B., Folliard, K., Ideker, J. and Shehata, M. 2006. "Test methods for evaluating preventive measures for controlling expansion due to alkali-silica reaction in concrete." Cement and Concrete Research, 36(10), 1842-1856.

Thomas, M.D.A., Fournier, B., Folliard, K.J., Shehata, M., Ideker, J. and Rogers, C.A. 2007. "Performance limits for evaluating supplementary cementing materials using the accelerated mortar bar test." ACI Materials Journal, 104(2), 115-122.

Tremblay, C., Berube, M-A., Fournier, B., Thomas, M.D.A. and Folliard, K.F. 2007. "Effectiveness of lithium-based products in concrete made with Canadian reactive aggregates." ACI Materials Journal, 104(2), 195-205.

6 – Alkali-Aggregate Reactions: Specifications

6.1 INTRODUCTION

This section discusses specifications for alkali-aggregate reaction focusing on AASHTO PP65-11 "Standard Practice for Determining the Reactivity of Concrete Aggregates and Selecting Appropriate Measures for Preventing Deleterious Expansion in New Concrete Construction." Requirements within ASTM and ACI are also discussed.

Essentially there are two approaches to specifying preventive measures for controlling alkali-aggregate reaction. In the prescriptive approach, reactive aggregates are permitted provided they are used with prescribed amounts of ameliorative materials such as a minimum level of fly ash or slag (for example). In a performance-based approach, a certain combination of materials (reactive aggregates and preventive measures) are permitted provided they are tested and shown to meet certain performance requirements such as meeting the expansion limits of mortar-bar or concrete-prism expansion tests. AASHTO PP65-11 has both prescriptive and performance options.

6.2 EXISTING SPECIFICATIONS IN USA

6.2.1 ASTM

ASTM C 33 *Standard Specification for Concrete Aggregates* states that "concrete aggregates for use in concrete … shall not contain any materials that are deleteriously reactive with the alkalis in the cement in an amount sufficient to cause excessive expansion of mortar or concrete" unless one of the following three conditions is met:

- the aggregate is "used with a cement containing less than 0.60 % alkalis calculated as sodium oxide equivalent ($Na_2Oe = Na_2O + 0.658$ x K_2O),"

- there is a satisfactory service record evaluation,

- the aggregate is used "with the addition of a material that has been shown to prevent harmful expansion due to the alkali-aggregate reaction."

Within the ASTM concrete standards there are number of laboratory test methods that can be used to determine the potential for aggregates to be deleteriously reactive or the effectiveness of different preventive measures against ASR; these are:

- C 295 *Petrographic Examination of Aggregates*

- C 227 *Standard Test Method for Potential Alkali Reactivity of Cement-Aggregate Combinations (Mortar-Bar Method)*

- C 289 *Standard Test Method for Potential Alkali-Silica Reactivity of Aggregates (Chemical Method)*

- C 441 *Standard Test Method for Effectiveness of Pozzolans or Ground Blast-Furnace Slag in Preventing Excessive Expansion of Concrete Due to the Alkali-Silica Reaction*

- C 1105 *Standard Test Method for Length Change of Concrete Due to Alkali-Carbonate Rock Reaction*

- C 1260 *Standard Test Method for Potential Alkali Reactivity of Aggregates (Mortar-Bar Method)*

- C 1293 *Standard Test Method for Determination of Length Change of Concrete Due to Alkali-Silica Reaction*

- C 1567 *Standard Test Method for Determining the Potential Alkali-Silica Reactivity of Combinations of Cementitious Materials and Aggregate (Accelerated Mortar-Bar Method.*

Appendix X1 of ASTM C 33 provides some limited guidance on interpretation of the results from these tests and measures for mitigating alkali-silica reaction[2]. If the aggregate produces little or no expansion in C 1260 or C 1293, or has a satisfactory service history (with similar cementitious materials) no mitigation is necessary. On the other hand, if the aggregates are considered to be deleteriously alkali-silica reactive Appendix X1 (section 4.3) recommends one of the following preventive measures:

- Use of ASTM C 150 portland cement meeting the low-alkali option ($\leq 0.60\%$ Na$_2$Oe)

- Use of ASTM C 595 blended cement meeting the optional mortar-bar-expansion requirement

- Use of hydraulic cement meeting the ASTM C 1157 performance specification including Option R - *Low Reactivity with Alkali-Reactive Aggregates*.

- Use of pozzolans or slag meeting the optional requirements of the relevant material specifications (C 618 for fly ash and natural pozzolans, C 1240 for silica fume, and C 989 for slag) for preventing excessive expansion due to ASR.

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[2] ASTM C 33 Appendix X1 also provides guidance on mitigation of alkali-carbonate reaction. The guidance is limited to "avoiding reactive carbonate rocks; selective quarrying; diluting reactive rock to less than 20 % of the aggregate in the concrete; use of smaller maximum size; and the use of very low alkali cement."

The optional requirements for blended cements, C 1157 cements, pozzolans, and slag all make use of C 441 (Pyrex) mortar-bar method for demonstrating the effectiveness in controlling expansion due to ASR. However, each material specification has different performance requirements and these may be summarized as follows:

ASTM Specification	Expansion Limit of Mortars with Pyrex Glass
C 595 Blended cements	Maximum expansion of 0.020% at 14 days **and** 0.060% at 56 days
C 1157 Hydraulic cements (performance-based specification)	Maximum expansion of 0.020% at 14 days
C 618 Fly ash and natural pozzolans	Expansion of fly ash mortars not greater than expansion of control mortars with low-alkali cement ($\leq$ 0.60% Na_2Oe) at 14 days
C 989 Slag	Expansion of the job cement plus slag should not expand by more than 0.02% at 14 days **or**, if the job cement is not known, the slag should reduce the 14-day expansion of a mixture with high-alkali cement by at least 75% when compared with a mix with high-alkali cement on its own.
C 1240 Silica fume	Blend of high-alkali cement plus silica fume must reduce expansion by at least 80% compared with high-alkali cement alone

A joint C09/C01 Task Group recently recognized that numerous ASTM specifications provide requirements and guidance for avoiding deleterious ASR expansion for individual concrete materials (e.g. cementitious materials, aggregates, and SCMs) while, together, these specifications do not provide coherent guidance for preventing deleterious expansion in concrete; also, the specifications utilize standard test methods that are in many cases considered to be unreliable (Struble 2010). The Task Group thus recommended that clear and consistent guidance/specification for the prevention of ASR be developed that addresses performance at the concrete level and includes requirements for aggregates. A joint ASTM technical subcommittee, C01/C09-50, was then formed to develop new global requirements for the Risk Management of Alkali-Aggregate Reactions. This new subcommittee started its activities in 2010.

6.2.2 ACI

ACI 301 *Specifications for Structural Concrete* (ACI 301-10) states that potentially reactive aggregates may be used either with low-alkali cement ($\leq$ 0.60% Na_2Oe) or supplementary

cementing materials "in an amount shown to be effective in preventing harmful expansion due to alkali-aggregate reaction in accordance with ASTM C 441" and defers to ASTM C 33.

ACI 318 *Building Code Requirements for Structural Concrete* (ACI 318-08) requires aggregates to meet the requirements of ASTM C 33 but makes no reference to alkali-aggregate reactions (ASR or ACR) or the use of deleteriously-reactive aggregates in concrete.

Guidance for preventing damage due to AAR is provided in ACI 201 *Guide to Durable Concrete* (ACI 201.2R-08) and ACI 221 *Report on Alkali-Aggregate Reactivity* (ACI 221.1R-98).

6.2.3 State Specifications

Many state transportation agencies have developed specifications for minimizing the risk of deleterious alkali-aggregate reactions in concrete. These are too numerous to discuss here but examples of these specifications are available for review at the following link:
http://www.fhwa.dot.gov/pavement/concrete/asr/reference.cfm?main_category=Specifications

6.3 AASHTO PP65-11

A version of AASHTO PP65-11 *Standard Practice for Determining the Reactivity of Concrete Aggregates and Selecting Appropriate Measures for Preventing Deleterious Expansion in New Concrete Construction* was first published in 2010. As the title suggests, these guidelines can be broken down into two steps as follows:

- Evaluating aggregate reactivity (for alkali-silica and alkali-carbonate reactive aggregates)

- Selecting preventive measures (for alkali-silica reactive aggregates only)

6.3.1 Evaluating Aggregate Reactivity

Aggregate reactivity is evaluated by considering one or more of the following options: (i) field performance history, (ii) petrographic assessment, (iii) chemical composition (for quarried carbonates), (iv) data from accelerated mortar bar tests (AASHTO T 303), and (v) data from concrete prism tests (ASTM C 1293). Aggregates may be classified as non-deleteriously-reactive and can be used without preventive measures provided they meet one of the following conditions:

- There is an established history of satisfactory field performance (minimum 15 years) with the aggregate used in the same exposure conditions with similar cementitious materials.

- Petrographic examination of the aggregate by a skilled and experienced petrographer indicates that potentially deleterious minerals are not present in sufficient concentration to cause deleterious reaction and expansion.

- Expansion of mortar bars (AASHTO T 303; ASTM C 1260) is not greater than 0.10% after 14 days immersion in 1 M NaOH at 176ºF (80ºC).

- Expansion of concrete prisms (ASTM C 1293) is not greater than 0.040% at 1 year.

Although AASHTO PP65-11 allows aggregates to be accepted solely based on field history and/or petrography it strongly recommends expansion testing using either the accelerated mortar bar test or, preferably, the concrete prism test.

Prior to expansion testing, quarried carbonates must be subjected to chemical analysis to determine whether there is a risk of alkali-carbonate reaction (ACR). If the results of the analysis indicate a potential for ACR the aggregate must be tested by the concrete prism test as the accelerated mortar bar test is not capable of detecting alkali-carbonate reactive aggregate. If the concrete prisms expand during test, the prisms must be examined to determine whether the expansion was due to ACR, ASR, or a combination of ASR and ACR. If ACR is considered to have contributed to the expansion the aggregate must be rejected for use in concrete.

If, after testing, the aggregate is identified to be alkali-silica reactive, AASHTO PP65-11 requires that it is either rejected for use or used together with appropriate preventive measures.

6.3.2 Selecting Preventive Measures

PP65-11 provides two approaches for selecting preventive measures; these being: (i) a performance approach based on laboratory testing, and (ii) a prescriptive approach based on a consideration of the reactivity of the aggregate, type and size of structure, exposure conditions, and the composition of cementitious materials being used. The two test methods used for the performance-based approach are the accelerated mortar bar test for evaluating combinations of cementing materials and aggregates (ASTM C 1567) and the concrete prism test (ASTM C 1293), with preference given to the latter test. The rationale behind the use of these two tests and the preference for the concrete prism test is presented in FHWA-HIF-09-001 (Thomas et al. 2008) and in Thomas et al. (2006).

The options for preventive measures included in the prescriptive approach of PP65-10 are to (i) control the alkali content of the concrete to a maximum allowable level, (ii) use a minimum level of supplementary cementing material (SCM) or combination of SCMs, or (iii) use a combination of these two options (that is controlling the alkali content of the concrete and using SCM). The precise level of alkali permitted or SCM required depends on a number of factors including the

aggregate reactivity, type and size of structure, exposure conditions, and even the composition of the cement and SCM being used.

A brief summary of the prescriptive and performance options are described here.

6.3.2.1 Prescriptive Approach for Selecting Preventive Measures

The prescriptive approach of AASHTO PP65 can be summarized in the following steps:

Step 1. Determine aggregate reactivity class: The aggregate is tested in either the accelerated mortar bar test (AMBT), AASHTO T 303, or, preferably, the concrete prism test (CPT), ASTM C 1293. The criteria in Table 6.1 are used to classify the aggregate reactivity, which can range from "R0 – non-reactive" to "R3 – very highly reactive".

Step 2. Determine level of ASR risk: Based on the aggregate-reactivity class determined in Step 1 (Table 6.1) and the size and exposure conditions of the concrete under construction, the level of ASR risk is determined using the criteria in Table 6.2. The risk may range from Level 1 (lowest or negligible risk) to Level 6 (highest risk).

Step 3. Determine level of prevention: Based on the level of ASR risk determined in Step 2 (Table 6.2) and the classification of the structure[3], the level of prevention required is determined using Table 6.3. The level of prevention required may range from Level V (no measures necessary) to Level ZZ (extreme preventive measures necessary).

Step 4. Identification of preventive measures: Based on the level of prevention required that was determined in Step 3 (Table 6.3), a number of options are presented as acceptable measures for preventing ASR; these are:

Option 1 – limiting the alkali content of the concrete (Table 6.5)

Option 2 – using supplementary cementing materials, SCM[4] (Table 6.6)

Option 3 – limiting the alkali content of the concrete and using SCM (Table 6.8)

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[3] Table 6.4 is intended to provide guidance in selecting the Class of Structure.

[4] For Option 2, the minimum amount of SCM determined from Table 6 may be adjusted based on the alkali level of the portland cement using Table 6.7.

Table 6.1. Classification of Aggregate Reactivity

Aggregate-Reactivity Class	Description of Aggregate Reactivity	One-Year Expansion in CPT (%)	14-Day Expansion in AMBT (%)
R0	Non-reactive	≤ 0.04	≤ 0.10
R1	Moderately reactive	$> 0.04, \leq 0.12$	$> 0.10, \leq 0.30$
R2	Highly reactive	$> 0.12, \leq 0.24$	$> 0.30, \leq 0.45$
R3	Very highly reactive	> 0.24	> 0.45

Table 6.2. Determining the Level of ASR Risk

	Aggregate-Reactivity Class			
Size and exposure conditions	R0	R1	R2	R3
Non-massive[1] concrete in a dry[2] environment	Level 1	Level 1	Level 2	Level 3
Massive[1] elements in a dry[2] environment	Level 1	Level 2	Level 3	Level 4
All concrete exposed to humid air, buried or immersed	Level 1	Level 3	Level 4	Level 5
All concrete exposed to alkalis in service[3]	Level 1	Level 4	Level 5	Level 6

[1] A massive element has a least dimension > 3 ft (0.9 m).

[2] A dry environment corresponds to an average ambient relative humidity lower than 60%, normally only found in buildings.

[3] Examples of structures exposed to alkalis (sodium and potassium) in service include marine structures exposed to seawater and highway structures exposed to deicing salts (e.g. NaCl) or anti-icing salts (e.g. potassium acetate, potassium formate, sodium acetate, sodium formate, etc.).

Table 6.3. Determining the Level of Prevention

Level of ASR Risk (Table 6.4)	Classification of Structure (Table 4)			
	S1	S2	S3	S4
Risk Level 1	V	V	V	V
Risk Level 2	V	V	W	X
Risk Level 3	V	W	X	Y
Risk Level 4	W	X	Y	Z
Risk Level 5	X	Y	Z	ZZ
Risk Level 6	Y	Z	ZZ	††

†† It is not permitted to construct a Class S4 structure (see Table 6.4) when the risk of ASR is Level 6. Measures must be taken to reduce the level of risk in these circumstances.

The level of prevention V, W, X, Y, Z and ZZ are used in Tables 6.5 to 6.8.

Table 6.4. Structures Classified on the Basis of the Severity of the Consequences Should ASR[1] Occur (Modified for Highway Structures from RILEM TC 191-ARP)

Class	Consequences of ASR	Acceptability of ASR	Examples[2]
S1	Safety, economic, or environmental consequences small or negligible	Some deterioration from ASR may be tolerated	▪ Non-load-bearing elements inside buildings ▪ Temporary structures (e.g. < 5 years)
S2	Some safety, economic, or environmental consequences if major deterioration	Moderate risk of ASR is acceptable	▪ Sidewalks, curbs, and gutters ▪ Service-life < 40 years
S3	Significant safety, economic, or environmental consequences if minor damage	Minor risk of ASR acceptable	▪ Pavements ▪ Culverts ▪ Highway barriers ▪ Rural, low-volume bridges ▪ Large numbers of precast elements where economic costs of replacement are severe ▪ Service life normally 40 to 75 years
S4	Serious safety, economic, or environmental consequences if minor damage	ASR cannot be tolerated	▪ Major bridges ▪ Tunnels ▪ Critical elements that are very difficult to inspect or repair ▪ Service life normally > 75 years

[1] This table does not consider the consequences of damage due to ACR. This practice does not permit the use of alkali-carbonate reactive aggregates.

[2] The types of structures listed under each Class are meant to serve as examples. Some owners may decide to use their own classification system. For example, sidewalks or curbs and gutters may be placed in the Class S3.

Table 6.5. Maximum Alkali Contents in Portland Cement Concrete to Provide Various Levels of Prevention

Prevention Level	Maximum Alkali Content of Concrete (Na$_2$Oe)	
	lb/yd^3	kg/m^3
V	No limit	
W	5.0	3.0
X	4.0	2.4
Y	3.0	1.8
Z[1]	Table 6.8	
ZZ[1]		

[1] SCMs must be used in Prevention levels Z and ZZ.

Table 6.6. Minimum Levels of SCM to Provide Various Levels of Prevention

Type of SCM[1]	Alkali level of SCM (% Na_2Oe)	Minimum Replacement Level[3] (% by mass of cementitious material)				
		Level W	Level X	Level Y	Level Z	Level ZZ
Fly ash (CaO $\leq$ 18%)	≤ 3.0	15	20	25	35	Table 6.7
	$> 3.0, \leq 4.5$	20	25	30	40	
Slag	≤ 1.0	25	35	50	65	
Silica Fume[2] (SiO$_2$ $\geq$ 85%)	≤ 1.0	1.2 x LBA or 2.0 x KGA	1.5 x LBA or 2.5 x KGA	1.8 x LBA or 3.0 x KGA	2.4 x LBA or 4.0 x KGA	

[1] The SCM may be added directly to the concrete mixer or it may be a component of a blended cement. SCMs should meet the requirements of AASHTO M 295, M 302 or M 307. Blended cements should meet the requirements of AASHTO M 240 or ASTM C 1157.

[2] The minimum level of silica fume (as a percentage of cementitious material) is calculated on the basis of the alkali (Na_2Oe) content of the concrete contributed by the portland cement and expressed in either units of lb/yd^3 (LBA in Table 6.6) or kg/m^3 (KGA in Table 6.6). LBA is calculated by multiplying the cement content of the concrete in lb/yd^3 by the alkali content of the cement divided by 100. For example, for a concrete containing 500 lb/yd^3 of cement with an alkali content of 0.81% Na_2Oe, the value of LBA = 500 x 0.81/100 = 4.05 lb/yd^3. For this concrete, the minimum replacement level of silica fume for Level Y is 1.8 x 4.05 = 8.1%. KGA is calculated by multiplying the cement content of the concrete in kg/m^3 by the alkali content of the cement divided by 100. For example, for a concrete containing 300 kg/m^3 of cement with an alkali content of 0.91% Na_2Oe, the value of KGA = 300 x 0.91/100 = 2.73 kg/m^3. For this concrete, the minimum replacement level of silica fume for Level X is 2.5 x 2.73 = 6.8%. Regardless of the calculated value, the minimum level of silica fume shall not be less than 7% when it is the only method of prevention.

[3] The use of high levels of SCM in concrete may increase the risk of problems due to deicer salt scaling if the concrete is not properly proportioned, finished, and cured.

Table 6.7. Adjusting the Minimum Level of SCM Based on the Alkali Content in the Portland Cement

Cement Alkalis (% Na$_2$Oe)	Level of SCM
≤ 0.70	Reduce the minimum amount of SCM given in Table 6.6 by one prevention level[1]
> 0.70, ≤ 1.00	Use the minimum levels of SCM given in Table 6.6
> 1.00, ≤1.25	Increase the minimum amount of SCM given in Table 6.6 by one prevention level
> 1.25	No guidance is given

[1] The replacement levels should not be below those given in Table 6.6 for prevention level W, regardless of the alkali content of the portland cement.

Table 6.8. Using SCM and Limiting the Alkali Content of the Concrete to Provide Exceptional Levels of Prevention

Prevention Level	SCM as Sole Prevention	Limiting Concrete Alkali Content Plus SCM	
	Minimum SCM Level	Maximum Alkali Content, lb/yd^3 (kg/m^3)	Minimum SCM Level
Z	SCM level shown for Level Z in Table 6.6	3.0 (1.8)	SCM level shown for Level Y in Table 6.6
ZZ	Not permitted	3.0 (1.8)	SCM level shown for Level Z in Table 6.6

The prescriptive approach does not allow the option for using lithium compounds as a preventive measure. Research has shown that the efficacy of lithium compounds in controlling expansion due to ASR is highly influenced by the nature of the reactive aggregate (Tremblay et al. 2007). Currently, it is not possible to prescribe the required lithium dose based on aggregate reactivity or mineralogy and, consequently, lithium compounds must be tested using the prescriptive approach to determine the minimum dose required with a specific aggregate.

6.3.2.2 Performance Approach for Selecting Preventive Measures

AASHTO PP65-11 recommends that the concrete prism test (ASTM C 1293) is used to evaluate the efficacy of supplementary cementing materials or lithium-based compounds for controlling alkali-silica reaction. Preferably tests should be conducted at a range of SCM levels or lithium doses to determine the "safe" level. The preventive measure is considered to be effective provided the expansion of concrete prisms is not greater than 0.040% after 2 years.

If there is insufficient time to conduct concrete tests, ASTM C 1567 (modified accelerated mortar-bar test for evaluating combinations of reactive aggregate and SCM) may be used to evaluate SCMs. The SCM is deemed to be effective in controlling ASR expansion with the aggregate under test provided the expansion of mortar bars is not greater than 0.10% after 14 days immersion in 1 M NaOH at 176ºF (80ºC).

AASHTO PP65-11 also includes a modified version of the accelerated mortar bar test for evaluating lithium-based compounds. However, there are few data available for calibrating this test method and it is strongly recommended that the concrete prism test be used to evaluate lithium-aggregate combinations.

REFERENCES

Struble, L. 2010. "Opening remarks." Workshop on specifications on alkali-aggregate reaction. Sponsored by ASTM Committees C09 on Concrete and Concrete Aggregates and C01 on Cement, June 6, 2010, St. Louis, Missouri, USA. (available on ASTM website).

Thomas, M.D.A., Fournier, B., Folliard, K., Ideker, J. and Shehata, M. 2006. "Test methods for evaluating preventive measures for controlling expansion due to alkali-silica reaction in concrete." Cement and Concrete Research, Vol. 36, No. 10, 1842-1856.

Thomas, M.D.A., Fournier, B. and Folliard, K.J. 2008. "Report on determining the reactivity of concrete aggregates and selecting appropriate measures for preventing deleterious expansion in new concrete construction." Federal Highways Administration, FHWA-HIF-09-001, National Research Council, Washington, D.C. A34.

Tremblay, C., Bérubé, M-A., Fournier, B., Thomas, M.D.A. and Folliard, K.F. 2007. "Effectiveness of lithium-based products in concrete made with Canadian reactive aggregates." ACI Materials Journal, Vol. 104, No. 2, 195-205.

7 – Diagnosis and Prognosis of Alkali-silica Reaction in Concrete Structures

7.1 INTRODUCTION

Alkali-aggregate reaction (AAR) is one of the many factors that might be fully/partly responsible for the deterioration and premature loss in serviceability of concrete infrastructure. Comparative field and laboratory investigations can be carried out to confirm that AAR is the main cause or a significant contributor to the deterioration observed, thus aiming at selecting appropriate remedial actions. Such investigations will likely include one or several of the following steps:

- the field inspection of the structure(s) under study to identify the presence/distribution and severity of the defects affecting the various structural elements (especially those features diagnostic of AAR), as well as the exposure conditions to which the structure is subjected;
- the in-situ monitoring of deterioration (especially signs of expansion and deformation); and
- a range of laboratory tests (including petrographic characterization, chemical, physical, and mechanical tests) on samples collected from one or several components of the affected concrete structure.

7.2 MANAGEMENT PROGRAM FOR AAR-AFFECTED STRUCTURES

A global approach was recently proposed by FHWA for the diagnosis and prognosis of alkali-silica reaction (ASR) in transportation structures (Fournier et al. 2010). Figure 7.1 presents a summary of the above protocol. This step-by-step approach aims at evaluating the cause of concrete distress (**diagnosis**) and the potential for future expansion/damage (**prognosis**), both elements providing information for the selection of appropriate mitigation measures in ASR-affected structures. The extent to which each of the various methods described in the global approach will need to be implemented in a particular case will depend upon the nature/extent of the problem and the criticality of the structure, including the impact on the safety of users.
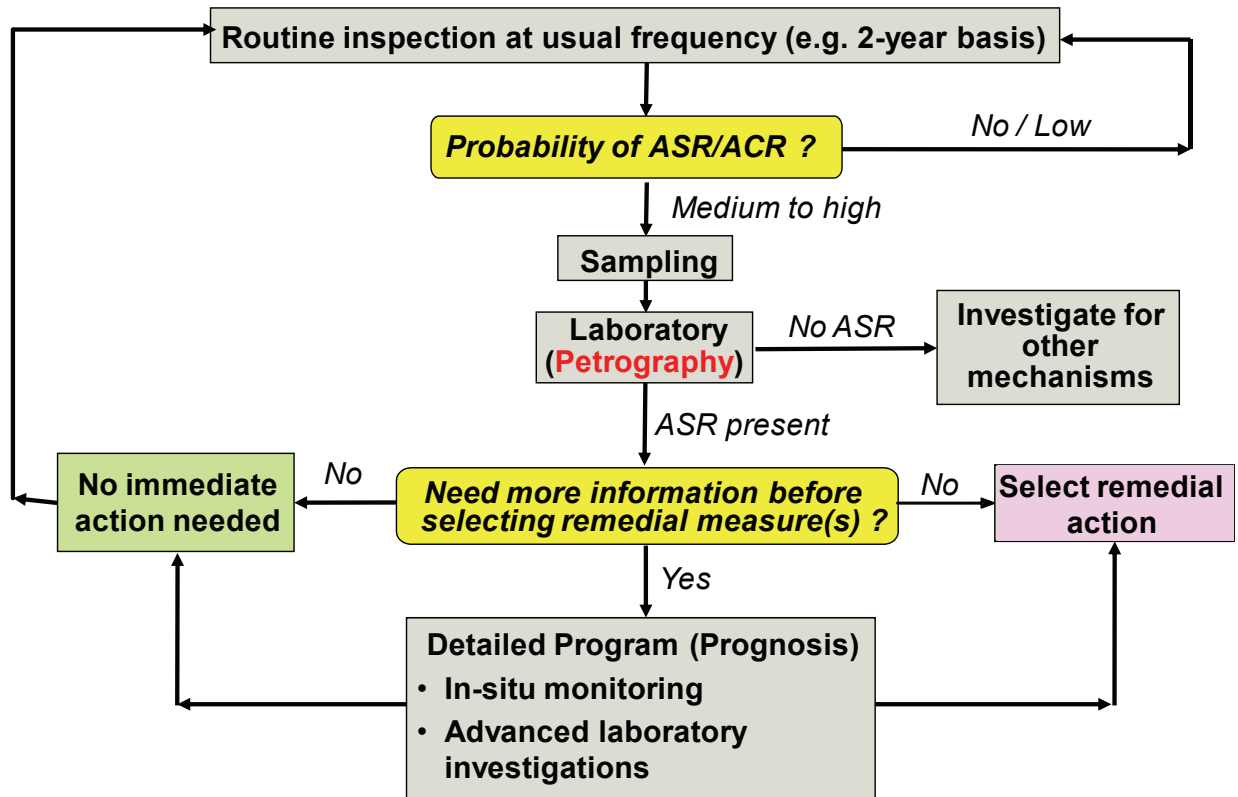
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┌──────────────────────────────────────────────────────────────┐
│ Routine inspection at usual frequency (e.g. 2-year basis) │◄──┐
└──────────────────────────────────────────────────────────────┘   │
                              │                                     │
                              ▼                   No / Low           │
                  ┌───────────────────────┐ ──────────────────────►─┘
                  │ Probability of ASR/ACR ? │
                  └───────────────────────┘
                              │ Medium to high
                              ▼
                        ┌──────────┐
                        │ Sampling │
                        └──────────┘
                              │
                              ▼                          ┌──────────────┐
                   ┌───────────────────┐   No ASR        │ Investigate for │
                   │ Laboratory        │ ──────────────► │ other           │
                   │ (Petrography)     │                 │ mechanisms      │
                   └───────────────────┘                 └──────────────┘
                              │ ASR present
                              ▼
┌──────────────┐   No  ┌──────────────────────────────┐   No   ┌──────────────┐
│ No immediate │◄───── │ Need more information before │ ─────► │ Select remedial │
│ action needed│       │ selecting remedial measure(s) ?│        │ action          │
└──────────────┘       └──────────────────────────────┘        └──────────────┘
                              │ Yes
                              ▼
                   ┌──────────────────────────────┐
                   │ Detailed Program (Prognosis) │
                   │ • In-situ monitoring         │
                   │ • Advanced laboratory        │
                   │   investigations             │
                   └──────────────────────────────┘
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Figure 7.1. Global Flow Chart for the Evaluation and Management of Concrete Structures for ASR (from Fournier et al. 2010)

7.2.1 ASR investigation program – Step 1 (Diagnosis)

7.2.1.1 Condition Survey

Signs of premature deterioration in concrete transportation structures that could be related to ASR can generally be detected during routine site inspections (condition survey). FHWA recently proposed *Alkali-Silica Reactivity Surveying and Tracking Guidelines* (Thomas et al. 2012a) that are intended to assist engineers, inspectors, and users in tracking and surveying ASR-induced expansion and cracking in bridges, pavements, and tunnels.

In the case of *bridge structures*, inspection elemental data can first be collected for the various elements of the structure, for example in accordance with the new AASHTO Guide Manual for Bridge Element Inspection (2011). The presence and extent of ASR-related defects are noted and quantified in accordance to the four conditions states described in Table 7.1.

Table 7.1. Standard Condition States for Defects in Bridge Elements (after AASHTO 2011)

Condition State #	Condition State
1	Good
2	Fair
3	Poor
4*	Severe

* Condition state 4 (severe) is typically reserved for conditions that warrant
  safety concerns and that are beyond the range of defects described in
  condition states 1 through 3.

The concept is to identify defects that are specific/unique enough so the manifestation of distress may be attributed to ASR. An updated version of the Strategic Highway Research Program (SHRP) ASR Handbook (Stark 1991), titled "Alkali-Silica Reactivity Field Identification Handbook" (Thomas et al. 2011), provides detailed coverage on the causes and effects of ASR and a wide range of photographs illustrating common symptoms of ASR, as well as examples of severity ratings. These include *map cracking*, *aligned cracking*, *gel exudation*, and *relative dislocation/ misalignment* of adjacent sections. Table 7.2 summarizes the proposed ASR-related defects, defect descriptions, and criteria/threshold for bridge element types using a similar approach as in the AASHTO guide manual (2011).

Table 7.2. Recommended Defects and Condition States for Bridge Elements Potentially Affected by ASR

Defect	Condition State 1	Condition State 2	Condition State 3	Condition State 4
Map Cracking	None to hairline	Narrow size or density, or both	Medium size or density, or both	The condition is beyond the limit state of Condition State 3, warrants a structural review to determine the strength or serviceability of the element or bridge, or both
Aligned Cracking	None to hairline	Narrow size or density, or both	Medium size or density, or both	
Gel Exudation	None	Moderate	Severe (with gel staining)	
Relative dislocation/ misalignment	None	Tolerable	Approaching or exceeding limits (including causing local crushing)	

More detailed descriptions of the above defects are provided in Table 7.3; examples of defects with different condition states are given in Figure 7.2. In addition, environmental conditions, especially temperature, relative humidity, exposure to sun and winds, and precipitation, can be tracked and coupled with other inspection findings to attempt linking the specific climatic conditions to the progress of ASR.

Table 7.3. Recommended Defects and Condition States for Bridge Elements Potentially Affected by ASR

Defect	Hairline-Minor	Narrow-Moderate	Medium-Severe
Map Cracking	Crack width < 0.0625" (1.6 mm) % Map Cracking < 5%	Crack width: 0.0625" (1.6 mm) – 0.1250" (3.2 mm) % Map Cracking: 5 to 25%	Crack width > 0.1250" (3.2 mm) % Map Cracking > 25%
Aligned Cracking	Crack width < 0.0625" (1.6 mm)	Crack width: 0.0625" (1.6 mm) – 0.1250" (3.2 mm)	Crack width: > 0.1250" (3.2 mm)
Gel Exudation	None	Gel visible on surface (< 20% of concrete surface, with no build-up of gel)	Gel build-up on surface (>20% of concrete surface), typically at or near cracks; gel staining visible (especially once structure dries after a rain event)
Relative dislocation/ misalignment	None	Tolerable (movement is visible but no loss of clearance, exudation of sealants at joints, or local crushing)	Movement is visual, with loss of clearance, exudation of sealants at joints, or local crushing

Figure 7.2. ASR-related defects in bridge elements and severity assessment. A. Map cracking on bridge abutment (condition state 3). B. Aligned cracking on column (condition state 2). C. Aligned cracking in beam (condition state 3). D. Gel exudation (condition state 3). E. Relative dislocation/misalignment, leading to loss of clearance and cracking (SHRP C-315 1991) (condition state 3).

A similar approach can be followed for pavements. The distresses most commonly associated with ASR in pavements include *map cracking*, *joint deficiencies/deterioration*, and in some cases, *popouts*. Tables 7.4 and 7.5 provide detailed criteria/description for the distress types, followed by selected photographs of the defects, some with varying degree of severity (Figure 7.3).

Table 7.4. Recommended Distress and Condition States for Pavements Potentially Affected by ASR

Distress	Condition State 1	Condition State 2	Condition State 3	Condition State 4
Map Cracking	None to hairline	Narrow size or density, or both	Medium size or density, or both	The condition is beyond the limit state of Condition State 3, warrants a structural review to determine the strength or serviceability of the pavement, or both.
Joint Sealant Failure	None	Moderate	Severe	
Joint Deterioration	None	Moderate	Severe	
Popouts	None	Moderate	Severe	

Table 7.5. Recommended Distress and Distress Descriptions for Pavements Potentially Affected by ASR

Defect	Hairline-Minor	Narrow-Moderate	Medium-Severe
Map Cracking	Crack width < 0.0625" (1.6 mm) % Map Cracking < 5%	Crack width: 0.0625" (1.6 mm) – 0.1250" (3.2 mm) % Map Cracking: 5 to 25%	Crack width > 0.1250" (3.2 mm) % Map Cracking > 25%
Joint Sealant Failure	Joint sealant failure in less than 10% of joints.	Joint sealant failure in 10 to 50% of joints.	Joint sealant failure in greater than 50% of joint
Joint Deterioration	None or only minor cracking near corners/joints	Wide, open cracks exist and mass loss has occurred in joint region (less than 5% of joints). No patching applied.	Wide, open cracks and mass loss has occurred in joint region (greater than 5% of joints). Patching has been applied.
Popouts	None	Popouts isolated and few [< 1 popout per 10 ft]	Popouts prevalent [> 1 popout per 10 ft]

* Popout data generally not collected and not included in LTPP. Estimates are in []
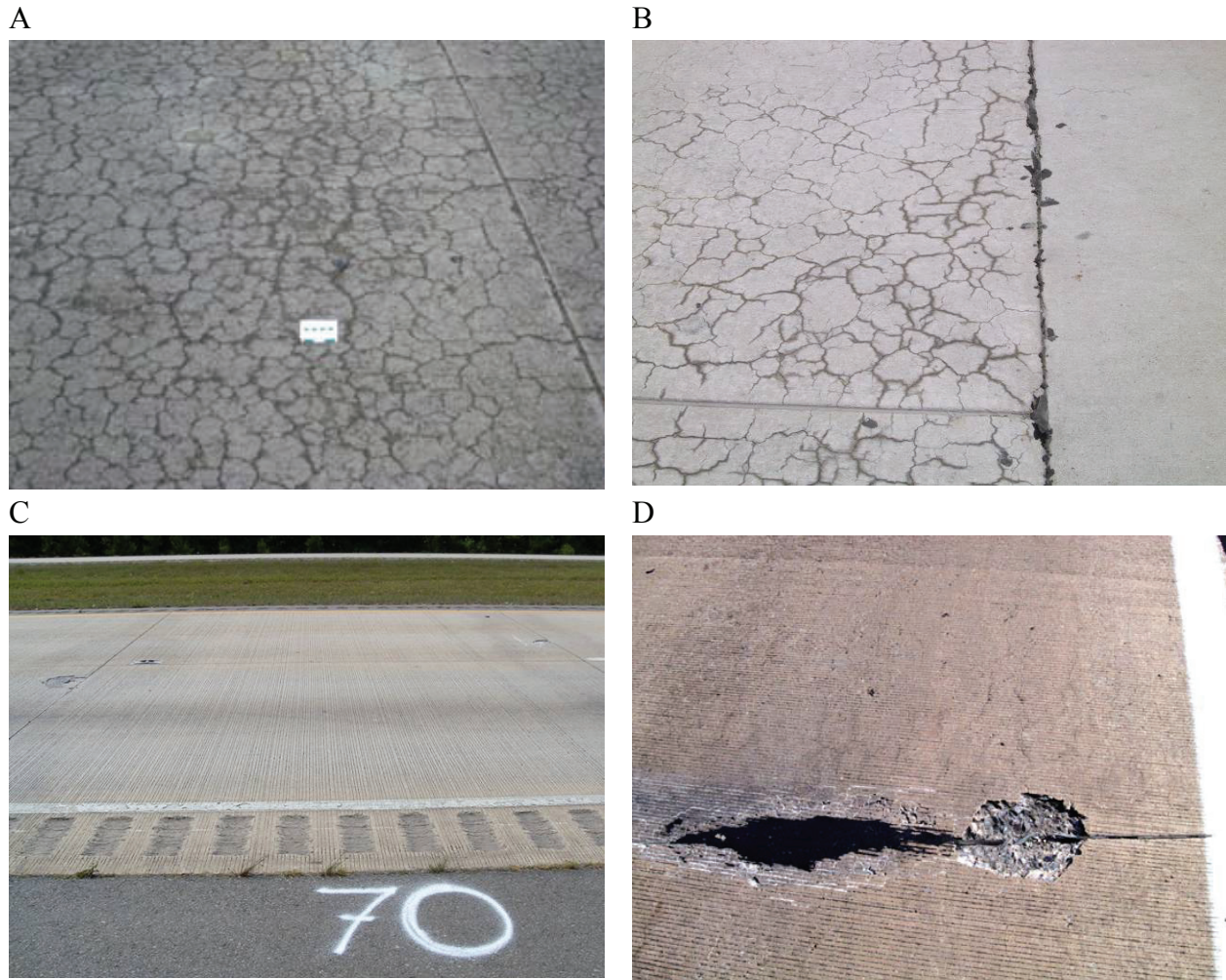
A

B

C

D

Figure 7.3. ASR-related defects in pavements and severity assessment. A. Map cracking of concrete pavement (condition state 3). B. Joint Sealant Failure in Concrete Pavement (condition state 3). C. Joint Deterioration in Concrete Pavement (condition state 2). D. Joint Deterioration in Concrete Pavement (condition state 3).

If the condition survey points out issues that can impair the integrity of the structure or public safety (related or not to ASR), immediate action should be taken in consultation with experts in the respective fields.

7.2.1.2 Documentation

Any documents (i.e., testing of materials, construction, and inspection reports) related to the structure examined should be gathered as they may provide valuable information in the appraisal process. This activity could also be carried out either in preparation for the condition survey or

following it, i.e., for structures where some signs of deterioration potentially indicative of ASR have been noticed. Useful information could include the following (CSA 2000):

- Type and location of the structure and, hence, its likely exposure conditions due to its nature of operation and geography.
- Age of the structure and details and dates of any modifications or repairs. ASR may take from 3 to even more than 25 years to develop significantly in concrete structures depending on factors such as the nature (reactivity level) of the aggregates used, the moisture and temperature conditions, and the concrete alkali content.
- Plans, drawings, and specifications.
- Details of concrete mixes used, particularly mix proportions, source of cement and aggregates, and details of any analyses or tests carried out on concrete materials. The availability of samples of these materials should also be checked; some agencies store samples of cements and aggregates used in major projects.
- Previous inspection/testing reports, especially dates when deterioration was first observed.
- Information from other local structures that may have been constructed with the similar materials, especially if these structures are exhibiting signs of deterioration typical of ASR.

Details regarding the concrete materials, especially the composition and proportion of the cement and the type of aggregate used, are most useful when assessing the likelihood of ASR. It is recognized that information of this nature is often not available or lacks specific detail in the case of many structures; however, it is important to collect whatever data is available.

### 7.2.1.3   *Laboratory investigation - petrography*

When the nature and extent of the visual signs of deterioration observed on the structure(s) under inspection are such that ASR is a possibility, investigations can be carried out to determine whether or not ASR is a contributing factor in the deterioration observed. Cores are collected from components showing typical features of deterioration suggestive of ASR (e.g. Figures 7.2 and 7.3), which, most often, correspond to structural components exposed to a constant or renewable supply of moisture. For comparison purposes, it will also be appropriate to collect cores from non or less deteriorated/exposed members of the structure.

Various types of specimens, e.g. polished sections or slices, broken (fresh) surfaces, and thin sections can be prepared from the core samples. They will then be examined petrographically to recognize any signs of deterioration associated with ASR and other deleterious mechanisms (BCA 1992; St-John et al. 1998; Walker et al. 2006). Although not necessarily exclusive to ASR, petrographic signs of ASR generally consist of (Figure 7.4):

- microcracking in the aggregate particles (coarse and/or fine) and/or the cement paste;
- presence of reaction products "alkali-silica gel" in the cracks of the aggregate particles and/or of the cement paste, as well as in air voids of the cement paste;
- reaction rims (to differentiate from weathering rims); and
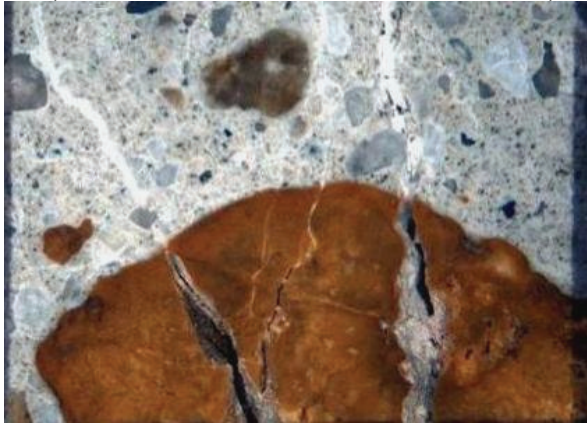- loss of the cement paste-aggregate bond.
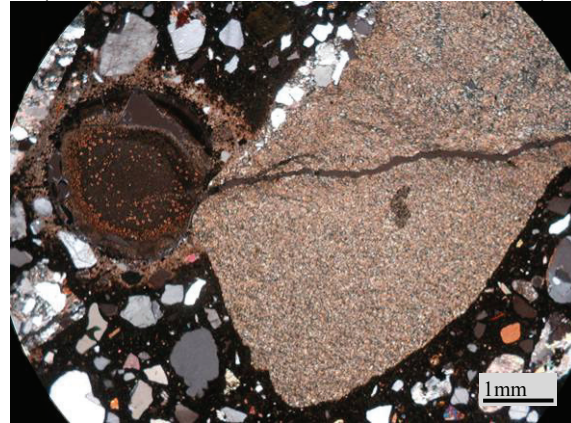
A (distance between the vertical lines = 1 cm)
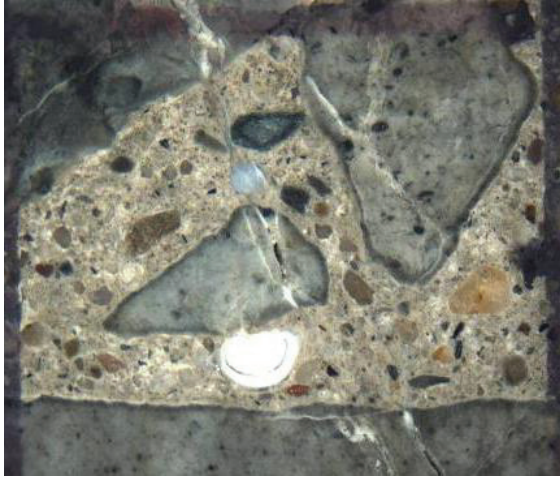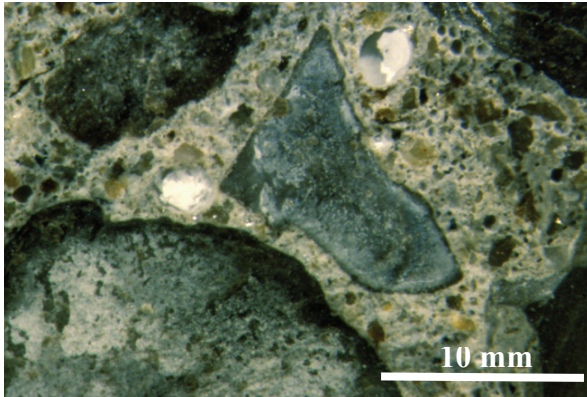
B (distance between the vertical lines = 1 cm)

C (distance between the vertical lines = 1 cm)

D (distance between the vertical lines = 1 cm)
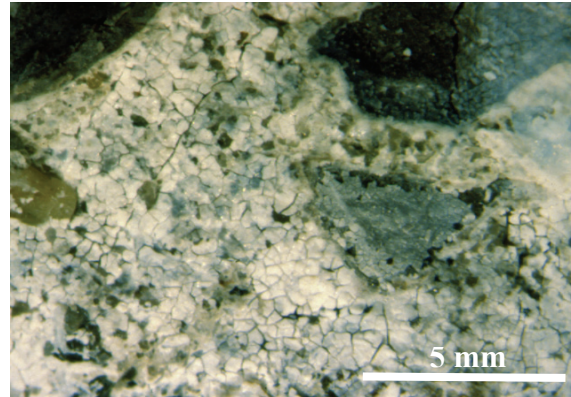
1mm

E

F

10 mm

5 mm

Figure 7.4. Typical petrographic features of deterioration due to ASR

- Polished concrete sections (stereomicroscope): Cracks with reaction products in coarse (A,C) and fine (B) aggregate particles. Reaction rims around reactive coarse aggregate particles (A). Reaction products (ASR gel) in cracks (A,C) and voids (A) of the cement paste.
- Thin sections (petrographic microscope). Reaction products (ASR gel) in cracks of coarse aggregate particles and in an adjacent air void of the cement paste (D).
- Broken surface of concrete sample (stereomicroscope): Alkali-silica reaction products on broken surfaces of reactive coarse aggregate particles (E), and in voids (E) or covering cracked surfaces of the cement paste (F).

Staining techniques have been proposed to facilitate identification of the reaction product gel in concrete affected by ASR (Natesaiyer et al. 1991; Stark 1991; Guthrie and Carey 1997). A technique developed at Cornell University (Natesaiyer et al. 1991) consists in applying an uranyl acetate solution on polished or fresh broken surfaces of concrete specimens to be examined followed by a visual observation of the section under a UV light; the technique has even been used on field structures (Stark 1991; AASHTO 1993; ASTM C 856-02). Stark (1991) indicated that "by applying the uranyl acetate solution to a surface containing the gel, the uranyl ion substitutes for alkali in the gel, thereby imparting a characteristic yellowish-green glow when viewed in the dark using short wavelength ultraviolet light ASR gel fluoresces much brighter than cement paste due to the greater concentration of alkali and, subsequently, uranyl ion in the gel". This technique should be used with great care following appropriate health and safety procedures because of the potentially hazardous nature of the product. Technically speaking, the results of the test should be interpreted with great care since some aggregates fluoresce naturally, which can incorrectly suggest the presence of alkali-silica gel through macroscopic or microscopic examinations. Guthrie and Carey (1997) proposed a method that consists in treating fresh concrete surfaces to successive applications of Sodium Cobaltinitrite and Rhodamine B. Upon treatment, regions affected by ASR stain either yellow or pink. According to Guthrie and Carey (1997), yellow staining would be associated with massive ASR-related precipitate with gel-like morphology as well as granular precipitate consisting of crystals that have grown from the gel. On the other hand, yellow stained regions would correspond to alkali-bearing siliceous reaction products resulting from ASR.

Table 7.6 classifies the occurrence of the features obtained from the petrographic examination as indicative of low, medium, and high probability of ASR. When petrographic evidence of ASR is confirmed, a decision on further steps to be taken is based on factors such as the severity of the damage and the "criticality" of the structure (Fournier et al. 2010) (Figure 7.1). In some cases, it may be decided that additional "technical" investigations are not required and some remedial actions could/should already be implemented. Examples of such cases are further discussed in Fournier et al. (2010). Also, in some cases, the extent of the damage is such that no immediate action is needed; the structure will then be re-examined as part of the routine condition survey. However, in the case of "critical" structures (e.g. large size/major highway bridges and pavements, hydraulic dams) or when the extent of deterioration is significant, a detailed laboratory and/or in-situ investigation program may be necessary.

Table 7.6. Classification system for petrographic examination (Fournier et al. 2010)

Probability of ASR	Nature and Extent of Features
Low	No potentially reactive rock types (from petrographic examination of thin sections): • no alkali-silica gel present (or only in a very few air voids), no (or very few) reaction rims, no (or very few) sites of expansive reaction, very limited cracking within the aggregate particles that extends, or not, into the cement paste; • presence of other indicative features rarely found.
Medium	Presence of some features generally consistent with AAR: • damp patches on core surfaces; • presence of potentially reactive rock types; • cracking/microcracking within a fair number of aggregate particles; some of the cracks may extend into the cement paste; • alkali-silica gel observed in cracks within a fair number of aggregate particles and/or cracks within the cement paste and/or air voids; • darkening of cement paste around reactive aggregate particles, cracks or voids; • reaction rims around the internal periphery of a fair number of reactive particles.
High	Presence of extensive signs of ASR (as described in the previous section but observed in larger frequency), for instance: • evidence of site of expansion reaction, i.e. where evidence or reaction and emanation of swelling pressure can be positively identified; and/or • presence of gel in cracks and voids associated with several reactive particles and readily visible to the unaided eye or under low magnification.

7.2.2 ASR investigation program – Step 2 (Prognosis)

This part of the study aims at generating additional technical information, leading to a more complete assessment of the degree of damage due to ASR in the concrete structure and the selection of most appropriate remedial measures (Figure 7.1). The selection of the activities will depend on the criticality of the structure, the amount of time/funding available to generate the data, and the degree of precision expected.

7.2.2.1   In-situ investigations

An in-situ investigation program, which includes monitoring of expansion, deformation, and availability of moisture over several years (to account for seasonal temperature changes), can provide "prognostic" information for ASR-affected structural members. The detailed in-situ investigation program can include one or several of the following:

• *Surface cracking*. The extent of surface cracking on severely exposed/cracked sections of concrete elements is somewhat related to the overall amount of expansion reached by the affected member. The *Cracking Index (CI)* method consists in the measurement and summation of crack widths along a set of lines drawn perpendicularly on the surface of the concrete element investigated (LCPC 1997, 1999) (Figure 7.5). The method gives a quantitative assessment of the extent of cracking in structural members, either punctually

(when obtained at a specific time) or as a progressive process when performing the measurements, on a regular basis, at the exact same location (rate of expansion – prognosis).

- *Expansion and deformation measurements*. Measurements of expansion/deformation can be performed using different means (e.g. demec points and/or metallic references at the surface of selected structural members, extensometers, invar wires/rods or optical systems (leveling)) (Fournier et al. 2010; LCPC 2003, 2009; Thomas et al. 2012b) (Figure 7.6).
- *Temperature and humidity measurements*. The relative humidity in a concrete structure can be measured over time with depth or laterally in different concrete elements using various techniques (e.g. wooden stick, portable or permanent probes) (Thomas et al. 2012b; Jensen 2004) (Figure 7.7).
- *Non-destructive testing*. Periodic measurements, such as pulse velocity, impact echo, acoustic methods, etc. can be made on specific members of the affected structure (at the surface or in the bottom of drilling holes) to determine the evolution/extent of internal cracking or deterioration (Thomas et al. 2012b; Moradi-Marani et al. 2011; Tajari et al. 2011; Sargolzahi et al. 2010) (Figure 7.8).

A
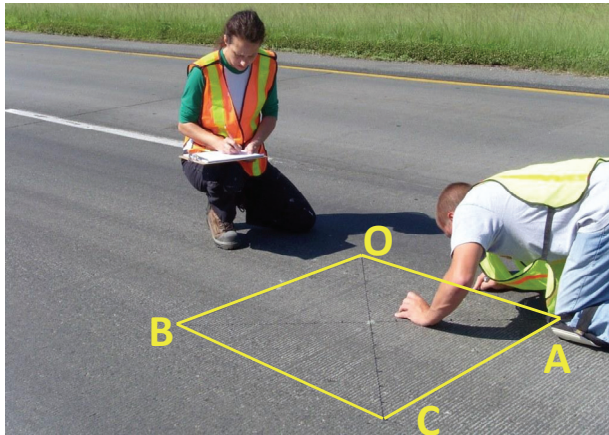
B

C

Interval (10 cm)		1	2	3	4	5	Base Length (m)	# cracks	Crack opening (mm)			
		6	7	8	9	10			Total sum	Avg. /crack	Avg. /m	C.I. mm/m
Vertical direction	OA	0.1, 0.1	0.2	--	--	0.6	0.5	6	1.8	0.3	3.6	3.2
		0.4	--	--	0.4	--						
	BC	0.1	0.4	--	0.1	0.1	0.5	7	1.4	0.2	2.8	
		--	0.3	0.2	0.2	--						
Horizontal direction	OB	--	--	0.3, 0.5	0.4	0.3	0.5	7	2.8	0.4	5.6	4.6
		0.4	0.3	--	--	0.6						
	AC	0.5	0.2	--	--	0.3	0.5	8	1.6	0.2	3.6	
		0.05	0.05	0.1, 0.2	--	0.2						

Figure 7.5. Measurements of the *Cracking Index* on ASR-affected pavement (A) and jersey barrier (B). An example of the data generated in the *Cracking Index method* is given in (C). The values of C.I. are given separately for the vertical and horizontal measurements.

A
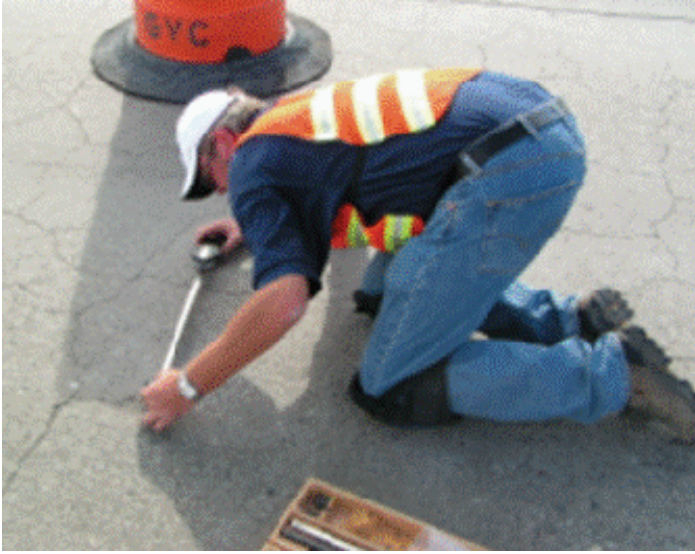
B

Figure 7.7. In-situ measurements of humidity in concrete. A. Wooden-stick method. B. Vaisala probes.

A

B

Figure 7.7. In-situ measurements of humidity in concrete. A. Wooden-stick method. B. Vaisala probes.
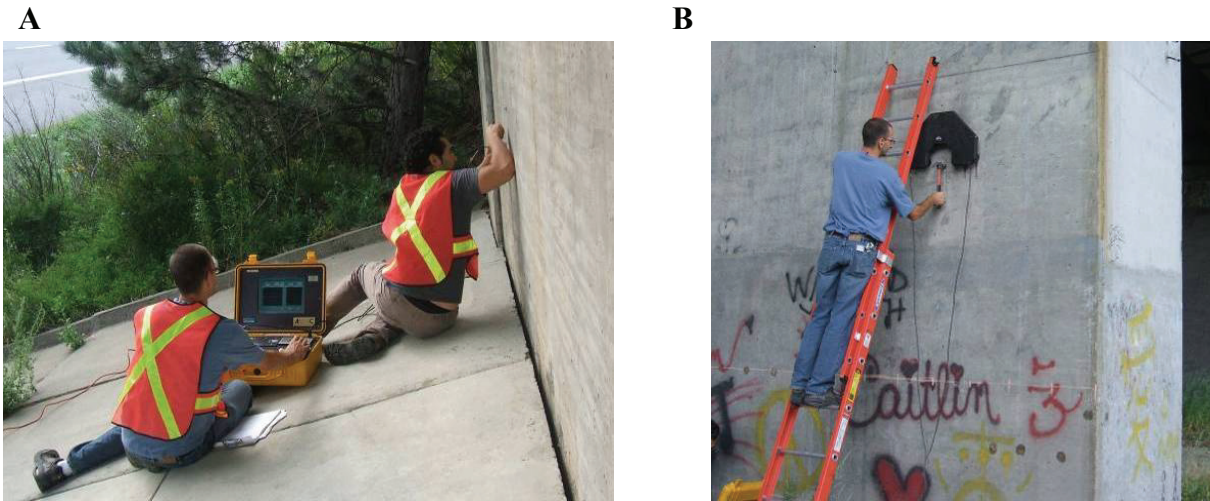
A   B

Figure 7.8. Non-destructive testing of ASR-affected concrete structures. A. Impact-echo. B. Non-linear acoustic technique.

7.2.2.2 Detailed laboratory investigations

A series of tests can be performed on core samples extracted from structural members showing different degrees of deterioration and/or of exposure conditions. The data will provide information for further evaluating the current concrete condition, the expansion reached to date, the current rate of expansion, and the potential for future expansion of the concrete.

- *Quantitative petrography*. The *Damage Rating Index* (DRI), a semi-quantitative petrographic technique, evaluates the condition of concrete by counting, under the stereomicroscope (~16x magnification), the number of typical petrographic features of ASR on polished concrete sections (Grattan-Bellew 1992). The DRI represents the normalized value (to 100 mm^2) of the presence of these features after the count of their abundance over the surface examined has been multiplied by weighing factors representing their relative importance in the overall deterioration process (Villeneuve et al. 2012) (Table 7.7; Figures 7.4 and 7.9).

- *Mechanical testing*. In most cases, the properties most rapidly affected by ASR are the modulus of elasticity and the direct tensile strength (ISE 1992; Pleau et al. 1989). The *Stiffness Damage Test (SDT)*, which consists in subjecting concrete cores to 5 cycles of uniaxial loading/unloading up to a maximum of 5.5 MPa (Chrisp et al. 1989) or 10 MPa (Smaoui et al. 2004) (Figure 7.10A), can be used for assessing the ASR expansion attained to date through (1) the energy dissipated during the first cycle (hysteresis loop), and (2) the accumulated plastic strain after the 5 load/unload cycles (Figure 7.10B). Recent research indicates that more accurate results are obtained when the SDT is carried out at a percentage of the design (28-day) strength (40%) instead of a fixed load (Sanchez et al. 2012). Calibration curves correlating the above output parameters and expansion obtained on

laboratory specimens can be used for estimating the amount of expansion reached to date by the field concrete (Smaoui et al. 2004) (Figure 7.10C).

- *Expansion tests on cores*. Expansion tests on cores (in air at > 95% R.H. and 38°C) can provide an "estimate" of the potential for further expansion of ASR-affected concrete over a relatively short period of time, e.g. six months to one year (Fournier et al. 2010; Fecteau et al. 2012; Bérubé et al. 2002; CSA 2000) (Figure 7.11). Although this method is commonly used, the correlation between the "free" expansion of cores (i.e. when extracted from their restraining environment) and the expansion of the corresponding structural element in the field is yet to be established.
- *Alkali content of concrete*. The measurement of the "available/residual" alkali content in concrete can yield interesting information in assessing whether the concrete tested contains sufficient alkalis to sustain this reaction (*prognosis*). The "available/residual" alkali content in concrete can be obtained by hot-water extraction or, when possible, using pore solution extraction devices (Fournier et al. 2010; Bérubé and Tremblay 2004).

Table 7.7: Petrographic Features and Weighing Factors for the DRI (*Grattan-Bellew and Mitchell 2006*)

Petrographic feature	Abbreviation	Weighing factor
Coarse aggregate with cracks	CrCA	x 0.75
Open crack in coarse aggregate	OCrCA	x 4.0
Coarse aggregate with cracks and reaction products	Cr+RPCA	x 2.0
Coarse aggregate debonded	CAD	x 3.0
Reaction rims around aggregate	RR	x 0.5
Cement paste with cracks	CrCP	x 2.0
Cement paste with cracks and reaction products	Cr+RPCP	x 4.0
Air voids lined or filled with reaction products	RPAV	x 0.50

A

B

Figure 7.9. Examination of the polished concrete section under the stereomicroscope for the determination of the *Damage Rating Index*. Petrographic features of deterioration (Table 7.7; Figure 7.4) are counted in a one cm by one cm grid system drawn at the surface of the polished concrete section.
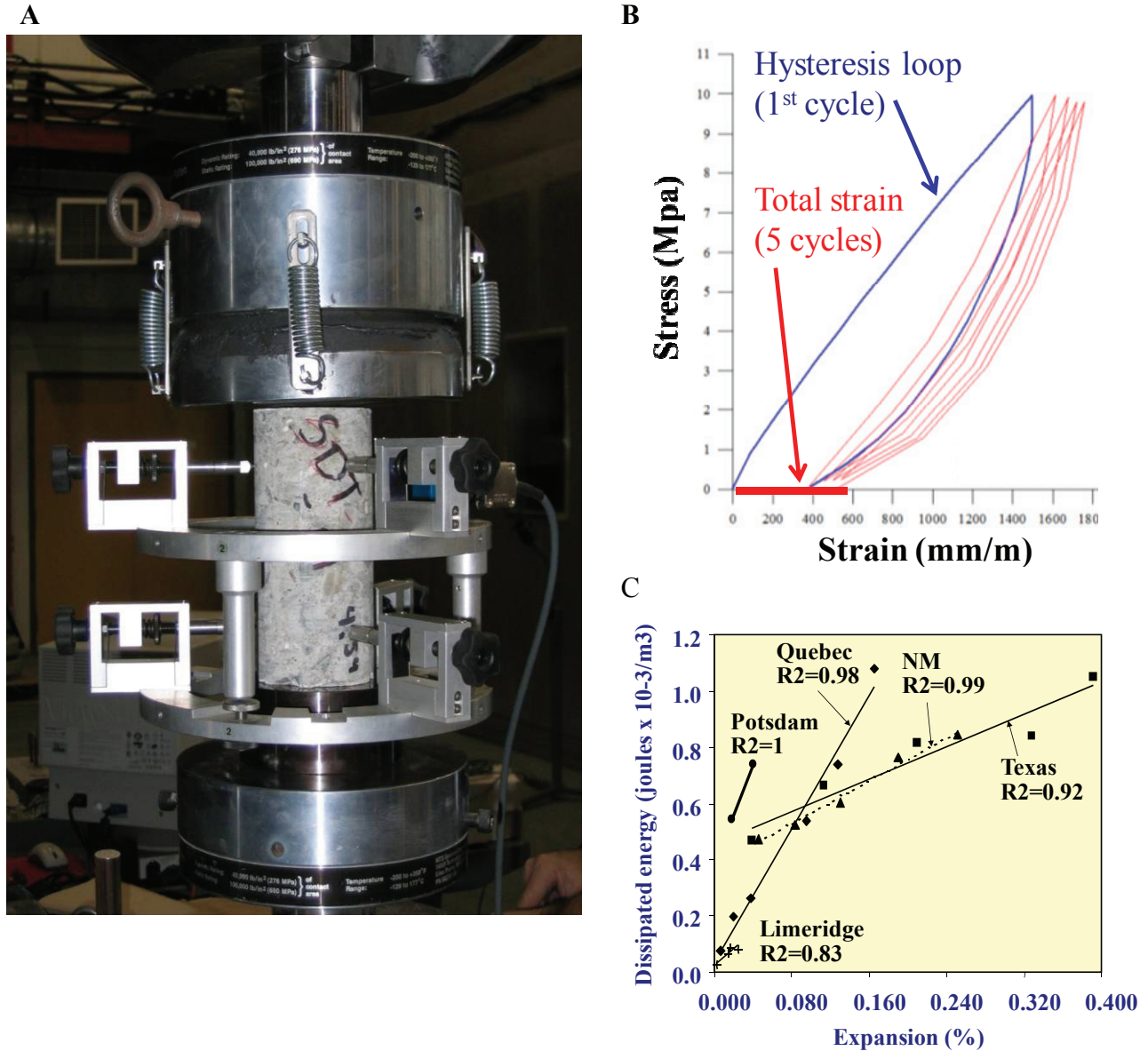
A

B

C

Figure 7.10. Mechanical testing of concrete cores using the Stiffness Damage Test (SDT). A. Cores subjected to 5 loading/unloading cycles up to 10 MPa. B. The best output parameters correspond to the energy dissipated during the first cycle (hysteresis loop) and the accumulated plastic strain after the 5 cycles. C. Calibration curves for different reactive aggregates (Smaoui et al. 2004).
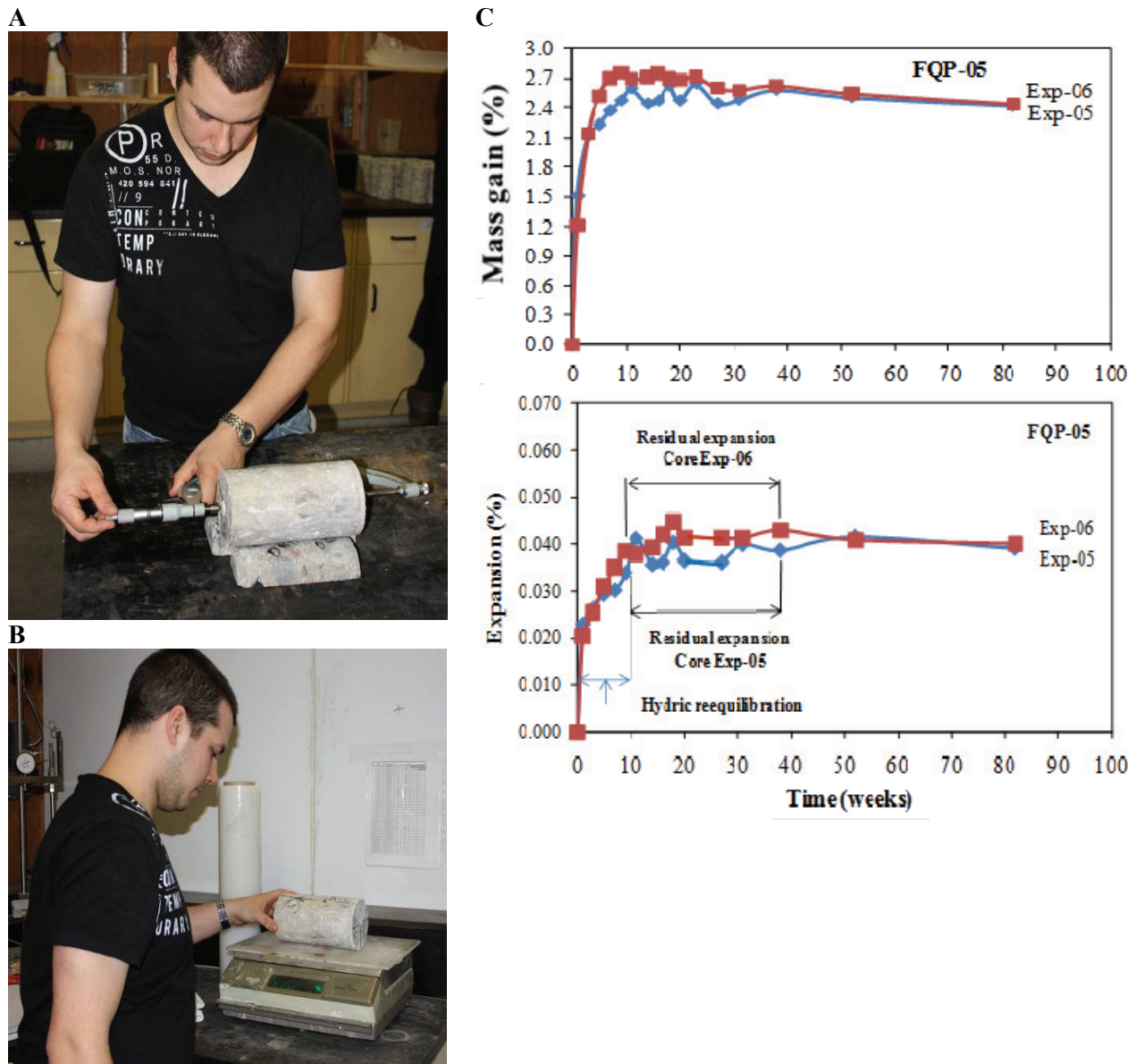
Figure 7.11. Residual expansion testing carried out on core samples (38°C, R.H. > 95%). A. Expansion measurement of the core. B. Mass measurements are also carried out in parallel to the expansion measurements to determine the moisture uptake capacity of the core. C. Example of comparative mass and residual expansion measurements for cores. This allows differentiating the *hydric reequilibration* of the concrete at the beginning of the test (due to moisture uptake only) and the *residual expansion* potential due to ASR.

7.2.2.3 Collective assessment of in-situ and laboratory investigations

Fournier et al. (2010) provides a scheme for the analysis of the results obtained from the in-situ and laboratory investigations for prognostic evaluation of ASR. In summary, the authors suggest

that in the case of reinforced concrete members (e.g. bridges), the potential for further expansion due to ASR will be expressed by the number of years before the reinforcing steel yield (in the direction of lower or lack of restraint) could occur, which requires data on the ASR _expansion attained to date_ and the _current expansion rate_. In the case of concrete pavements, the potential for further expansion due to ASR will be expressed by the number of years before the joints could close, which requires information on the current expansion rate and widths of joints. The urgency of applying remedial actions will then be partly based on criteria related to the delay before steel yielding in reinforced concrete members (expansion criteria of > 0.20% is proposed), or the delay before the closure of expansion joints occur in the case of concrete pavements. It is recommended that further action be taken when the delay before steel yielding or joint closure is estimated to be less than 5 years, for example by starting an in-situ monitoring program of expansion, with measurements at least on a yearly basis, and/or by performing a structural assessment of the member/structure; it would be appropriate to confirm an assessment that has been based essentially on expansion tests on cores rather than on in-situ monitoring.

7.3 CONCLUSION

Unexpected or premature concrete deterioration due to alkali-silica reactivity (ASR) is a widespread problem worldwide. Routine site inspections performed on a regular basis may permit identification of the problem; however, ASR in concrete cannot generally be diagnosed without detailed site investigations. Such investigations would include determination of the distribution and severity of the various defects affecting the concrete structure, as well as laboratory testing (petrography) of samples collected from the affected concrete structures. For critical structures such as large dams and fair to large size highway bridges, detailed investigations including a more extensive sampling program might be necessary to underline quantify the current condition of the concrete, and to evaluate the potential for future deterioration (prognosis). Such investigations can involve a detailed sampling program for further testing in the laboratory and in-situ monitoring of the progress of expansion/deterioration. The results of the above processes of investigation will then be analyzed to propose appropriate management actions to be taken for each of the particular applications.

REFERENCES

American Association of State Highway and Transportation Officials (AASHTO). "Standard Method of Test for Rapid Identification of Alkali-Silica Reaction Products in Concrete (Uranyl Acetate Procedure)," AASHTO T299-93, 1993.

American Association of State Highway and Transportation Officials (AASHTO). 2011. AASHTO guide manual for bridge element inspection (1st edition), 172 p.

American Standards for Testing and Materials. "Standard Practice for Petrographic Examination of Hardened Concrete," ASTM International, ASTM C856-02, *Annual Book of ASTM Standards*, Section Four, Vol. 04.02 Concrete and Aggregates, 434-450, 2003.

Bérubé, M.A. and Tremblay, C. 2004. "Chemistry of pore solution expressed under high pressure – influence of various parameters and comparison with hot-water extraction method." 12th International conference on AAR in Concrete, Beijing (China), October 2004, Int. Academic Publishers, Beijing World Publishing Corp., 833-842.

Bérubé, M.A., Frenette, J., Rivest, M. 2002. "Laboratory Assessment of the Potential Rate of ASR Expansion of Field Concrete." Cement, Concrete, and Aggregates, 24 (1), 28-36.

British Cement Association (BCA). 1992. "The Diagnosis of Alkali-Silica Reaction – Report of a Working Party." Wexham Springs, Slough (UK), SL3 6PL, 44 p.

Canadian Standards Association (CSA). 2000. "Guide to the Evaluation and Management of Concrete Structures Affected by Alkali-Aggregate Reaction." CSA A864-00, Canadian Standards Association, Mississauga, Ontario, Canada.

Chrisp, T.M., Wood, J.G.M., Norris, P. 1989. "Towards Quantification of Microstructural Damage in AAR Deteriorated Concrete." International Conference on Recent Developments on the Fracture of Concrete and Rocks, Elsevier Applied Science, London, U.K., 419-427.

Fecteau, P.L., Fournier, B., Duchesne, J. 2012. "Residual expansion testing: new aspects on cores extracted from exposure blocks submitted to environmental conditions." 14th International Conference on AAR in Concrete, May 20-25 2012, Austin (Texas), electronic.

Fournier, B., Bérubé, M.A., Folliard, K.J., Thomas, M.D.A. 2010. "Report on the diagnosis, prognosis and mitigation of alkali-silica reaction (ASR) in transportation structures." Federal Highway Administration, FHWA-HIF-09-004.

Grattan-Bellew, P.E. 1992. "Comparison of Laboratory and Field Evaluation of Alkali-Silica Reaction in Large Dams," First International Conference on Concrete AAR in Hydroelectric Plants and Dams, Fredericton, NB, Canada, 23 p., September-October 1992.

Grattan-Bellew, P.E., Mitchell, L.D. 2006. "Quantitative petrographic analysis of concrete –The Damage Rating Index (DRI) method, a review." Marc-André Bérubé symposium on AAR in concrete, CANMET/ACI Advances in concrete technology, Montréal, Canada, 321-334.

Guthrie, G.G., Jr. and Carey, J.W. "A Simple Environmentally Friendly, and Chemically Specific Method for the Identification and Evaluation of the Alkali-Silica Reaction," Cement and Concrete Research, Vol. 27, No. 9, 1407-1417, 1997.

Institution of Structural Engineers (ISE). 1992. "Structural Effects of Alkali-Silica Reaction – Technical Guidance Appraisal of Existing Structures." Institution of Structural Engineers, 11 Upper Belgrave Street, London SW1X 8BH, 45 p.

Jensen, V. 2004. "Measurements of cracks, relative humidity and effects of surface treatments on concrete structures damaged by alkali-silica reaction." 12th International conference on AAR in Concrete, Beijing (China), October 2004, Int. Academic Publishers, Beijing World Publishing Corp., 1245-1253.

Laboratoire Central des Ponts et Chaussées (LCPC). Ministère de l'équipement, des transports et du logement, 58 bd Lefebvre F75732 Paris cedex 15,
- 1997. Détermination de l'indice de fissuration d'un parement de béton. Méthode no. 47.
- 1999. Manuel d'identification des réactions de dégradation interne du béton dans les ouvrages d'art.
- 2003. Aide à la gestion des ouvrages atteints de réactions de gonflement interne.
- 2009. Méthodes de suivi dimensionnel et de suivi de la fissuration des structures.

Moradi-Marani, F., Kodjo, S.A., Rivard, P., Lamarche, C.P. 2011. "Application of the mechanical perturbation produced by traffic as a new approach of nonlinear acoustic technique for detecting microcracks in concrete: a laboratory simulation." Conference on Review of progress in quantitative nondestructive evaluation (QNDE), Burlington (USA), July 17-22, 2011.

Natesaiyer, K., Stark, D. and Hover, K.C., "Gel Fluorescence Reveals Reaction Product Traces," Concrete International, 25-28, January 1991.

Pleau, R., Bérubé, M. A., Pigeon, M., Fournier, B., Raphaël, S. 1989. "Mechanical Behavior of Concrete Affected by AAR. 8th International Conference on AAR in Concrete." Kyoto, Japan, 1989, 721-726.

St-John, D.A., Poole, A.W., Sims, I. 1998. "Concrete Petrography: A handbook of investigative techniques." Arnold. London. 474 p.

Sanchez, L., Fournier, B., Jolin, M. 2012. "Study of the parameters of the stiffness damage test for assessing concrete damage due to alkali-silica reaction." Proceedings of the 14th International Conference on AAR in Concrete, May 20-25, 2012, Austin (Texas), electronic.

Sargolzahi, M., Kodjo, S.A., Rivard P., Rhazi, J. 2010. "Effectiveness of nondestructive testing for the evaluation of ASR in concrete." Construction & Building Materials, 24(8): 1398-1403.

Sims, I. and Nixon, P.J. 2010. "Outline guide to the use of RILEM methods in assessments of alkali-reactivity potential." RILEM Recommended Test method AAR-0. RILEM committee documentation (September 2010).

Smaoui, N., Bérubé, M.A., Fournier, B., Bissonnette, B. and Durand, B. 2004. "Evaluation of the Expansion Attained to Date by ASR-Affected Concrete – Part I: Experimental Study." Canadian Journal of Civil Engineering, 31: 826-845.

Stark, D. 1991. "Handbook for the Identification of Alkali-Silica Reactivity in Highway Structures." SHRP-C/FR-91-101, TRB National Research Council, 49p.

Tajari, M., Shekarchi, M. and Sadri, A. 2011. "Use of Impact-echo Technique for Detection of Distributed Damage in Concrete due to ASR." Material Evaluation, 69 (7): 881-890.

Thomas, M.D.A., Fournier, B., Folliard, K.J. and Resendez, B. 2011. "Alkali-silica reactivity field identification Handbook." FHWA-HIF-12-022, 80p.

Thomas, M.D.A., Fournier, B., Folliard, K.J. and Resendez, B. 2012a. "Alkali-silica reactivity surveying and tracking guidelines." July 2012, 32p.

Thomas, M.D.A., Folliard, K.J., Fournier, B., Drimalas, T. and Rivard, P. 2012b. "Study of remedial actions on highway structures affected by ASR." 14th International Conference on AAR in Concrete, May 20-25, 2012, Austin (Texas), electronic.

Villeneuve, V., Fournier, B. and Duchesne, J. 2012. "Determination of the damage in concrete affected by ASR – the Damage rating Index (DRI)." 14th International Conference on AAR in Concrete, May 20-25, 2012, Austin (Texas), electronic.

Walker, H.N., Lane, D.S. and Stutzman, P.E. 2006. "Petrographic Methods of Examining Hardened Concrete: A Petrographic Manual." (Revised 2004). Federal Highway Administration, FHWA-HRT-04-150.

8 – Mitigation Methods for ASR-Affected Structures

8.1 INTRODUCTION

This section briefly describes various mitigation measures that are available for ASR-affected structures, identifies options that have been used the most, discusses those whose effectiveness has been proven in the laboratory and field, and describes those that remain experimental in nature due to lack of data/information proving their merit in real-world applications. The main objective of this section is to provide guidance on means of extending the service life of ASR-affected structures. The term "mitigation" is used in lieu of "repair" because the methods described herein are generally not able to, nor are they intended to, repair or restore the original properties or integrity to the ASR-affected structure. Rather, the intention is to reduce future expansion of the structure or to lessen the detrimental impact of future expansion.

The majority of the work to date on treating existing structures has focused on ASR-affected structures, as opposed to ACR-affected structures, and there are by far many more ASR-affected structures worldwide. As such, the focus of this portion of this protocol is aimed at ASR-affected structures. However, some of the mitigation measures, particularly those aimed at drying the concrete, would be helpful whether it is ASR or ACR that is impacting the structure.

8.2 OVERVIEW OF MITIGATION METHODS

Figure 8.1 summarizes the various mitigation options that have been applied to field structures affected by ASR. This section will briefly discuss each of the options shown in Figure 8.1 and will then focus on those that have the greatest potential for effectively treating ASR-affected structures. For each of these options, the merits will be discussed, as well as inherent shortcomings, both in terms of general applicability to field structures and specific application to certain structures. Recent FHWA-funded field trials will be highlighted, when applicable, as they relate to several of the methods highlighted in Figure 8.1.

> ➤ **Improved drainage**
> ➤ **Application of coatings/sealers**
> ➤ **Application of cladding**
> ➤ **Crack filling**
> ➤ **Application of lithium compounds**
> ➤ **Application of restraint (FRP, etc.)**
> ➤ **Saw cutting/slot cutting**

Figure 8.1. Mitigation Methods for ASR-Affected Concrete Structures

The first four methods highlighted in Figure 8.1 are all aimed at reducing the relative humidity in concrete, and for convenience, these methods will be discussed together in Section 8.1. Section 8.2 will describe various methods of applying lithium-based compounds to ASR-affected structures. Lastly, methods aimed at restraining ASR-induced expansion and relieving ASR-induced stress are briefly discussed in Sections 8.3 and 8.4, respectively.

8.3 REDUCING INTERNAL RELATIVE HUMIDITY

Moisture is an essential component of ASR-induced expansion and cracking. Pedneault (1996) showed that below a relative humidity (RH) of 80 percent, ASR-induced expansion is significantly reduced or suppressed, as shown in Figure 8.2.
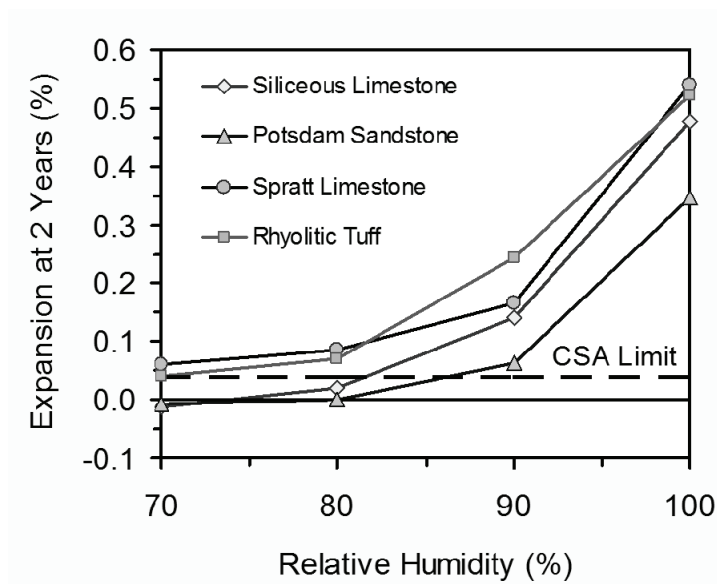

Figure 8.2. Influence of Relative Humidity on Expansion due to ASR (after Pedneault, 1998)

Given the importance of moisture availability on ASR, any methods that can be applied in the field to reduce the internal RH are worthwhile. This can involve improving drainage for a given structure, for example, by diverting drainage from a bridge deck away from an ASR-affected column. The application of exterior cladding that prevents the ingress of additional moisture may be beneficial, but it should be noted that the moisture already present within the concrete may be sufficient for ASR to remain active, and this fact must be considered when contemplating a cladding as a mitigation measure.

A more sound solution with regard to reducing internal moisture is to apply a coating or sealer that prevents external water from penetrating into the concrete, but also allows water vapor from within the concrete to exit, thereby resulting in an overall decrease in the internal relative humidity. There are a range of products, some proprietary in nature, that satisfy the above characteristics, but siloxanes, and especially silanes, tend to be most suitable and are, as a result, the most commonly used as mitigation measures for not only ASR, but also to help reduce the ingress of water (to enhance frost resistance) and external chlorides (to reduce the rate of corrosion of reinforcing steel). In recent years, silanes have become the most important and most widely used product for these purposes. There are a variety of silane products available, varying primarily based on the concentration of silane in the specific formulation (ranging from 20 percent to close to 100 percent) and based on the type of carrier with which the silane is combined (either water-based or solvent-based). More stringent restrictions regarding VOC emissions have resulted in more water-based silanes or solvent-based silanes with higher silane contents (and thus lower solvent content and reduced VOCs).

There have been several studies that have confirmed the benefits of applying siloxanes, and especially silanes, to field structures to reduce future ASR-induced expansion. The research by Bérubé et al. (2002) was particularly encouraging as it showed that applying silane to highway barriers heavily damaged by ASR resulted in a dramatic reduction in cracking (Figure 8.3), as well as future expansion (Figure 8.4). Several FHWA-funded field trials over the past few years have included the topical application of silane-based products. Figure 8.5 shows the significant reduction in cracking of highway barriers in Massachusetts, approximately three years after the topical application of silane.
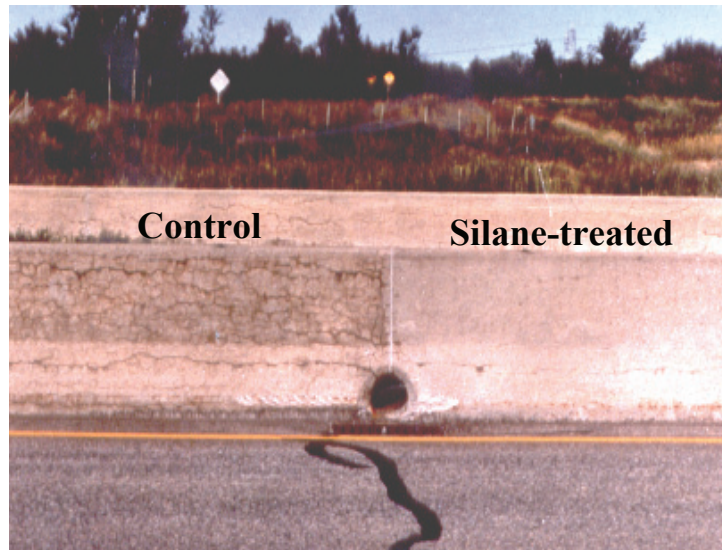
Figure 8.3. Reduction in cracking of highway barriers in Canada, after application of silane
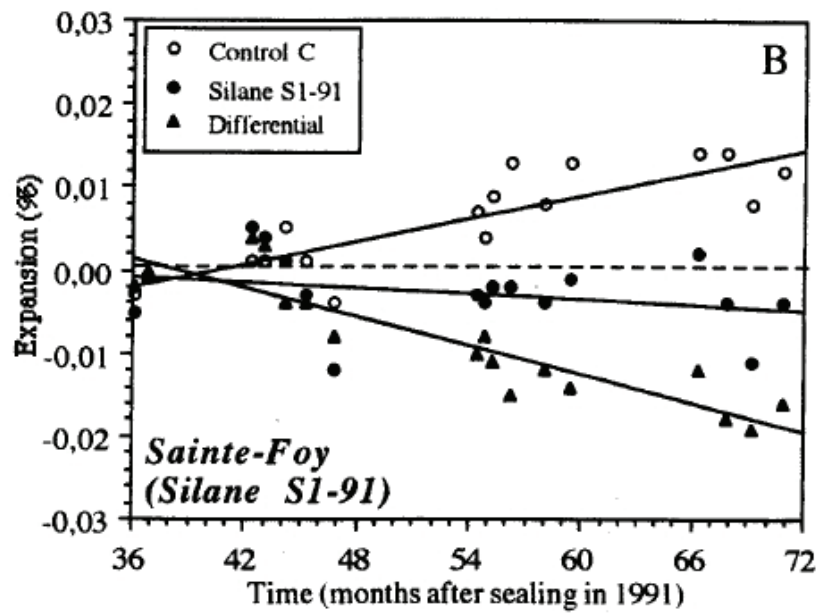(Bérubé et al. 2002)

Figure 8.4. Reduction in expansion of highway barriers in Canada, after application of silane
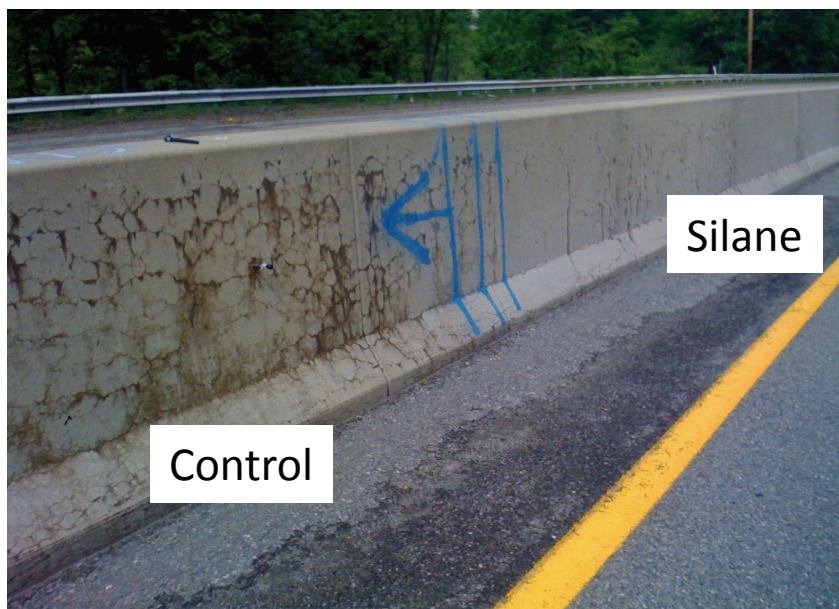(Bérubé et al. 2002)

Figure 8.5. Reduction in cracking of highway barriers in Massachusetts, after application of silane (FHWA Field Trials)

As described, silanes can be quite effective in the field in reducing ASR-induced expansion, but there are certainly limitations. For pavements, slabs on grade, wingwalls, or other applications where moisture is available from below (or beneath), silanes will likely not be as effective as their benefits are only realized from the treated surface. To quantify the effects of treating such elements with silane, FHWA-funded field trials have in recent years included the treatment of wingwalls in Maine and Rhode Island and pavements in Arkansas; it is too early to determine what benefits, if any, will be derived from such treatments.

Lastly, it should be noted that the application of silanes will not be effective in concrete with large crack widths. For these larger cracks, flexible caulking or similar products should be used to seal the larger cracks. In a FHWA field trial in Wetumpka, Alabama, flexible caulking was applied to selected cracks in a bridge that were deemed to be too wide to be effectively treated by silanes. There have been other recent developments, including the use of "high-build" paints or elastomeric coatings that may show promise in bridging larger cracks and avoiding the need for caulking of individual cracks. The need to seal larger cracks becomes critical when reinforced concrete is exposed to external chlorides or in regions exposed to cycles of freezing and thawing. These elastomeric coatings have recently been applied to bridge structures in Maine and Vermont and highway barriers in Massachusetts as part of ongoing FHWA field trials; again, it is too premature to make any conclusions about the efficacy of such applications, but monitoring is ongoing.

8.4 APPLICATION OF LITHIUM COMPOUNDS

There have been several laboratory-based publications related to using lithium compounds to treat concrete already suffering from ASR-induced expansion. Research by Stark et al. (1993), Stokes et al. (2000), and Barborak et al. (2004) have shown that lithium compounds can reduce future expansion of small, ASR-affected concrete specimens in accelerated laboratory tests. Although the mechanism is not fully understood, it is generally believed that lithium compounds enter into the existing gel and change the nature and behavior of the gel from expansive to essentially non-expansive. Because of these positive results in laboratory-based work, there has been considerable interest in treating ASR-affected field structures, especially in recent years under FHWA-funded research. A detailed review of past field trials using lithium compounds can be found in Folliard et al. (2006), and several field trials are still being monitored under current FHWA projects. The most common method of applying lithium compounds in field trials has been via topical application, primarily for pavements (Figure 8.6), highway barriers (Figure 8.7(a)), and bridge decks. There have also been a handful of field trials where lithium was applied either by vacuum (Figure 8.7(b)) or through electrochemical means, both aimed at increasing the depth of penetration of lithium.

Figure 8.6. Photograph showing topical application of 30%-$LiNO_3$ solution to concrete pavement in Idaho

a.   b.

Figure 8.7. Photograph showing topical application (a) and vacuum application (b) of 30%-LiNO$_3$ solution to highway barrier in Massachusetts

Although lithium compounds have been found to be effective in laboratory-based research, which has focused on treating small specimens affected by ASR, there is, unfortunately, very little, if any, documentation that lithium is effective in reducing ASR-induced expansion in actual structures in the field. Part of this is due to the general lack of monitoring of field trials in which lithium compounds have been applied to structures (or pavements). However, in recent FHWA-funded field trials (under FHWA Project DTFH61-02-C-00097) in Idaho, Massachusetts, and Texas, which have likely been the most instrumented and monitored lithium-based field trials to date, the depths of penetration of lithium have been measured to be quite minimal, especially for topical applications. In fact, depths of penetration for topical applications in an Idaho pavement were found to be only a few millimeters, with dosages of lithium necessary to suppress expansion measured only down to the first 2 to 3 mm, even after three treatments in heavily cracked pavements. These results are consistent with laboratory evaluations performed under the same project, and when taken as a whole, it appears that due to an inherent lack of penetration, the topical application of lithium compounds shows little, if any, promise of mitigating ASR in structures and pavements.

Because of the documented lack of penetration in field and laboratory trials in which lithium compounds have been applied topically, recent focus has shifted towards more aggressive means of driving lithium into ASR-affected concrete, specifically through vacuum impregnation and electrochemical methods. Unfortunately, in research performed under FHWA Project DTFH61-02-C-00097, vacuum impregnation was not found to be effective in the laboratory or in field structures in Texas and Massachusetts. For example, for ASR-affected bridge columns in which lithium nitrate was applied via vacuum, the depths of lithium penetration were found only to be present in the outer 9 to 12 mm, drawing into question whether such an elaborative and expensive vacuuming technique is justified. Substantially higher depths of penetration were observed in the same study when lithium nitrate was electrochemically driven into bridge columns, with dosages sufficient to reduce ASR measured all the way down to the reinforcing

steel (50 mm from outer surface). Information on the specific details of the electrochemical method used for this bridge structure can be found in East (2007). However, one major concern with this technique is that the electrochemical process, itself, tends to drive alkalis already present in the concrete towards the rebar, which may be a significant obstacle to this technology. It appears that as a whole, this technique is quite powerful in driving external lithium into the concrete, but the rearrangement of internal alkalis and accumulation of sodium and potassium (which in turn leads to an augmented pH near the rebar) is a serious concern that deserves further attention. More work is in progress to evaluate this treatment technique and to quantify the benefits (and downsides) of this approach.

Despite the general lack of penetration observed in laboratory and field structures in which lithium was applied topically or by vacuum, it is hoped that data will be generated from other field trials, thereby increasing the state of knowledge and expanding the database of depth of penetration data. It will especially be quite useful to determine if lithium can help to extend the service life of structures exposed to external deicing salts. Lastly, the success in driving lithium all the way to the reinforcing steel is encouraging, but the adverse effects of pushing sodium and potassium to the vicinity around the steel deserve further attention. Given that lithium compounds have clearly been shown to be effective in reducing future expansion in ASR-affected concrete in the laboratory, and given that the options for treating the cause of ASR in the field are limited, it is hoped that additional lithium-based field trials be conducted and monitored, thereby helping to quantify the effects, if any, of lithium application on remaining service life.

8.5 APPLICATION OF EXTERNAL RESTRAINT

Numerous studies and field trials have shown that physical restraint or confinement (e.g., encapsulation of the affected member by a surrounding non-reactive concrete, applied stress, or reinforcement) can significantly reduce deleterious expansion due to ASR in the direction of restraint (Fournier et al. 2004). Because of the unique nature of this mitigation approach and the fact that the structural response is impacted, it is imperative that a structural engineer play the leading role in specifically designing the methodology for a given ASR-affected structure.

Post tensioning in one or two dimensions, or by encasement in conventional reinforced concrete, is currently used as a means to restore the integrity of the structure; however, it should generally be restricted to relatively small masses of structural concrete because of the huge forces that may result from the expansive process due to ASR (Rotter 1995, CSA 2000). Post-tensioned tendons or cables are considered to be an effective solution for thin arch dams (Singhal and Nuss 1991) or structural members of bridge/highway structures; however, they may be less attractive for large concrete structures because of the necessity of periodic destressing (Rotter 1995). Methods to restrain expansion and movement in mass concrete foundations such as tower bases have also included rock anchors and/or encapsulation (Bérubé et al. 1989).

Strengthening by introducing reinforcement with straps, steel plates, and tensioning through bolts was also found to be effective in providing containment for selected ASR-affected concrete members (Wood and Angus 1995). Wrapping ASR-affected reinforced concrete columns with composite materials (FRP) has also been applied in field structures (Figure 8.8) and more recently in Vermont as part of ongoing FHWA field trials.

Figure 8.8. Photograph showing topical bridge column wrapped with FRP (Carse 1996)

8.6 STRESS RELIEF

For certain applications, such as a pavement suffering from ASR-induced expansion, a viable option to extend the service life is to remove sections of concrete near the joints by saw cutting. Removing these sections is helpful in eliminating joint-related failures and minimizing ride quality issues. The sections that have been removed can be replaced by sound concrete, with careful attention paid to restoring the intended joint details (opening, dowel bars, etc.). This approach has been done on a much larger scale for concrete dams, where large slots have been cut to accommodate future expansion. It should be noted that this approach (saw cutting/joint cutting) only relieves stresses but does nothing to address the root cause of the expansion. It is common for this method to be performed repeatedly as expansion continues and negates the benefits achieved from the previous concrete removal.

8.7 SUMMARY

This section described a variety of methods that have been applied to field structures suffering from ASR-affected expansion and cracking. Some methods, such as the application of silanes,

have shown significant promise, especially when applied to elements such as small columns and highway barriers, whereas other methods, such as the topical application of compounds, have shown little or no promise in reducing ASR-induced expansion and cracking. It is hoped that ongoing FHWA field trials will better quantify the potential benefits of these and other preventive measures and will help to contribute to improved management of ASR-affected structures in the future.

REFERENCES

Barborak, R., Folliard, K.J. and Thomas, M.D.A. 2004. "Using Lithium Compounds to Treat Hardened Concrete Suffering from ASR: Preliminary Laboratory Results." Proceedings of the 12th International Conference on Alkali-Aggregate Reactivity (ICAAR), Beijing, China, 483-489.

Bérubé, M.A., Fournier, B. and Frenette, J. 1989. "Détérioration de fondation de pylônes d'ancrage de lignes de transport d'électricité par des réactions alcalis-granulats, performance mécanique et réparation du béton." Canadian Journal of Civil Engineering, 16,(6), 945-959.

Bérubé, M.A., Chouinard, D., Pigeon, M., Frenette, J., Rivest, M. and Vézina, D. 2002. "Effectiveness of Sealers in Counteracting Alkali-Silica Reaction in Highway Median Barriers Exposed to Wetting and Drying, Freezing and Thawing, and Deicing Salts." Canadian Journal of Civil Engineering, 29(2), 329-337.

Canadian Standards Association (CSA). 2000. "Guide to the Evaluation and Management of Concrete Structures Affected by Alkali-Aggregate Reactions." CSA A864-00, 2000b, Ontario, Canada.

Carse, A. 1996. "The Asset Management of Alkali-Silica Rection in a Long Bridge Structure." Proceedings of the 11th International Conference on Alkali-Aggregate Reaction in Concrete, Quebec City, Canada, (Eds. M.A. Bérubé, B. Fournier and B. Durand), 1025-1032.

East, B. 2007. "Laboratory and Field Investigations on the use of Lithium Nitrate to Prevent or Mitigate Alkali-Silica Reaction." MS Thesis, The University of Texas at Austin.

Pedneault, A. 1996. "Development of Testing and Analytical Procedures for the Evaluation of the Residual Potential of Reaction, Expansion, and Deterioration of Concrete Affected by ASR." M.Sc. Memoir, Laval University, Québec City, Canada, 133p.

Rotter, H.M. 1995. "Alkali-Aggregate Reaction: from Basic Principles to Structural Behaviour – A literature Review." Rapport No. EPM/GSC 1995-11, École Polytechnique de Montréal, Montréal, PQ, 166p.

Singhal, A.C. and Nuss, K.L. 1991. "Cable Anchoring of Deteriorated Arch Dam." Journal of Performance of Constructed Facilities, 5(1), 19-37.

Stark, D., Morgan, B., Okamoto, P. and Diamond, S. 1993. "Eliminating or Minimizing Alkali-Silica Reactivity." SHRP-C-343, National Research Council, Washington, DC.

Stokes, D.B., Thomas, M.D.A. and Shashiprakash, S.G. 2000. "Development of a Lithium-Based Material for Decreasing ASR-Induced Expansion in Hardened Concrete." Proceedings of the 11th International Conference on Alkali-Aggregate Reaction in Concrete, (Eds. M.A. Bérubé, et al.), CRIB, Quebec City, Canada, 1079–1087.

Wood, J.G.M. and Angus, E.C. 1995. "Montrose Bridge: Inspection, Assessment and Remedial Work to a 65 Year Old Bridge with AAR." Structural faults and Repair – 95, 6p.